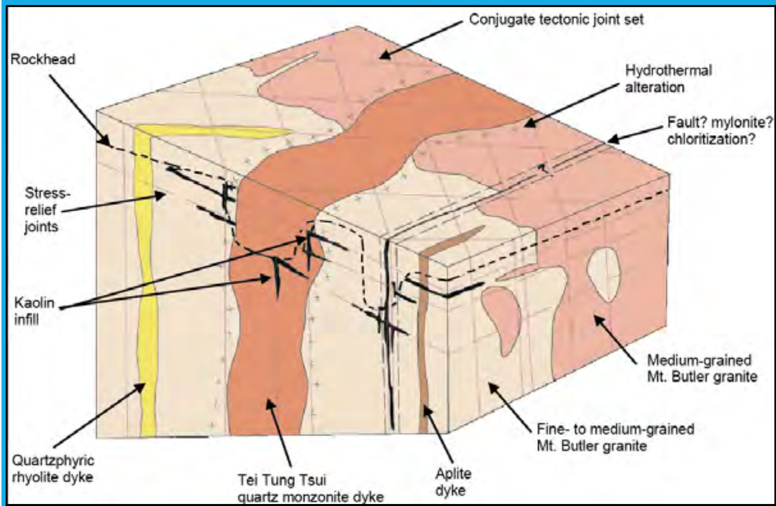
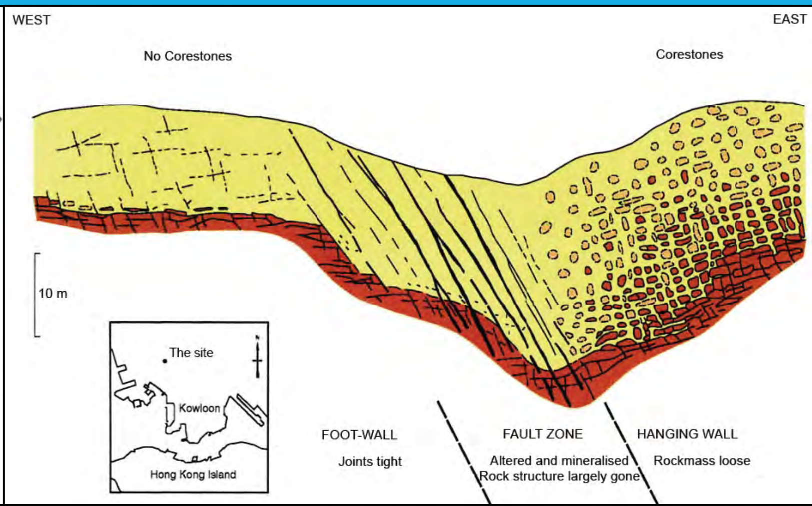
GEO PUBLICATION No. 1/2007

ENGINEERING GEOLOGICAL PRACTICE IN HONG KONG





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

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Captions of Figures on the Front Cover:

Top Left	Example of an initial geological model based on an interpretation of published geological maps (see Section 3.2.1)
Top Right	Irregular rockhead and corestone development influenced by faulting and jointing at a site in northeast Kowloon (see Section 4.4.4)
Bottom Left	Outlines of debris trails from notable natural terrain landslides that occurred above Area 19, Tuen Mun (see Section 6.3.7)
Bottom Right	Moderately-inclined southwest face of the Route 3 Ting Kau cutting which was influenced by adverse geological structures (see Section 6.4.4)

FOREWORD

This publication is intended to enhance geotechnical practice in Hong Kong, and help geotechnical practitioners to recognise when specialist engineering geological expertise should be sought.

The principles of engineering geology as applicable to Hong Kong are introduced, and the application of these principles to civil engineering works are illustrated by means of examples and references. The publication is aimed primarily at experienced geotechnical engineers, to demonstrate the importance of engineering geology to the timely, cost effective and safe completion of civil engineering works. It will also assist experienced engineering geologists in Hong Kong by acting as an *aide mémoire* and will provide a valuable source of information for young and overseas practitioners.

The publication is based on literature reviews and experience of engineering geological practice in Hong Kong. Owing to the broad scope of engineering geology and its wide range of applicability, this publication can provide only relatively limited amounts of information and discussion. However, where available, references are provided to allow more detailed information on each particular subject to be obtained if required.

The publication was prepared by a team led by Dr L.J. Endicott of Maunsell Geotechnical Services Ltd. The team members were Mr J.W. Tattersall (Principal Author), Mr P.G.D. Whiteside, Mr S.J. Williamson, Mr G. Charlesworth and Ms W.S. Ip. Production was overseen by Mr Y.C. Chan and Mr H.N. Wong, and coordinated by Dr K.C. Ng. The latter, together with Mr S. Parry and Dr R.P. Martin were the principal reviewers. Mr J.B. Massey, Dr P.L.R. Pang and Dr L.K.R. Woodrow reviewed the final draft document.

The work was overseen by a Steering Committee chaired by the Head of the Geotechnical Engineering Office of the Civil Engineering and Development Department. Members of the Steering Committee are listed on the next page.

Working papers and previous drafts were circulated to a Working Group comprising representatives of the Geotechnical Engineering Office, professional institutions and learned societies. Members of the Working Group are listed on the next page. Copies of a draft version of this document were circulated to local professional bodies, consulting engineers and academics. Useful comments, many of which have been adopted in finalising this document, were received from many quarters. These contributions are gratefully acknowledged.

As experience and good practice evolve, practitioners are encouraged to comment at any time to the Geotechnical Engineering Office on the content of this publication, so that improvements may be made to future editions.

R.K.S. Chan
Head, Geotechnical Engineering Office
March 2007

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1. INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this document is to introduce the principles of engineering geology as applicable to Hong Kong and to illustrate the application of these principles to civil engineering works, by means of examples and references. The document is aimed primarily at experienced geotechnical practitioners, to demonstrate the importance of engineering geology to the timely, cost effective and safe completion of civil engineering works. It will also assist experienced engineering geologists in Hong Kong by acting as an *aide mémoire* and will provide a valuable source of information for young and overseas practitioners.

The science of geology is concerned with the study of the natural materials and processes that have formed the Earth. A detailed understanding of this science allows representative geological models to be constructed from limited data to characterise what might otherwise appear to be chaotic and unpredictable ground.

Engineering geology provides the link between geology and engineering through the formation of geological models which can be used to identify geological hazards and uncertainty, plan effective ground investigations, and define blocks of ground and geological structures in an engineering context to facilitate geotechnical risk assessment and design. Knill (2002) considers that, to be successful, "*engineering geology must demonstrate a balance between high-quality understanding of geology and a sufficient appreciation of engineering to ensure that the relevant information will be processed and communicated effectively*".

The amount of engineering geological input required for a particular civil engineering project varies depending on geological factors such as rock type, superficial deposits, geological structure and weathering as well as engineering considerations such as the type of scheme and the construction method adopted. This document provides a compendium of knowledge and experience of the various geological settings in Hong Kong in order that problematic conditions can be recognized in a timely fashion and the necessary engineering geological input can be obtained at the appropriate stage of a project.

1.2 LAYOUT

Chapter 2 gives an introduction to current engineering geological practice in Hong Kong. In particular, it covers published guidance and areas identified for improvement with respect to reducing risk and uncertainty.

Chapter 3 deals with engineering geological input to geotechnical works. This involves the development of geological models and their refinement during the engineering processes from planning to maintenance. Emphasis is placed on the value of the 'model approach' for effective anticipation and characterisation of the ground conditions in order to better manage the geotechnical risks. Data requirements at different stages of a project are outlined for reference, with focus on key elements critical for geotechnical investigations and design.

Chapter 4 summarises the basic geological processes that are pertinent to the understanding of the engineering properties of the rocks and soils in Hong Kong. The summaries are based on a consolidated review of documented knowledge and experience in Hong Kong and key references from elsewhere.

Chapter 5 describes the main types of rock and soil in Hong Kong and provides perspective on their engineering geological characteristics. Variations in chemical composition, mineralogy, lithology and block/particle size give rise to different weathering characteristics and geotechnical and hydrogeological properties. Where appropriate, reference to case studies which give insight into aspects of engineering geological interpretation and associated geotechnical problems is also given.

Chapter 6 presents key engineering geological issues and practices which are relevant to the main types of civil engineering applications. These are illustrated by reference to projects giving insight and focus to the engineering geological issues which may need to be considered. Some of the issues, practices and examples are relevant to several engineering applications, and cross-referencing has been used to avoid repetition.

1.3 LIMITATIONS

The document is primarily based on a review of relevant literature and current practice. As such, information on certain topics may be limited in extent. Also, the level of information given for each lithological type is roughly proportional to their engineering importance and distribution with respect to development. Furthermore, owing to the wide range in the application of engineering geological practice, the document can only provide limited, albeit key , information with respect to relevant engineering geological considerations. However, where available, references are provided to allow more detailed information of each particular subject to be obtained if required.

The document is intended to enhance geotechnical practice in Hong Kong, and help geotechnical practitioners to recognise when specialist engineering geological expertise should be sought. The document is not intended to be used as a geotechnical standard, or used as a checklist for diferent types of engineering geological works. Furthermore, the document should not be regarded as a substitute for providing adequate engineering geological input.

Given the nature of the subject it is necessary to use geological terminology within the document, explained in simple terms where necessary. However, the purpose of the book is not to explain geology to engineers and it is expected that readers will obtain more detailed geological background material from other sources if required.

2. INTRODUCTION TO ENGINEERING GEOLOGICAL PRACTICE

2.1 INTRODUCTION

Engineering geological practice is primarily concerned with the determination of geological and hydrogeological conditions to facilitate ground engineering with respect to the recognition and management of geotechnical risk. This requires the application of geological knowledge and skills to define and communicate the potential and actual variations in ground conditions that are relevant to the engineering project at hand.

The ground in Hong Kong has the potential to be geotechnically complex as a result of geological variations. However, this complexity may not be random or unpredictable, but is the result of genetic and process-related geological and anthropogenic factors that have contributed to the present-day ground conditions. Much of this complexity can be anticipated, identified, understood and quantified through the application of sound engineering geological principles. It therefore follows that one of the most cost-effective measures that can be taken for any project involving geotechnical works is to exercise good engineering geological practice in the planning, execution and interpretation of site investigations. The primary aim is to increase the recognition of 'foreseeable' ground conditions which need to be investigated, in order to reduce the risk of 'unforeseen' ground conditions being encountered at a later stage.

Chan & Kumaraswamy (1995) report in a survey that 'unforeseen ground conditions' was cited as the most significant factor in causing construction delays to civil engineering works in Hong Kong. Unforeseen ground conditions have also been cited as major factors in a number of large man-made slope failures in Hong Kong (Wong & Ho, 2000a; Ho *et al.*, 2003). Two of the main contributing factors relevant to engineering geological practice were:

- the presence of adverse geological features and/or adverse groundwater conditions, and
- the use of an over-simplified geological and/or hydrogeological model which does not adequately cater for safety-critical geological features in the ground.

Similar observations have also been reported with respect to international civil engineering practice (Site Investigation Steering Group, 1993a,b; Hoek & Palmieri, 1998; Morgenstern, 2000; BTS/ABI, 2003, 2004).

Only a tiny fraction of the volume of ground which will affect or be affected by the proposed works can usually be observed directly or tested during a site investigation. Therefore, the risk of 'unforeseen ground conditions' has the potential to increase with geological complexity.

Good engineering geological practice evolves in response to improvements in local and international knowledge, experience and technology, which are largely based on observations and lessons learnt from well documented studies and case histories. Good engineering geological practice facilitates effective recognition and resolution of geotechnical problems through the application of fundamental geological principles, local knowledge and precedent, thereby enhancing engineering practice in general.

2.2 DEFINITION

Geology is the study of the Earth; it embraces knowledge of geological materials (characteristically soils and rocks) and the processes that formed them and that currently transform them. Engineering geology is the application of the science of geology to the technology of ground engineering. The subject requires a comprehensive knowledge of geology, as well as an understanding of engineering properties and behaviour of the geological materials. The practice involves site investigation and site characterisation specific to the needs of the engineering project. In outline, the investigation should cover the area of terrain that is affected by the project, and any adjacent terrain from which geological processes could affect the project, such as the natural hillside above the project site, from where a landslide could impact on the site.

The characterisation of the site includes the identification of the geological materials and structures present, their extent and disposition. This includes the integration of relevant geological processes to enable a realistic geological model of the site to be formed (see Section 3.1.2). This model includes engineering descriptions to characterise the relevant materials and discontinuities, and to facilitate the formation of a representative ground model which includes engineering parameters (see Section 3.1.3). The ground model characterises the site in an integrated manner to enable assessments of geotechnical hazard and engineering design options (see Section 3.1.4 - design model).

The output of engineering geological practice primarily consists of the geological model and advice to engineers and others involved with the project regarding development of the ground and design models.

During the progress of the project, as more information becomes available, the models can be updated and refined to reduce geotechnical uncertainties. This is particularly important where the design model needs to be verified by site observations during construction (see Section 3.1.5).

2.3 EXISTING GUIDANCE ON ENGINEERING GEOLOGICAL PRACTICE

The Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) gives guidance on standards for geotechnical engineering in Hong Kong. This includes publications and Technical Guidance Notes (TGN). TGN 1 (GEO, 2005d) provides a list of publications which are used by GEO as *de facto* standards. Some of these also cover engineering geological issues and practice. The TGNs and many other relevant documents can be downloaded from the Civil Engineering and Development Department's (CEDD) website http://www.cedd.gov.hk. This website also contains an interactive online bibliography on the geology and geotechnical engineering of Hong Kong.

The TGNs are updated regularly, primarily in response to improvements in geotechnology, better understanding of local geological conditions, and geotechnical lessons learnt both in Hong Kong and elsewhere. This evolutionary process means that existing guidance should be viewed as minimum standards of practice applicable when each document was promulgated or revised. Good engineering geological practice requires that the existing guidance and reference documents are adapted and further developed as necessary in response to advances in knowledge and technology, and with respect to the site-specific conditions and requirements of the project at hand.

2.4 RECOMMENDATIONS FOR IMPROVEMENTS IN ENGINEERING GEOLOGICAL PRACTICE

The importance of engineering geology in slope engineering and the need for improved assessment and design practices have been highlighted by many authors, e.g. Wong & Ho (2000a); Campbell & Parry (2002); Ho *et al.* (2003); Martin (2003); GEO (2004b,c,d,e).

Key areas for improvement in slope engineering, based primarily on Martin (2003) and Ho *et al.* (2003), which are also applicable to other engineering applications, include:

- Increased awareness among all geotechnical professionals that the heterogeneity of the ground conditions renders the assessment of appropriate geological models and design groundwater conditions difficult. This calls for rigorous engineering geological input and a holistic approach in the anticipation, understanding and characterisation of ground conditions.
- Allowance for uncertainty and continued engineering geological review during design and construction when judging the degree of adversity of the geological and hydrogeological conditions. In particular, judgement about the significance of adversely-orientated discontinuities needs to be exercised. Uncertainties and assumptions with respect to the ground conditions should be regularly reviewed and verified by experienced personnel, and documented before the end of contract maintenance periods. As-built engineering geological records should be included in maintenance manuals for future reference.
- Early recognition of potentially problematic sites with unfavourable ground and groundwater conditions that require special attention, and rigorous geotechnical and engineering geological input to facilitate integrated assessments.
- Increased appreciation of landscape evolution to assess sites in a regional geological and geomorphological context.
- More detailed site reconnaissance to assess the overall engineering geological setting and performance history of the site and its surroundings. Increased attention should be given to examining the ground beyond the margins of the site, especially natural terrain.
- More emphasis on appraisal of relict discontinuities in saprolite and potential transient perched water tables.

- More detailed and considered hydrogeological assessments to determine groundwater monitoring requirements and the use of continuous monitoring devices.
- Consideration and identification of possible changes to environmental conditions which may adversely affect the groundwater regime.
- Increased awareness and recognition of features which pre-dispose the ground to time-dependent changes that could adversely affect its stability or deformation characteristics (e.g. steeply-inclined relict joints and other geological weaknesses) and consideration of the effects of stress-relief, groundwater ingress and possible development and/or blockage of soil pipes.
- Increased application and integration of soil mechanics and rock mechanics principles in conjunction with engineering geological assessment of mass properties with due regard to geomorphology, hydrology and discontinuities.
- Use of a formal risk management framework to identify and assess potential impacts due to geotechnical hazards, so as to provide a rational basis for the determination of the most appropriate design and construction strategies.

Fundamental to these recommendations is the need to systematically develop geological, including geomorphological and hydrogeological, models to facilitate the planning of site investigations and engineering designs. These models should be updated on a regular basis throughout the design and construction processes to increase awareness of potential geological uncertainties and geotechnical hazards. This facilitates the checking and verification of the design, and helps to form the basis of geotechnical risk analysis and management frameworks that are becoming increasingly required by clients, contractors, and insurance underwriters for large projects.

The use of geological models and their role in reducing geotechnical risk are reflected in GEO (2005a,b) and Pang *et al.* (2006). Although these documents are concerned with tunnelling works, the fundamental principles can also be applied to other engineering applications, with due consideration being made to the nature and consequences of non-performance of the works during the construction and post construction stages.

3. ENGINEERING GEOLOGICAL INPUT TO GEOTECHNICAL WORKS

3.1 MODEL APPROACH

3.1.1 Introduction

For geotechnical applications, models are developed with varying degrees of rigour to:

- consider potential variations in ground conditions,
- determine site investigation requirements, and
- facilitate the interpretation of the ground conditions to provide a basis for design.

In order to provide a framework for the input of engineering geological work, a three-step approach comprising 'geological', 'ground' and 'design' models, based on local and international recommendations is adopted. The degree to which these steps are applicable to a speci fic engineering project and the level of engineering geological input required will depend on the nature and scale of the engineering works and perceived geotechnical risks. However, the development of a geological model is the first step towards the assessment of geotechnical risks for most engineering projects.

An international perspective on the positions of engineering geology , soil mechanics and rock mechanics within the broad field of 'geo-engineering', including their respective international societies (i.e. IAEG - International Association for Engineering Geology and the Environment, ISSMGE - International Society for Soil Mechanics and Geotechnical Engineering and ISRM - International Society for Rock Mechanics), is provided in JEWG (2004) and discussed in Bock (2006). An interpretation of this overall framework is shown in Figure 3.1.1. While some details may need adapting to suit different engineering applications and local geotechnical practice, the positions of the geological model and ground model are clearly shown. The design model is represented in the diagram by the interface of the 'geo-engineering' triangle with the 'geo-engineering structure'. A fundamental concept

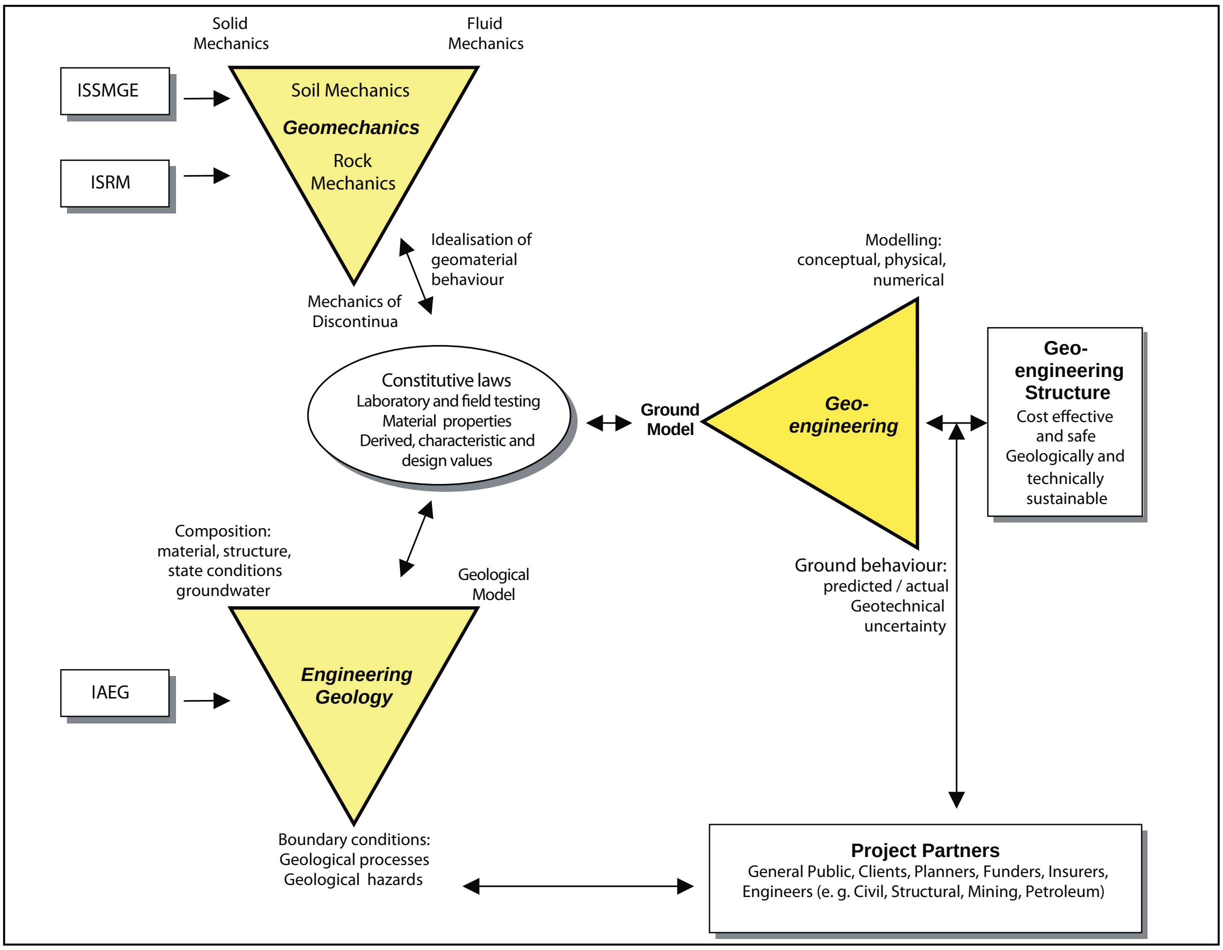


Figure 3.1.1 – International perspective of the positions of engineering geology, soil mechanics and rock mechanics within 'geo-engineering' practice (after Bock, 2006)

of this framework is that the ground model should be developed through the interactive efforts of all the relevant team members. This is to ensure, as far as practicable, that the model is representative of all the site conditions that are relevant to the project.

For projects where good geotechnical practice is applied, well researched and documented models will be developed to illustrate the anticipated range of ground conditions and effectively target the investigations towards reducing geological uncertainty and geotechnical risk. Initial models will be updated during the investigation, design and construction processes such that all the team members are fully aware of the relevance of the findings at all stages of the project. Hence, any changes to the design or construction methods that might be required due to unexpected conditions can be implemented in a timely manner.

For projects where poor geotechnical practice is applied, the model approach is either not considered or is poorly implemented. This can result in an inadequate desk study with little in the way of skilled input or good documentation. The site investigation may also be planned in a prescriptive manner which may not be effective in reducing geological uncertainty and geotechnical risk. Communication between members of the investigation, design and construction personnel may also be poor, and design reviews (if any) may be conducted by inadequately skilled and experienced staff. In such cases, safety-critical inadequacies in interpretation and design assumptions are less likely to be recognised during the design checking and construction stages.

Examples of the use of engineering geology for the development of the various types of model are given in Sections 3.2 to 3.6 with reference to the engineering geological issues discussed in Chapters 4, 5 and 6.

It should be noted that in Chapters 3, 4 and 5, the word 'rock' is used in a primarily geological sense to include saprolite soil (in engineering terms) derived from chemical decomposition of rock (in engineering terms) unless otherwise noted.

3.1.2 Geological Model

The concept of geological models is not new. GCO (1987b) states " *Before commencing ground investigation, all relevant information collected.... should be considered together to obtain a preliminary conception of the ground conditions and the engineering problems that may be involved.*". The importance of the geological model has been recognised as one of the key components of geotechnical design in BD (2003): *"it is always a good practice to first formulate a preliminary geological model based on existing information obtained from a thorough desk study. The ground investigation fieldwork should then be planned with the objective of refining and confirming the geological model and the parameters to be used in the design, and identifying the various uncertainties involved as far as possible."*. The use of geological models for foundation works design is further discussed in GEO (2006). GEO (2004b) also stresses that " *the geological model assumed for design should be verified during construction and the verified information, including any amendments made to the design geological model during slope works, should be incorporated as part of the as-built records.*".

The term "geological" model used in this document refers to a geological model that characterises the site, i.e. it focuses on geological, geomorphological and hydrogeological features and characteristics that are relevant to the engineering project. A site may for instance be geologically complex; however, this does not necessarily imply that it is also geotechnically difficult for the engineering application. The focus of the model will also depend on the nature of the project. For instance a geological model for a cut and cover tunnel in superficial deposits will have a different emphasis from that for a deep tunnel in rock at the same location. The main elements of the geological model are diagrammatically shown in Figure 3.1.2.

By its very nature a geological model is conceptual

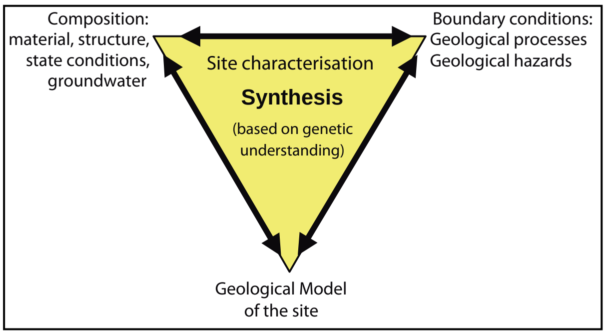


Figure 3.1.2 – Main elements of the geological model (Bock, 2006)

in that it is initially based on an examination of regional and local geological conditions, which are assessed in terms of the potential geotechnical significance of the site's geological history. As such it draws on engineering geological knowledge, skills and experience to anticipate variations in material properties and boundaries in three dimensions. How this model is presented can vary depending on the complexity of the site and the nature of the works being undertaken. Fookes (1997a) notes that models can be presented in written descriptions, two-dimensional sections and plans or block diagrams, and may be slanted towards some particular aspect such as groundwater, geomorphology, or rock structure, i.e. it focuses on the engineering needs of the project.

In its simplest form a geological model can be constructed from an interpretation of a geological map or a site reconnaissance.

The geographical extent of the model will depend primarily on the type of proposed works and the hazards that may be relevant. For example, when considering landslides, the extent of the model may have to be widened to include nearby terrain with similar geomorphology. To assess the effects of tunnelling or deep excavation on hydrogeology, the extent of the model may also need to extend a considerable distance from the site of the works.

It is good practice to refine and update the model during the ground investigation and construction phases as new information is obtained, with reviews undertaken by suitably skilled personnel. Such reviews can reduce the possibility of errors and misinterpretations which could have an adverse impact on the relevance and effectiveness of the site investigation, design and construction methodology.

3.1.3 Ground Model

The ground model builds on the geological model and embeds the range of engineering parameters and ground conditions that need to be considered in the design (Knill, 2002). The ground model refines the geological model by defining and characterising bodies of ground with similar engineering properties, and identifies boundaries at which changes in geotechnical conditions may occur. Engineering geological input assists in ensuring as far as practicable that the ground model reflects the ground conditions indicated by the geological model. Such input is useful in ensuring that stability-critical or performance-critical features such as faults, dykes, discontinuities and hydrogeological boundaries are considered and, if necessary, incorporated. This enables critical features to be targeted for more detailed ground investigation, testing and characterisation if necessary. For maximum cost-effectiveness and design reliability, a multi-disciplinary approach with integration of engineering geological input to the design is beneficial (Figure 3.1.1).

The ground model gives due consideration to the possible ranges of material and mass properties. Environmental factors such as the groundwater regime, contamination, *in situ* stress conditions, and qualitative estimates of the possible ground and groundwater response to the changes in environmental conditions imposed by the proposed works may also need to be considered.

The ground model should include plans and sections through critical areas to indicate the possible range of ground conditions. It should convey an understanding of these conditions, geotechnical hazards and areas of uncertainty that is commensurate with the nature of the proposed engineering works. For example, a ground model for a slope engineering project will need to focus on stability-critical features, while a ground model for a foundation engineering project will need to focus on features that will affect the type and design of foundations.

For large projects where the basic details of the proposed works are known or can be adequately estimated, any geotechnical uncertainty can be incorporated into preliminary risk registers which can then be used during the design stage to target further investigations. These registers can be audited and traced by the design team throughout the rest of the investigation and design process as part of the overall risk management strategy. This approach can also be adapted to suit the needs of smaller projects, depending on the nature and consequences of the perceived risks.

3.1.4 Design Model

The design model is concerned primarily with assessment of the response of the ground to the proposed works and vice versa for use in geotechnical assessment or engineering design. Design models for empirical, prescriptive and quantitative designs

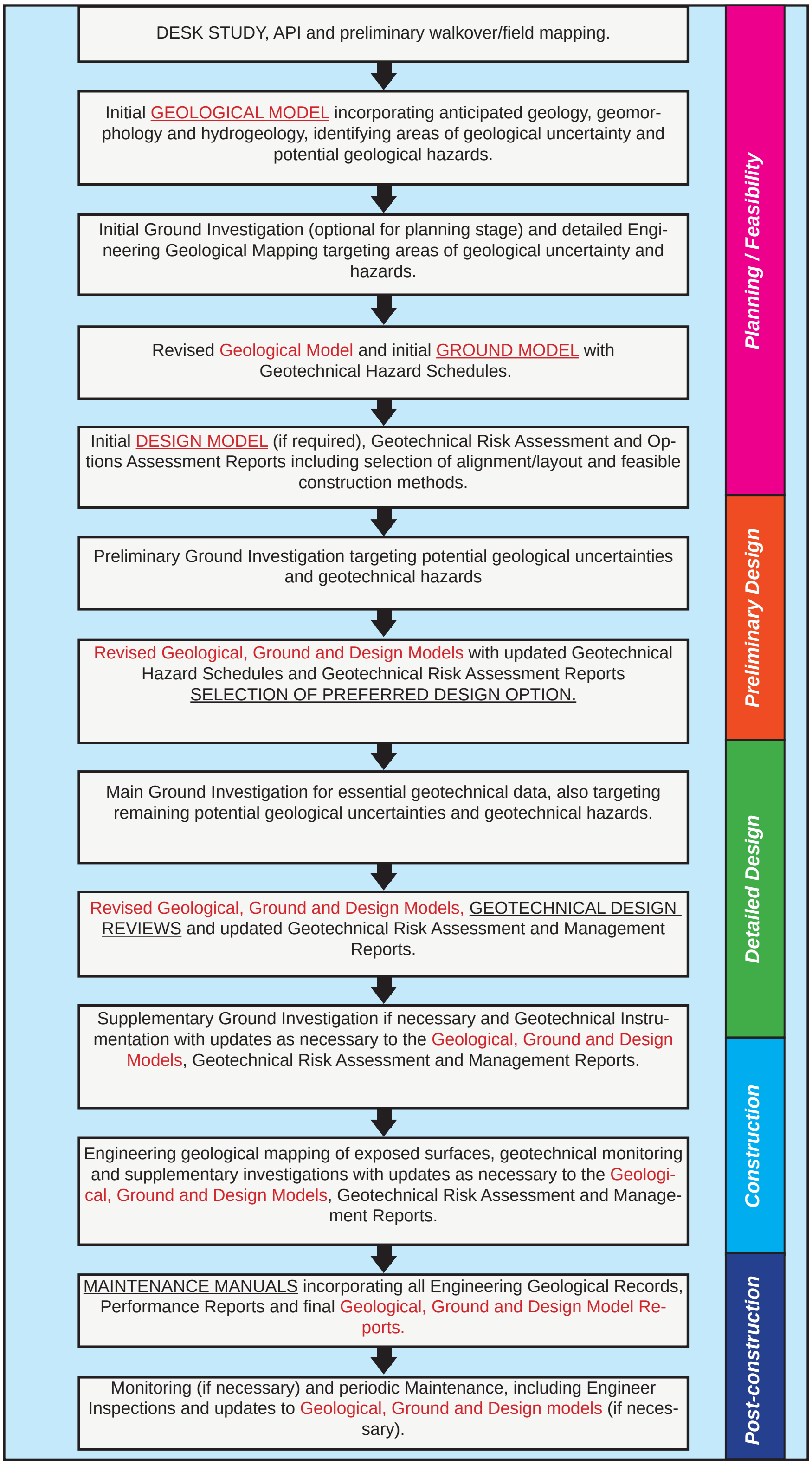


Figure 3.1.3 – Typical development and application of the 'Model Approach' for a major project

depend on the engineering application, degree of conservatism in the empirical/prescriptive models and the level of geotechnical risk.

An example of an empirical design approach is the assessment of allowable bearing capacity for foundations on rocks based on presumed values derived from empirical correlation (BD, 2004a). In this case the ground model would typically comprise a series of plans and sections indicating the variations in decomposition grade and percentage of core recovery based on the results of the ground investigations. The ground model could be used for preliminary purposes to identify the level at which the ground may satisfy the requirements of the foundation design (i.e. as part of the initial design model).

The design of soil nailed slopes in accordance with GEO (2004n) and Wong *et al.* (1999) provides an example of a prescriptive design approach. In this case, the geological models and ground models are first constructed to provide an initial check on whether the slope satisfies the geotechnical and geometrical qualifying criteria for the application of the prescriptive design methodology recommended by Wong *et al.* (1999).

Unless the design is based on empirical or prescriptive approaches, some method of numerical analysis is required. Knill (2002) considers that the steps which need to be taken to convert a geological model, through the ground model, to the design model (i.e. Knill's "geotechnical model") will require refinement to meet the requirements of the selected method of engineering analysis. During the conversion, engineering geological input is essential to ensure that the actual conditions are represented as accurately as possible in the eventual analysis.

The design model therefore incorporates and simplifies the main elements of the ground model so that a representative range of ground conditions can be defined for use within a suitable design framework. In all but the simplest cases, it is advisable that the design model be reviewed to ensure that it adequately incorporates all the safety-critical engineering geological features in the ground model. Furthermore, when additional ground information becomes available, for instance during the excavations for the works, the ground model should be reviewed to identify any new features which might require revision of the design model.

3.1.5 Application

The typical development and application of the model approach for a major project is shown in Figure 3.1.3. Although the chart depicts a linear progression from one activity to the next, there is normally considerable overlap and iteration in practice.

Engineering geological input is particularly effective from the planning and feasibility stages, through to the stage when all site investigation data has been interpreted and incorporated into the design models. Engineering geological mapping of exposed ground during construction also assists in confirming the ground conditions to facilitate verification of the design assumptions, particularly where the final design is based on the 'Observational Method'(GEO, 2005b).

Application of an appropriate level of engineering geological skill and perspective usually enables a large percentage of the geotechnical characteristics of the area of interest to be anticipated at an early stage. Timely identification of areas of uncertainty and potential hazards enables subsequent ground investigations to be efficiently focused, thereby reducing costs and the risk that 'unforeseen ground conditions' may be encountered during construction.

Continuous review of the geological, ground and design models throughout the different stages of a project should be undertaken as more information about the site is obtained, and the models and risk assessments updated as necessary.

Inclusion of as-built engineering geological records in maintenance manuals for the completed works is useful for the purposes of reviewing post-construction performance.

3.2 DESK STUDY AND SITE RECONNAISSANCE

3.2.1 Introduction

This section outlines the main engineering geological considerations that facilitate the initial development of geological models based on a review of existing data and site reconnaissance.

The main data sources for developing an initial geological model are geological maps, aerial photographs, archival ground investigation data,

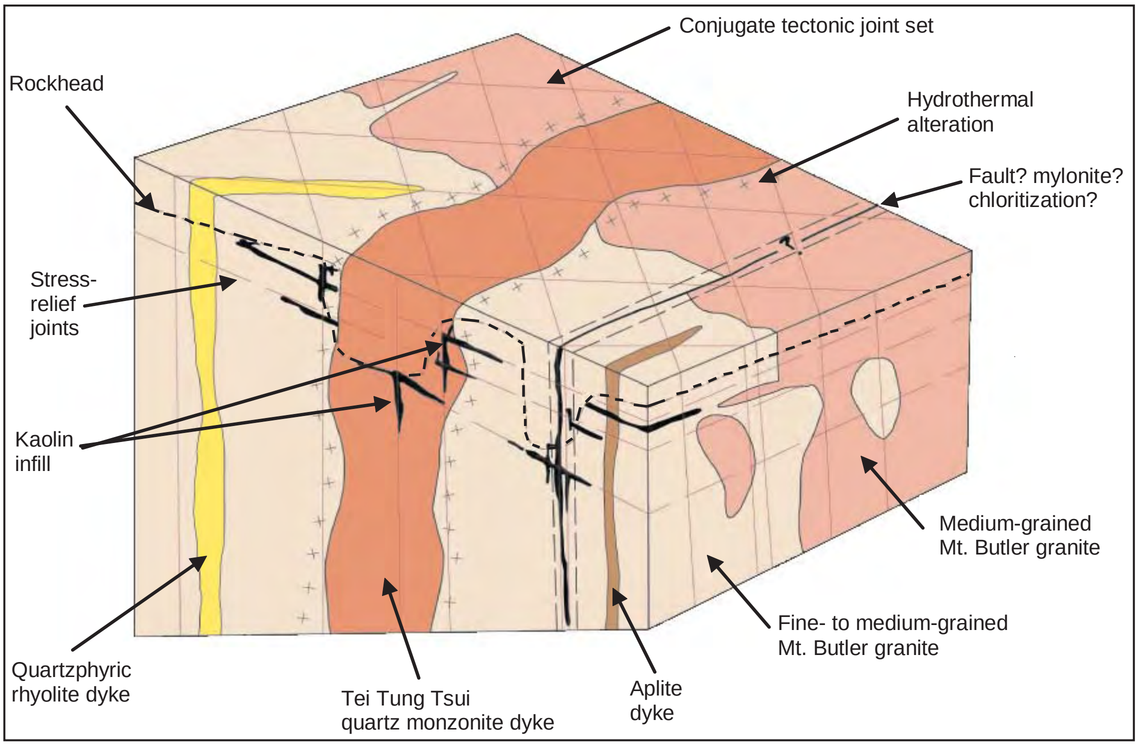


Figure 3.2.1 – Example of a geological model based on a desk study (Parry et al., 2004b)

foundation records and a site reconnaissance. A listing of useful sources of existing information in Hong Kong is contained in GEO (2004f).

The detail and scope of the desk study will depend on the nature and scale of the particular project. The initial geological model is developed during the desk study, modified following site mapping and then revised again after site-specific ground investigation results are available.

Parry *et al.* (2004b) provide an illustrative approach to the development of geological models, based largely on the review of existing data and a site reconnaissance. Figure 3.2.1 shows a block model which illustrates the range of engineering geological conditions that may be present at a site based on an evaluation of the 1:20,000-scale geological map. More detailed, site-specific models can be developed following a site reconnaissance (Figure 3.2.2).

Although there may be considerable overlap between each successive stage, the initial geological model should be as well developed as possible before planning and carrying out any major ground investigation works.

3.2.2 Geological Maps

The existing Hong Kong Geological Survey territory-wide 1:100,000-scale and 1:20,000-scale geological maps, plus 1:5,000-scale coverage in specific areas, and their associated memoirs provide the initial starting point for developing the geological model. These contain information on the spatial distribution of the various stratigraphic units, main known and inferred geological structures and the main rock forming and rock modifying processes. Careful interpretation of geological maps allows preliminary evaluation of the possible geological conditions and their likely variations at a site.

However, the geology shown on the published geological maps is based on interpretations of limited data available at the time of compilation and is constrained in detail due to the scale at which the maps have been produced. In addition, the geological maps do not show variations in weathering patterns and do not show superficial deposits which are considered to be less than about 2 m in thickness.The information shown on the published geological maps is mostly interpretative rather than factual and is unlikely to meet the needs of an engineering project without further engineering geological mapping, interpretation and site investigation. Some examples of the differences between the geology shown on published maps and the geology encountered during

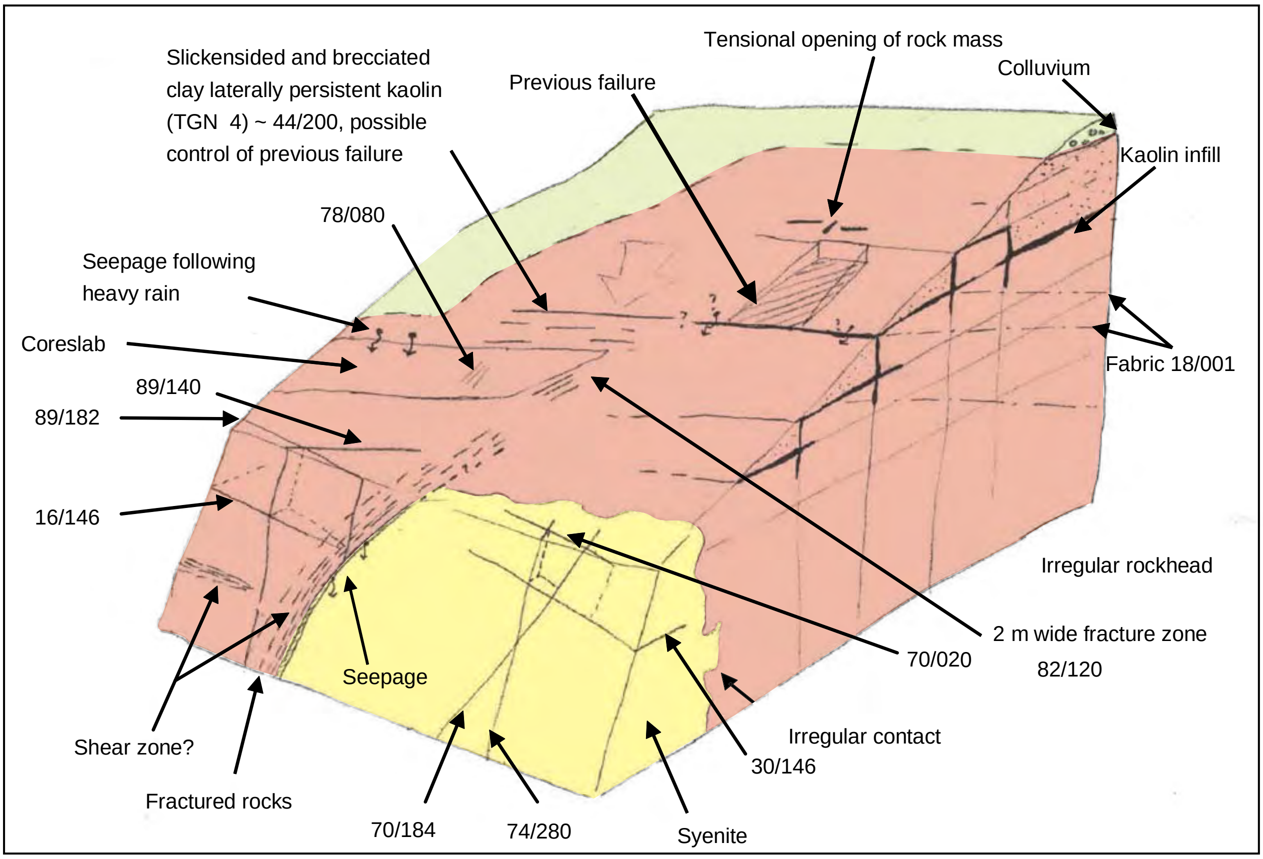


Figure 3.2.2 – Example of a geological model based on site reconnaissance (Parry et al., 2004b)

construction are shown in Section 6.7 (Figures 6.7.1 and 6.7.3). These limitations make it essential that adequate engineering geological knowledge and skills are used to assist in the development of realistic geological models at an appropriate scale for the proposed works.

Where previous geotechnical work has been carried out, archival ground investigation records and as-built construction records may provide valuable information to re fine the understanding of the geology of the area of interest. The principal source of archived ground investigation records is the Geotechnical Information Unit (GIU) which is maintained by GEO. In other areas, greater reliance has to be placed on the published geological maps and memoirs, engineering geological knowledge and experience, aerial photograph interpretation, field mapping and project-specific ground investigations.

3.2.3 Aerial Photograph Interpretation

Aerial photograph interpretation (API) is an essential element for the development of geological models. As it requires considerable interpretative skills, particularly in the analysis of geomorphology (see Chapter 4), API should be undertaken by , or under the close supervision of, an experienced and skilled professional.

The objective of the API is to examine and interpret the existing aerial photographic record relevant to the proposed engineering works. Given Hong Kong's extensive coverage of aerial photographs since the 1960s, detailed site histories can also be documented.

The first complete aerial photograph coverage of Hong Kong was undertaken in 1963. These high quality, low-altitude (c. 1200 m) aerial photographs, taken at a time of generally low vegetation cover in Hong Kong, enable interpretations of subtle ground features to be made. Consequently , these provide a 'baseline' for comparison with subsequent observations. This set is particularly useful for geomorphological interpretation.

While the 1963 set provides much useful information, viewing several sets of photographs is necessary to obtain different views from different orientations and times of day (e.g. low-angle of the sun makes for better definition of features). The most recent aerial photographs provide a useful check on the currency of the topographical maps and provide information on the condition of the study area. Observations from these recent photographs can also be useful for planning the site reconnaissance by identifying suitable locations for an overview of the site and possible access points.

For most sites it is necessary to record changes over time. In such cases a systematic evaluation of all available aerial photographs should be made. In addition to vertical aerial photographs, a collection of oblique aerial photographs dating back to the late 1970s is held in the Planning Division of GEO. Some areas in Hong Kong have been photographed using infrared aerial photography, which may be useful for identifying vegetation cover and areas of seepage.

Many of the existing territory-wide terrain datasets were derived from the interpretation of aerial photographs, e.g. Terrain Classification Maps (Styles & Hansen, 1989), Natural Terrain Landslide Inventory (King, 1999), Large Landslide Dataset (Scott Wilson, 1999), and Boulder Field Inventory (Emery, 1998). These data were prepared under various constraints. Consequently, these datasets should be compared with the results of the site-specific API in order to evaluate the relevance of the desk study information for the site of interest.

All observations should be shown on a plan. However, it is useful for purposes of presentation and auditing to record the observations directly onto scanned copies of the aerial photographs. The use of aerial photographs that have been ortho-rectified (ortho-photographs) combined with contour data is beneficial in that it allows the accurate location of features and enables scaled measurements to be made. Territory wide ortho-photographs have been prepared by GEO for selected years and by Lands Department since 2000.

The recording of features should be carried out using a well-defined legend that includes all the relevant aspects of geomorphology and geological features covered by the mapping. Examples of such legends are contained in Anon (1982).

The quality of an aerial photograph interpretation is directly related to the skill and experience of the interpreter. The API should be re-evaluated after site inspections have been carried out. The best results are obtained when the API and site inspections are carried out by the same personnel to allow for continuity and integration of information.

Parry & Ruse (2002) show examples of the use of API and geomorphological interpretation in developing geological models. Examples of the use of API in connection with natural terrain hazard assessment, slope engineering and tunnelling works are contained in Sections 6.2, 6.4 and 6.7 respectively. Additional guidelines on the interpretation of aerial photographs for geomorphological mapping are contained in GEO (2004g) and aspects of this are discussed in Section 6.2.

3.2.4 Site Reconnaissance

Site reconnaissance is required to confirm, correct or extend the geological conditions predicted by the desk study and API. It also allows an assessment of site accessibility for any ground investigation and enables the identification of utilities or cultural artefacts which could affect or be affected by the project, as well as an examination of existing features that may indicate problematic ground, e.g. cracked or displaced surface drains.

Depending on the scale of the project, an overview from suitable vantage points may be useful. Photographs, including oblique stereo pairs, can be taken to illustrate the site conditions for later field mapping, and for further analysis.

The reconnaissance can also include:

- inspection of outcrops for lithological variations, major joint sets and structural features,
- checking of groundwater seepage and surface drainage condition, and
- observations of the locations of unstable ground and relevant geomorphological features not evident from API.

3.2.5 Synthesis of the Initial Geological Model

For large studies, the amount and variety of data which might be collected during the desk study can present logistical problems for presentation and synthesis into the geological model. While transparent overlays are still useful for quick reviews and small projects, the advent of geographic information systems (GIS) software now greatly facilitates synthesis and interpretation of large amounts of data of diverse origin and subject matter in order to build up an understanding of the site.

For presentation purposes, the factual information and interpretations can be displayed on a series of thematic maps at the same scale using a common topographical base plan. Interpretative maps and sections, which combine the most relevant data, may need to be produced. Depending on project requirements, they may include:

- geomorphological maps (e.g. GEO, 2004g),
- hydrogeological maps,
- geo-hazard and uncertainty maps, and
- engineering geology maps.

Examples of the synthesis of geological models for different geological settings and engineering applications are contained in Chapters 4, 5 and 6. Other examples which illustrate the synthesis of data from diverse sources to assist in developing the initial geological model are outlined below.

Figure 3.2.3 shows a regolith map and the trace of a major photogeological lineament overlain on an oblique aerial photograph. The lineament coincides with a break in slope on the topographical map, a contact between granite and andesite marked on the geological map, a seepage line and change in vegetation type and an area of high landslide density noted from API and site reconnaissance (MFJV, 2003a). The example demonstrates that each observation is reinforced by the synthesis of the data as a whole which allows the relevant geomorphological and hydrogeological processes to be better understood, including their efect on regolith development and potential landslide initiation. In this example, the landslide density was found to be much higher near the photogeological lineament which separates the steeper granitic terrain from the gentler volcanic terrain, the latter being mostly covered by

Bg - Granite outcrop	Bvs - Volcaniclastic sediments outcrop	Bcg - Conglomerate outcrop
Sg - Granitic saprolite	Sv - Volcanic saprolite	Clb - Relict landslide colluvium
Cv - Valley colluvium	Cr - Residual colluvium	Crf - Rock fall debris

Figure 3.2.3 – Regolith units and photogeological lineament overlain on an oblique aerial photograph (MFJV, 2002)

various colluvial deposits resulting from landslides.

The curvature of photogeological lineaments crossing areas of high relief on either side of ridges or valleys may give a good indication of dip and dip direction of the structure. These can be established by drawing strike lines between the intersection of the photolineament and the ground surface (Figure 3.2.4). The dip and dip direction of other geological boundaries which give rise to similar topographic expressions can also be approximated by applying the same principles.

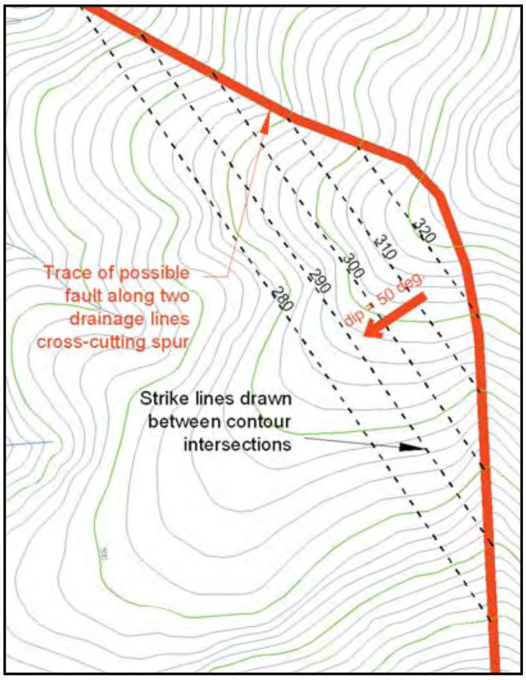


Figure 3.2.4 – Construction of strike lines on photolineament to determine dip of fault or geological boundary

The example shown in Figure 3.2.5 depicts rockhead based on archival information and an appreciation of the structural geology derived from the published geological maps and detailed API. While it is reasonably representative at the data points, it is highly interpretative in areas where data is lacking, and the actual rockhead surface is probably much more complex than is depicted. However, a high level of detail is not required for a desk study, but a realistic initial geological model with due allowance for variability is important for planning cost-efective investigation and design strategies.

3.3 ENGINEERING GEOLOGICAL MAPPING

3.3.1 Introduction

This section outlines the main engineering geological considerations that facilitate further development of geological models based on field mapping. This includes the definition of zones and boundaries that are likely to have different engineering and hydrogeological properties pertinent to development

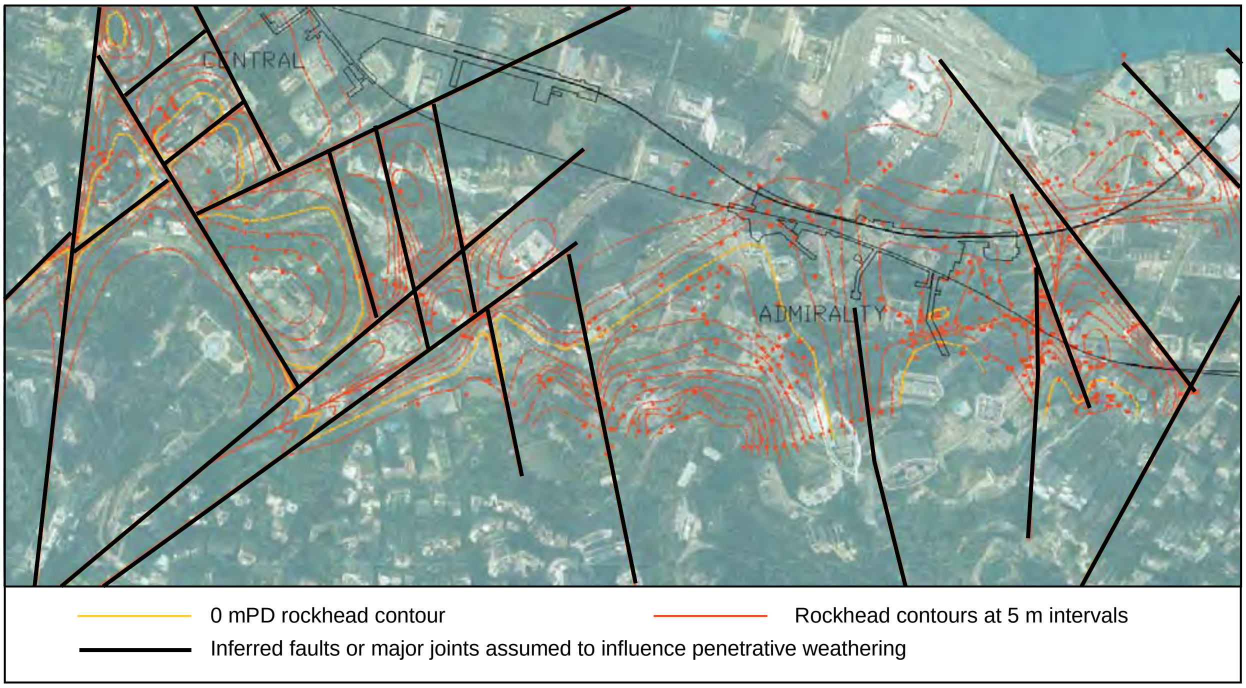


Figure 3.2.5 – Rockhead contours assuming fault and major joint control of penetrative weathering

of the ground model for the purposes of planning ground investigations. Sub-surface exploration and the geotechnical characterisation of rock and soil masses are outlined in Sections 3.4 and 3.5.

3.3.2 Approach

Field mapping is normally required to supplement the geological model developed during the desk study (Section 3.2). The field observations can assist in the identification of features with potential engineering significance which could have an impact on the design process.

The type of mapping undertaken will vary depending upon the scale and purpose of the project. Examples of a variety of mapping approaches relevant to engineering practice can be found in Griffiths (2001, 2002), Dearman (1991) and Smith & Ellison (1999). The mapping may need to extend beyond the area of direct concern in order that the geological and geomorphological setting is fully understood.

Field observations relevant to engineering projects in Hong Kong typically concern geological, geomorphological and ground performance factors and, as noted in Section 3.2.5, different maps may be required to record and synthesise the data. Examples of engineering geological mapping for area studies are contained in GCO (1987a,c,d), GCO (1988b,c,d,e,f,g,h), GCO (1991), Franks & Woods (1993), Campbell & Koor (1998) and Franks *et al.* (1999).

In some cases, mapping of specific geological features may be required where these may have a significant engineering implication. An example is the mapping of eutaxitic fabric within tuff following the Shum Wan Road landslide (Campbell & Koor, 1998), where the presence of a fault zone and associated deep weathering was indicated by changes in the fabric. Other examples of the mapping of locally significant features for different engineering applications are contained in Chapter 6.

3.3.3 Fieldwork

Preliminary maps from the desk study can be used as field sheets for recording additional observations. Combined topographical maps and ortho-photographs are useful to aid positioning. However, locating specific features on a broad, vegetated catchment can be difficult and it may be necessary to place surveyed markers across the study area (Pinches & Smallwood, 2000) or to use a Global Positioning System receiver if the vegetation cover allows. The use of a hip chain and field inclinometer can facilitate the production of representative longitudinal sections.

Field mapping of superficial deposits using a geomorphological approach is addressed in Sections 4.5 and 6.2.

Where relevant to the engineering project, the field mapping should include examination of rock mass characteristics where outcrops are accessible. The rock type, material weathering grade, joint data, other significant geological features which may affect the stability of the rock mass (e.g. bedding, fabric, clay infills, weak zones, faults etc), and seepage locations and flow rates should be noted. Attention should be paid to ensuring focus on persistent discontinuities and major weak zones.

The geological structure and the degree of significance of certain discontinuities may vary across the site. Delineation of structural domains (Sections 4.2 and 6.4.4) based on an understanding of the regional geology and mapping of the local geological structure is needed to identify and to differentiate between zones that are likely to have different engineering implications. GEO (2004c,i) provide guidance on the recognition and mapping of significant discontinuities which may affect slope stability. The Ting Kau example in Section 6.4.4 discusses where the presence of an adverse joint set was only realised during construction. This resulted in considerable amendments to the original design.

Depending on the project type, material and mass descriptions of rock exposures may need to be of sufficient detail for further characterisation using rock mass classifications and discontinuity shear strength models (Section 3.5). If necessary, project-specific discontinuity logging sheets may be developed from the examples shown in GCO (1988a). Potentially adverse weak zones and infills to discontinuities should be highlighted.

Where saprolite is exposed, the material and relict discontinuities should be described to the same level of detail as for a rock outcrop with specific attention being paid to soil pipes, differences in discontinuity condition between the saprolite and the parent rock mass, any kaolin concentrations, or displacement of the geological structure.

Soil and rock descriptions are discussed extensively in Geoguide 3 (GCO 1988a). However, it should be noted that:

- It is aimed *"primarily at the practising civil or geotechnical engineer"* and was "*prepared on the assumption that the user may not have any specialist knowledge*".
- It is *"recommended good practice"*, i.e. it is the minimum level of description expected and whilst the level of description recommended may be suitable for say simple foundations, it may not be adequate for a landslide study.
- Alternative descriptive systems are acknowledged and encouraged. The key principle is to clearly define all descriptive terms which are used to better characterise the ground. It further emphasises that *"the scope of the description, and the degree of emphasis given to particular descriptive items, may need to be varied to suit the particular application"* (e.g. projects involving slopes, tunnels, foundations, etc.).

Geoguide 3 is based on BS 5930 (1981), but notes that the BS fixed the boundary between fine and coarse soils at 35% implicitly requiring a laboratory particle size distribution test to be performed. BS5930 (1999) addresses this issue, placing emphasis on engineering behaviour and giving more flexibility with respect to the classification of fine and coarse soils based on particle size.

3.3.4 Presentation

The initial geological models, maps and reports described in Section 3.2 are updated using information from field mapping and any preliminary investigations. The updated models are used to progressively target further mapping efforts and to refine understanding of the geological and geomorphological processes that have formed the study area. The API should be re-evaluated on the basis of the fieldwork.

Composite maps synthesised from key findings of different mapping objectives can be useful in

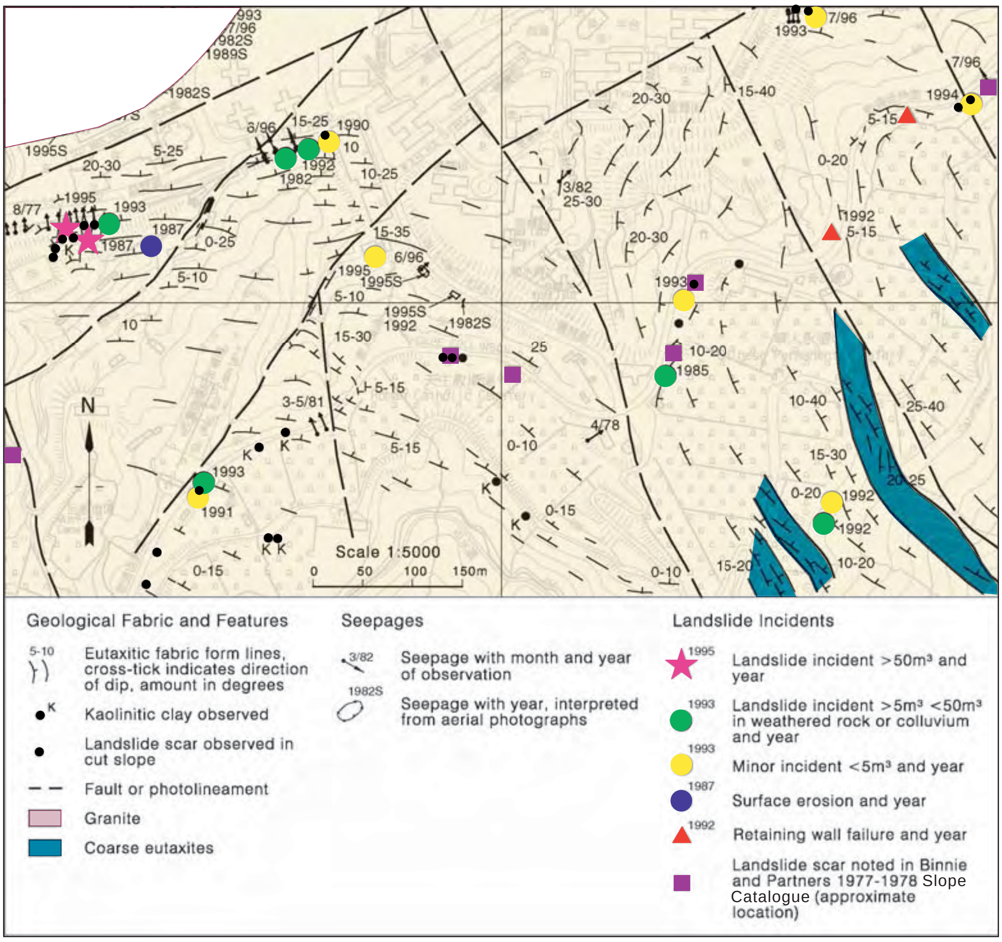


Figure 3.3.1 – Composite extract from maps of the Chai Wan Engineering Geology Area Study (Martin, 2003 after Campbell et al., 1998)

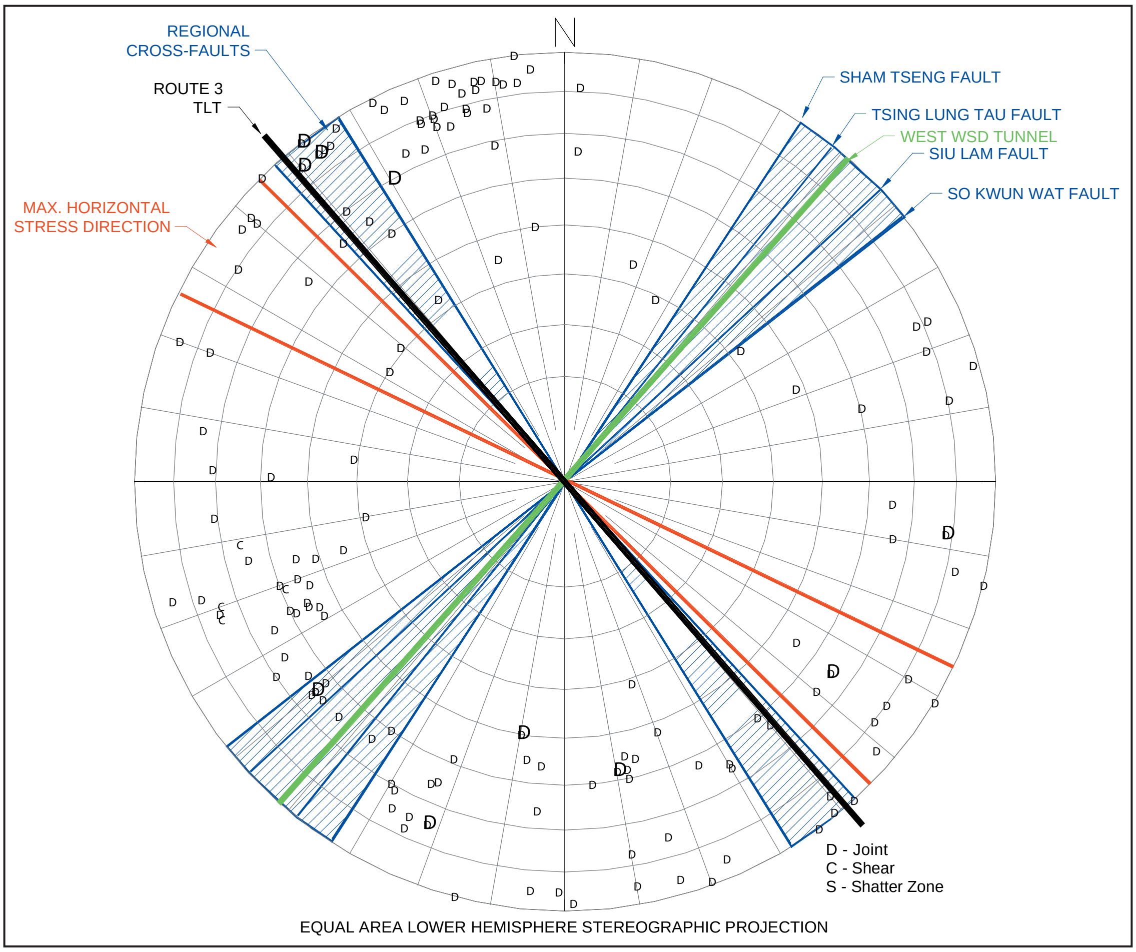


Figure 3.3.2 – Example of a structural domain summary plot for the Route 3 Tai Lam Tunnel

building-up an overall appreciation of the geological constraints present within a site. Figure 3.3.1 shows a composite map synthesised from separate maps of geological fabric and features, seepages and landslide incidents.

Figure 3.3.2 shows an example of a summary stereoplot for a structural domain in close proximity to the south portal of the Route 3 Tai Lam Tunnel which, in addition to providing discontinuity information, includes the main regional fault trends and *in situ* stress direction in relation to the tunnel alignment. Such maps and orientation diagrams are useful for displaying the potential structural geological controls on the stability of a site to the design team.

For projects that are potentially vulnerable to geological hazards, it may be beneficial to generate geological uncertainty schedules and maps. These can be audited, traced and updated as the site investigation and creation of the ground model progresses. Appropriate descriptions of key engineering geological issues such as existing instabilities, adverse geological structure, kaolin-infilled discontinuities, irregular rockhead profile and perched or high groundwater tables should be made.

Different geological models depicting plausible ranges of ground conditions can be developed to plan the ground investigation and aid communication among members of the design team.

3.4 SUB-SURFACE EXPLORATION

3.4.1 Introduction

Depending on the level of geological and geotechnical uncertainty, and the type and scale of the proposed works, ground investigations are usually required to further refine the geological and/or ground models for design purposes. Where time, access and environmental constraints allow, a staged approach to ground investigations is normally the most effective, with the initial investigation primarily focused on testing the geological model and the resolution of geological and hydrogeological uncertainty.

This allows the sub-surface materials, geological structures and hydrogeological regimes to be better defined in three dimensions which facilitates the planning of more detailed investigations primarily aimed at determining engineering parameters for ground and groundwater modelling and excavatability assessments.

3.4.2 Existing Guidance

Key guidance on site investigations is contained in GCO (1987b). Other relevant documents include:

- AGS-HK (2004a,b,c,d,e; 2005a,b) Ground Investigation Guidelines.
- Geospec 3 (GEO, 2001).
- BS5930 (BSI, 1999) gives international guidance.
- International Society for Rock Mechanics and American Society for Testing and Materials standards for rock testing.
- Ground Investigation Working Party Final Report, (IMMM-HK, 2003).
- GEO (2005a) for site investigations for tunnel works.

Sections 6.8, 6.9 and 6.10 of this document refer to site investigation for reclamations, contaminated land and landfills, and natural resources respectively.

3.4.3 Ground Investigation

General

The ground investigation needs to verify the geological and hydrogeological conditions, address areas of uncertainty and identify features which are of particular relevance to the stability or performance of the proposed works and its surroundings (see Figure 4.4.9 for an example). The types and methods of investigation will depend on the anticipated geology, local constraints, environmental considerations and the nature of the proposed works.

Geophysical Methods

Where the terrain and site conditions are suitable, geophysical surveys can be cost-effective in covering large areas in a relatively short time, particularly for offshore locations (see Sections 6.8 and 6.10) and also for onshore terrain which may be blanketed by thick regolith with few outcrops. However, problems can occur due to interference that makes it dificult to differentiate true signals from noise.

Typical geophysical methods for engineering geological application include gravity, magnetic, seismic reflection, seismic refraction and resistivity surveys. A large amount of geophysical data for the offshore areas of Hong Kong is held by the Hong Kong Geological Survey (HKGS) of GEO. Fyfe *et al.* (2000) show examples of the use and interpretation of seismic surveys for offshore areas in Hong Kong. Evans *et al.* (1995) provide guidance on the interpretation of seismic reflection surveys. The geological model should form the basis for the type and location of the surveys undertaken. For example, survey lines orientated perpendicular to known features such as infilled channels allow better resolution.

Examples of the use of gravity surveys to identify areas of deep weathering associated with karst areas and major faulting can be found in Collar *et al.* (1990) and Kirk *et al.* (2000) respectively. Collar *et al.* (2000) discuss the adaptation and limitations of gravity surveys to steeply sloping terrain, and indicate that where appropriate environmental adjustments are made, the method could prove to be useful in formulating preliminary rockhead models to aid the planning of further investigations (Figure 3.4.1).

GEO (2004h) contains reviews of the use of down-hole geophysical methods such as gamma density and spectral gamma to detect clay-rich seams in saprolite and rock.

As with any indirect method of investigation, considerable knowledge and skill are required in order to effectively specify and plan the investigation and interpret the results. Direct investigations are also necessary in order to calibrate and verify the model developed from geophysical data.

Direct Methods

Direct investigations may include drillholes, trial pits, trenches and slope stripping. GCO (1987b) provides guidance of a general nature on site investigation and sampling quality class. However, more detailed investigation may be necessary where the findings of the initial geological model indicate the possible presence of important geotechnical conditions (e.g. clusters of previous failures, heavy seepage, voids, soil pipes, deep weathering, etc.).

In order to maximize the information obtained, the ground investigation should be planned around the relevant aspects of the initial geological model

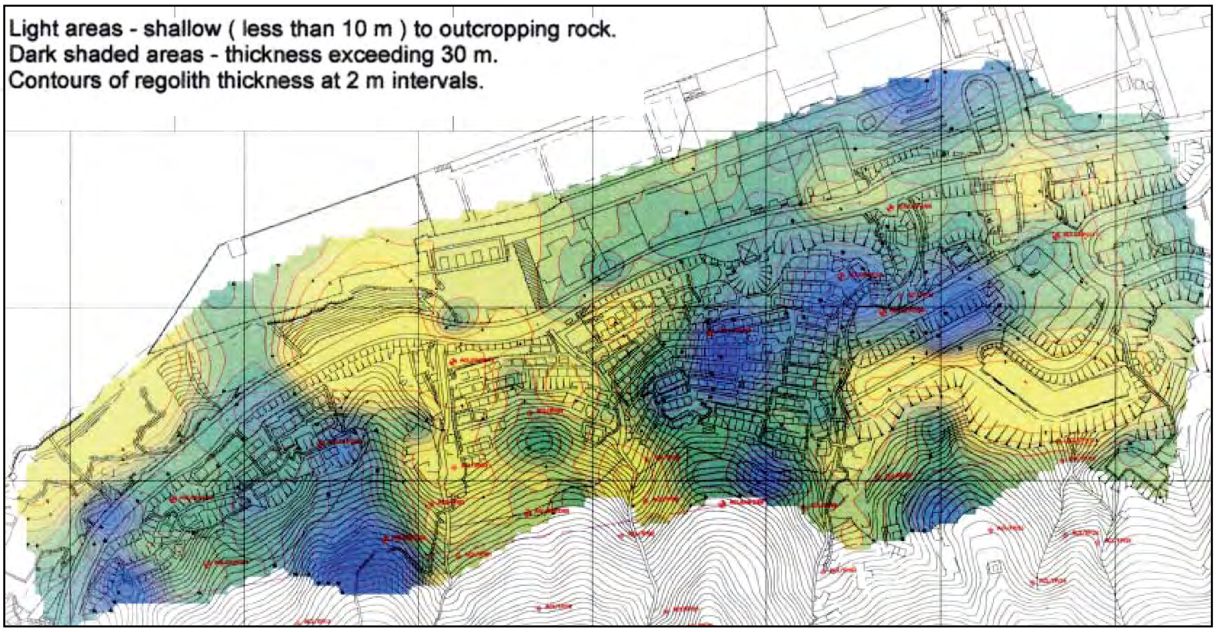


Figure 3.4.1 – Example of regolith thickness in steeply-sloping terrain inferred from 3D gravity model (Collar et al., 2000)

formulated from the desk study and detailed engineering geological mapping. In the initial stages, the investigation is best focused on the resolution of any geological uncertainties which are significant to the design. Detailed inspection and logging of continuously sampled drillholes can help to achieve this aim. Further investigations can then be focused efficiently to obtain the relevant hydrogeological and geotechnical properties of the different ground units identified in the initial investigations.

The investigation should utilise the most relevant investigation techniques. For example, Parry *et al.* (2004a) report on a slope which was noted as potentially problematic in the previous engineering geological area study (Franks *et al.*, 1999) due to past instabilities, day-lighting joints and groundwater seepage, and therefore the possibility of adversely orientated kaolin-infilled joints being present. However, the drillholes were designed to investigate rock and not the overlying layer of saprolite. These drillholes therefore gave no indication of the clay-infilled relict discontinuity subsequently observed in the slope face. Inspection of the slope revealed a persistent (>5m) low angle relict discontinuity infilled with up to 30 mm of slickensided buff grey clay slightly above the soil to rock interface which had not been detected by the drillhole investigation.

Depending on the engineering application and site conditions, trial pits/trenches and slope stripping can be more cost-effective and efficient than drillholes in determining the nature of superficial deposits and fill, as well as relict structure at shallow depth. They also allow for the examination of signs of previous movement such as joint infills, tension cracks and deformation structures.

Continuous triple tube mazier sampling in soil or triple tube coring in soil and rock can be used with air foam or bentonite/polymer mud flushing to maximise recovery of weak layers at depth. A case study of the recovery of cavity infill deposits in marble blocks at depths in excess of 100 m using careful drilling techniques is included in Section 6.5.6.

Where stability-critical geological structures are suspected, the drillhole orientation can be optimised to intersect these features. The recovery of weak layers in rock can be particularly difficult, due to the high contrast in drilling resistance between the rock and the weak layer, especially when the weak layer is not orientated normal to the direction of drilling.This can lead to erosion of the weak material by the drilling medium while the drill bit is still partially coring the rock. Sub-horizontal holes provide the best chance of recognition and recovery of sub-vertical features, while holes inclined into the slope at 50° to 60° from horizontal provide the best chance of recovery of features dipping at 30° to 40° out of the slope.

The representativeness of a drillhole log is dependent on the continuity and quality of the samples available for logging. The standard ground investigation sampling of alternating mazier samples and standard penetration test (SPT) liner samples, will result in only 5% of the ground investigated being available for inspection from the cutting shoes. In order to achieve a more continuous record, any mazier samples and SPT liner samples that will not be used for testing should be opened, logged in detail and photographed. Particular attention should be paid to discontinuities and weak layers (such as shear zones and more weathered zones, see the South Bay Close example in Section 6.4.4).

Drillholes and trial pits in Hong Kong are typically logged by personnel who may not have been informed of the purpose of the investigation and the details of the geological model. Figure 3.4.2 compares a trial pit log prepared without the benefit of a geolgical model (Log ‘a’) with that produced in the context of the geological model and purpose of the investigation (Log ‘b’). This demonstrates the need for good communication between the designer and the ground investigation contractor.

The orientation of discontinuities in rock drillholes can be assessed using impression packers, televiewers, mechanical core orientation methods, etc. The relative orientations of the drillholes and the discontinuity sets must be known in order to assess the true spacing of each set. If outcrops are available, mapping of discontinuities should be used to provide a better understanding of the geological structure and to provide a check on possible errors in core orientation. Figure 3.4.3(a) shows a plot of joint orientations as initially recorded on the logs for an inclined drillhole. Discontinuity data from mapping of outcrops and from other orientated drillholes indicated a consistent structural pattern in the area which was markedly different from the drillhole data. It was realised that the reference line on the drillhole had been erroneously rotated by 180°. Figure 3.4.3(b) shows the corrected plot for comparison.

A major limitation of measurement of discontinuities in drillholes is that no indication can be obtained of the relative persistence of discontinuity sets. Figure 3.4.4(a) shows a stereoplot of mapping data from a quarry face where discontinuities

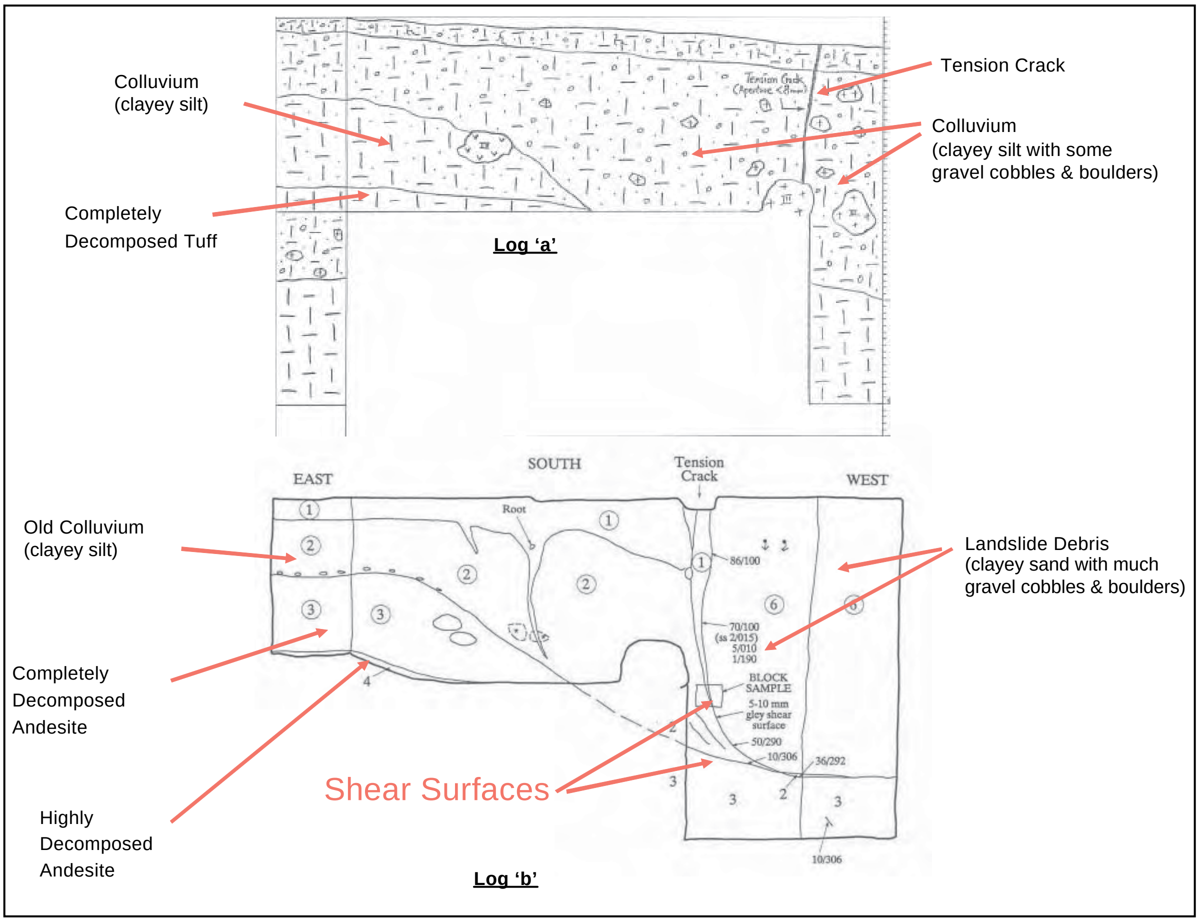


Figure 3.4.2 – Comparison of two logs for the same trial pit (after Parry & Campbell, 2003)

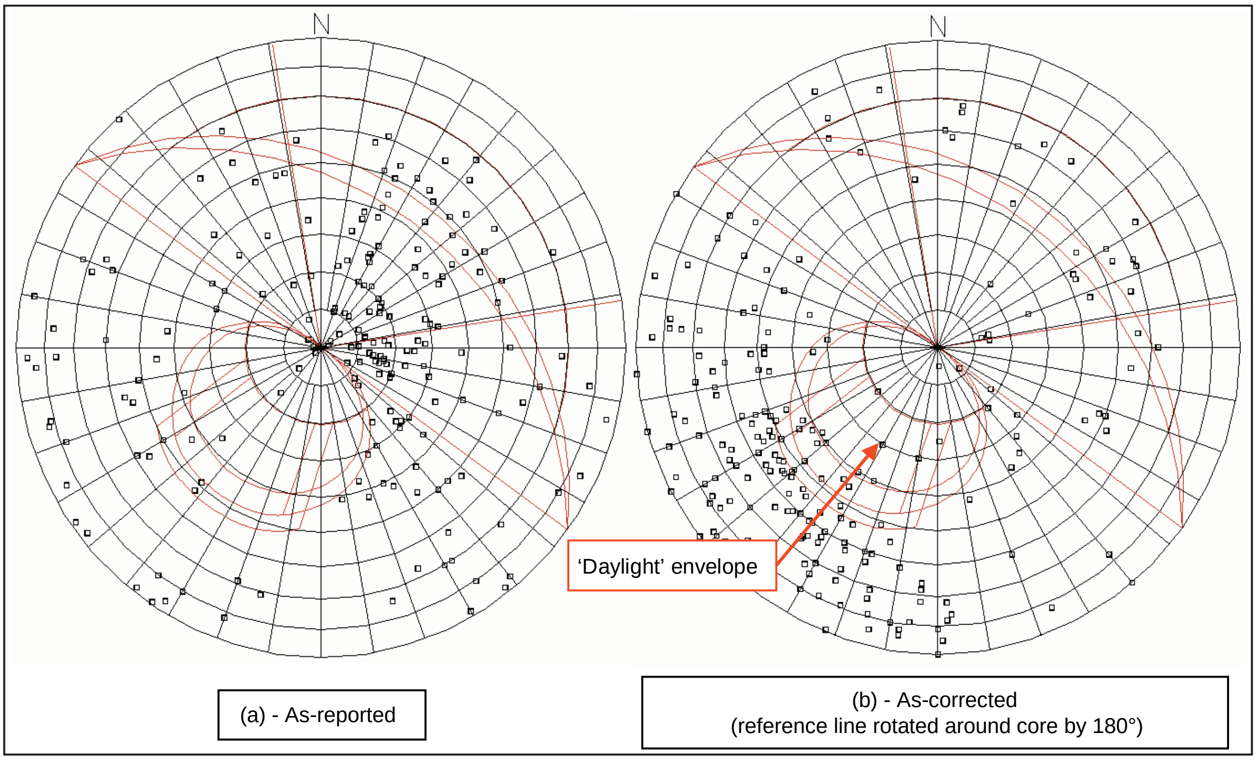


Figure 3.4.3 – Comparison of as-reported and as-corrected stereoplots of joint data for the same inclined drillhole core

with a persistence of less than 3 m were ignored. Figure 3.4.4(b) shows discontinuity data obtained from orientated acoustic televiewer logging of drillholes in the same quarry . The apparent difference in the relative orientation and the significance of 'Set 5' in each plot is primarily due to the impersistence of that particular set.

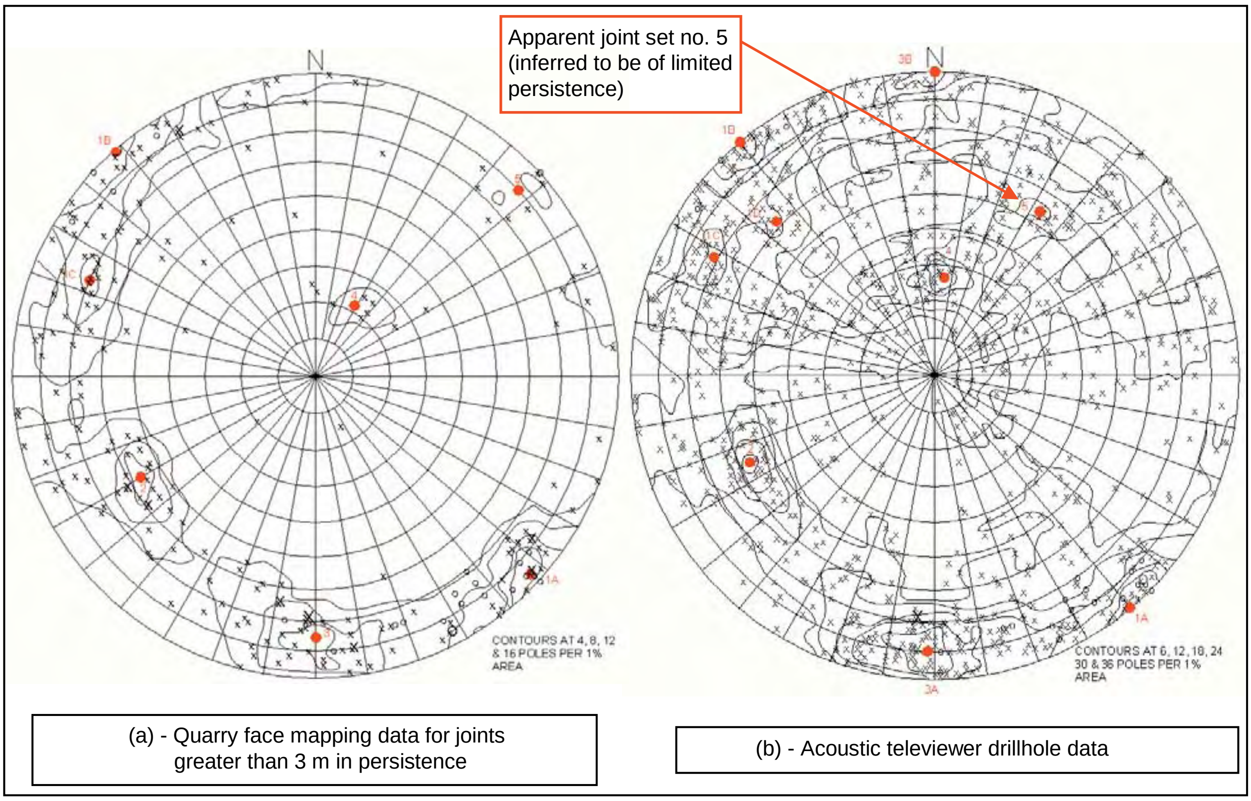


Figure 3.4.4 – Comparison of mapping data and acoustic televiewer data for the same quarry

In situ Testing

The applicability of *in situ* tests depends on the material type or mass conditions and the questions that need to be answered by the ground investigation which in turn are dependent upon the type of engineering application. Typical *in situ* testing methods for the measurement of strength, deformability and permeability are described in GCO (1987b).

The purpose of testing *in situ* is to characterise the mass properties of specific geological materials that are in the ground. When planning *in situ* testing and assessing the applicability of the results to the ground model, the influence of the proximity of discontinuities or discrete zones with markedly different properties to the rest of the mass should be considered. In general, the results are likely to be more representative where the test area and the volume subjected to the test are large, e.g. pile load tests, large-scale deformation tests and trial embankments. The certainty with which the results can be applied across a site depends upon the accuracy and representativeness of the geological model.

One of the most common *in situ* tests carried out in Hong Kong soils is the Standard Penetration Test (SPT), which can be used as one of the guides to predict trends in the shear strength of saprolite (Pun & Ho, 1996). Chan (2003a) contains observations on testing *in situ* and back analysis of ground movements for a number of deep basement excavations in Hong Kong and in the Asian region, and suggests relationships of Young's Modulus (E) value versus SPT N-value for various soils including Grade V granite, Grade IV-V granite, fill and marine deposits. However, it should be noted that different relationships between E and SPT values have been used in Hong Kong, and that project-specific design relationships will need to be confirmed by monitoring and back analysis during construction.

Geophysical methods of testing *in situ* can be useful for assessing rock mass characteristics such as Q-value and deformability, when knowledge of the porosity and unconfined compressive strength of the rock material is known (Barton, 2000). Although considerable care and judgement are required to convert the results to engineering parameters, the local influence of discontinuities on the test results is likely to be reduced compared to small-scale *in situ* tests in drillholes. Measurements of seismic velocity can be used to assess the susceptibility of unconsolidated ground to liquefaction and magnitude of ground motions during earthquakes (GEO, 1997; Pappin *et al.*, 2004). Measurements of cross-hole seismic velocity measurements were carried out for the Route 10 Tsing Lung Tau suspension bridge (Bachy Soletanche, 2001) to assess rock deformation properties and seismic response spectra for detailed design. An example plot is shown in Figure 3.4.5 which indicates an apparent increase in rock mass quality with depth.

Care should be taken when conducting tests in rock with a relatively small test area because of the

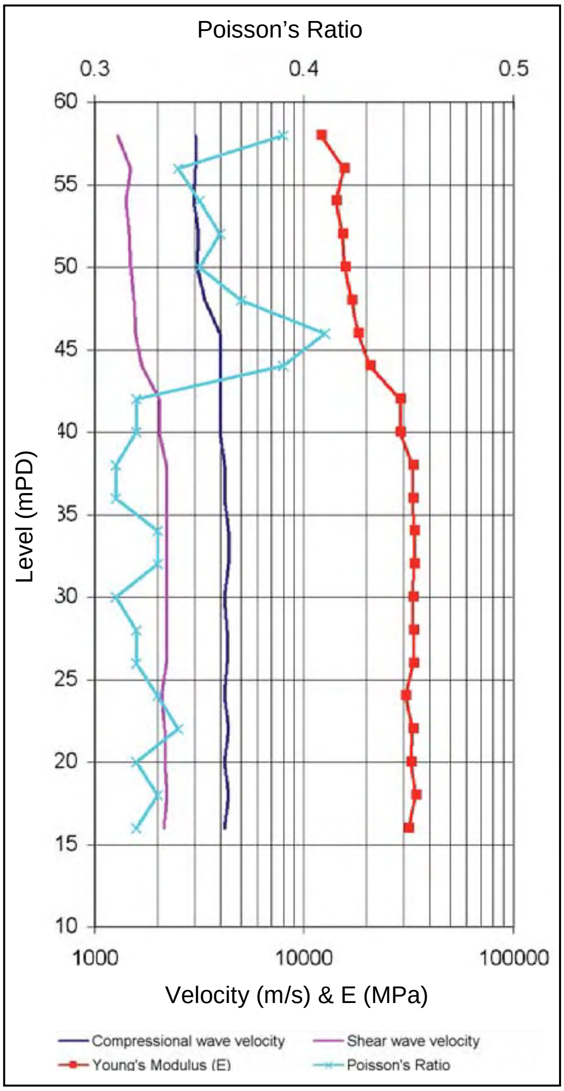


Figure 3.4.5 – Example of cross-hole seismic testing showing increase in apparent rock mass quality from 0 m to 15 m depth (after Bachy Soletanche, 2001)

diverse results depending on the local influence of discontinuities. For instance, whereas pump tests from wells can determine the mass hydraulic properties of a rock aquifer, packer tests over short lengths of drillholes can yield considerably variable results. For similar reasons, the hydraulic fracturing method of *in situ* stress measurement in rock generally gives less scatter than the over-coring method which is more sensitive to the presence of discontinuities near the strain gauges (Free *et al.*, 2000).

3.4.4 Hydrogeological Investigation

The potential variability of hydrogeological regimes, as outlined in Sections 4.6, 6.4 and 6.7 needs to be incorporated into the geological model to enable the planning of an effective hydrogeological investigation. Interpretation of groundwater regimes is generally based on regular monitoring, preferably throughout at least one wet season, for land-based projects. Groundwater monitoring may need to be continued throughout the construction and post-construction phases to gauge the possible effects due to the works.

The location of the response lengths and the number of piezometers should be optimised with reference to the geological model to ensure that all hydrogeologically significant zones and boundaries are adequately monitored. During drilling of holes for piezometers, detailed logging on site provides more information that can be used to refine the geological model and to optimise the locations and the response lengths. Typical locations for piezometers and tests, which may need to be considered, include:

- Upslope and downslope of steeply inclined geological features which may act as aquitards.
- Within discontinuities (potential high cleft-water pressure during rainstorms).
- Within permeable zones in the weathering profile, which may include Grade III/IV materials (perched or confined groundwater).
- Directly above low-angle clay-infilled discontinuities (potential perching).
- Above the rock to soil interface (potential perching).
- Below the rock to soil interface (potential confined groundwater).
- At soil pipes or zones showing evidence of groundwater flow (potential high response during rainstorms).
- Above the saprolite/superficial deposit interface (potential perching).
- Within any coarse layers in superficial deposits or fill (potential perching and/or potentially high response during rainstorms).

Given the above, the use of standpipes with long response zones which intersect a number of the features noted above makes interpretation of the data problematic.

Where significant seepage is noted, groundwater tracer tests may be conducted by introducing dyed or chemically identified water into the piezometer and monitoring the seepage points to detect resurgence (GCO, 1982; Nash & Chang, 1987).

The optimisation of hydrogeological investigations depends to a great extent on the progressive development of the geological model during the investigation to enable variations to be made with regard to depth and number of piezometers, the number, location and orientation of drillholes and also the frequency and period of monitoring.

For slope stability assessments, monitoring of groundwater levels using electronic transducers and data loggers provides information on rapid groundwater responses to individual rainstorms. In some less critical situations the installation of 'Halcrow buckets' in piezometer tubing is a cost-effective alternative which can indicate the highest water level reached between observations with an accuracy equal to the spacing of the buckets.

Where access is difficult, piezometers may be located in trial pits, although care must be taken to minimise the effects of disturbance (MFJV, 2004), or in holes bored with a lightweight portable drilling rig (Chau & Tam, 2003). Examples of detailed hydrogeological studies using a variety of techniques are documented in GCO (1982), Weeks & Starzewski (1985), Evans & Lam (2003a,b), and MFJV (2004).

For detailed studies, continuous monitoring of site-specific rain gauges, surface water weirs and groundwater drain flows (Section 4.6) may be required. This allows groundwater responses, and hence the effectiveness of drainage works, to be assessed in relation to specific rainstorms and antecedent rainfall patterns. In these cases, it may be beneficial to carry out real-time monitoring with all instrumentation having the capability of being remotely sampled and transmitted to the office for processing. A review of slope instrumentation practice in Hong Kong and its development trends is given in Wong *et al.* (2006).

3.4.5 Storage and Handling of Data

For large projects, it is common to use geotechnical database management software to store and analyse data from ground investigations with data coded in standard AGS format or variants thereof (AGS, 1999). An example is the data management system used for the Chek Lap Kok Airport (Plant *et al.,* 1998).

However, it is important that all relevant geological materials and features are properly identified, characterised and coded in such a way that the data can be easily verified, retrieved and manipulated to facilitate further development of the geological model. The parallel maintenance of hand-drawn models which depict the geology and hydrogeology in 3-dimensions can provide the necessary verification of any computer generated diagrams and can often be more cost-effective in providing an overall understanding of the site.

3.4.6 Updating the Geological Model

After each stage of ground investigation work, the data needs to be critically reviewed and incorporated into the geological model for the project. This enables refinement of the model to be carried out and also provides a check for ground investigation data that may be inconsistent with the overall model. Such anomalies may be due to inconsistent logging or interpretation of materials. Alternatively, the anomalies may indicate that the geological model needs to be adjusted or that areas of uncertainty exist which may need further investigation if they are judged to be sufficiently critical to the proposed works. These may need to be noted for auditing and tracing by the design team throughout the rest of the investigation and design process.

3.5 GEOTECHNICAL CHARACTERISATION

3.5.1 Introduction

This section outlines the main engineering geological considerations for the evaluation and assessment of data to select appropriate parameters or range of parameters for development of the ground model.

The data from the ground investigations will be used to determine possible ranges of material properties and mass properties, including environmental factors such as the groundwater regime, contamination and *in situ* stress conditions. Estimates of the possible ground and groundwater responses to changes in environmental conditions during construction and over the design life of the proposed works may also be necessary in order to define the spatial extent of any further ground investigation and monitoring requirements.

The main engineering geological inputs to the development of the ground model include:

- identification of any stability-critical discontinuities, zones of weakness or permeability contrasts that may need to be considered in the ground and design models,
- selection of samples for testing with reference to the engineering geological zoning of the site,
- assessment of empirically derived geotechnical parameters based on detailed observations of rock mass characteristics,
- assessment of the spatial variability of the ground based on correlation of the test data with the geological model to minimise inappropriate spatial averaging of data from different materials and masses, and
- revision of the geological and ground models as more data is received.

The methods of ground investigation, testing and instrumentation which can be used to facilitate geotechnical characterisation of the ground (see Section 3.4) will depend on the geological model, the proposed works and the intended design methodologies. As such, the degree of engineering geological input will vary. For instance, projects that involve large excavations in rock or mixed rock and soil profiles need a relatively high level of engineering geological input because the engineering performance of the works is likely to be controlled by discontinuities, variations in hydrogeology and variations in weathering that affect mass stability. By comparison, for small-scale excavations, after it has been determined that the geological and hydrogeological variability of the site is relatively minor, less engineering geological input is needed.

Engineering geological issues that affect the material and mass properties of the main rocks and soils of Hong Kong are further discussed in Chapters 4 and 5.

3.5.2 Material Properties

Material properties can be obtained by conducting tests in a laboratory on small samples. GCO (1987b) and GEO (2001) contain detailed descriptions and specifications of types of laboratory tests for soils, rocks and groundwater.

The representativeness of test results can be affected by variations in lithology, material type and fabric, bias in selection of samples, size of sample, presence of discontinuities and test procedures. In addition, weaker rocks and soils can be affected by sample disturbance and the scale effects of large inclusions. The representativeness of laboratory test data with respect to preparation of samples and the appearance of samples after testing should be critically reviewed before the data is incorporated into the ground model. Even the presence of microscopic flaws in apparently intact samples can reduce test values with increasing size of sample (Figure 3.5.1).

In view of the typically small volume of material tested relative to the volume and variability of mass affected by the proposed engineering works, it is good practice to assess the results and their applicability based on an engineering geological knowledge of the materials and their spatial distribution within the site. Correct identification of the material being tested also facilitates the grouping of test results to derive realistic ranges of engineering properties for application within the ground model.

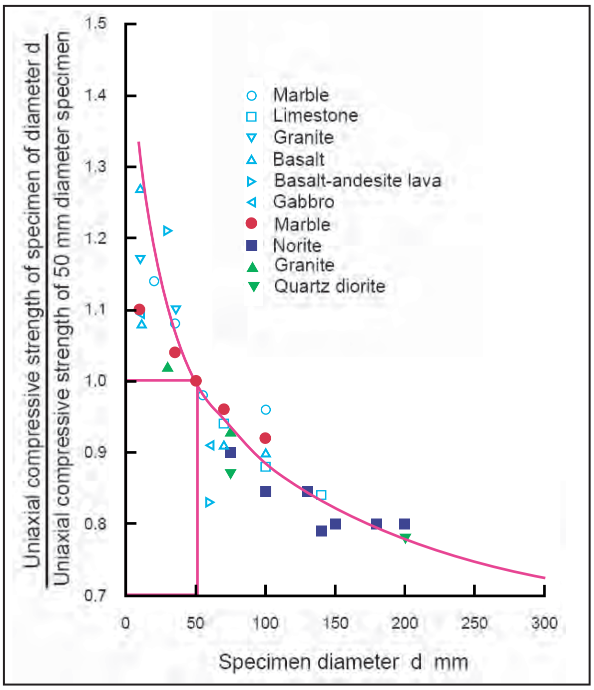


Figure 3.5.1 – Influence of specimen size on strength of 'intact' rock (Hoek, 2000)

Testing *in situ* is outlined in Section 3.4.3. Determination of material properties from *in situ* tests requires interpretation. Detailed knowledge of the geology and weathering profile, and trends in and established relationships between shear strength, decomposition, micro-fracturing, grading and moisture content, together with *in situ* test values can be used to determine realistic ranges of material properties for the different zones within a site (Martin, 2003).

3.5.3 Properties of Discontinuities

Laboratory Testing

Tests on small samples in a laboratory require interpretation. For example, when testing discontinuities in rock under direct shear, small scale asperities can greatly affect the measured angle of friction (Richards & Cowland, 1982). As the shear strength envelope of naturally rough discontinuities is strongly curved, it is also important to ensure that the range of normal stress during testing is of a similar order to the range of normal stress expected under field conditions. Furthermore, direct shear tests can be affected by misalignment of the shear box with the plane being tested (Campbell & Parry, 2002). Kulhawy & Goodman (1980) and ISRM (1981) provide information on the laboratory measurement of the shear stiffness of rock joints.

Field-scale Estimation of Discontinuity Shear Strength

The assessment of discontinuity shear strength at a scale relevant to the proposed works can be carried out using large-scale *in situ* shear tests. However, these are expensive and can be difficult to control or to accurately measure the forces during testing. For these reasons, recourse is often made to indirect methods of shear strength estimation, based on a basic friction angle, say as derived from laboratory tests. In addition, *in situ* factors such as surface roughness and irregularity, type and thickness of infill material and the strength of any asperities, which influence shearing resistance, need to be considered.

Guidelines for the description of surface roughness and irregularity are contained in GCO (1988a), and methods of assessing field-scale shear strength are summarised in Hoek (2000). This latter work draws on key references such as Barton & Choubey

(1977), Richards & Cowland (1982) and Hencher & Richards (1982). The reduction in the effect of small-scale asperities with increasing length of discontinuity is addressed by Barton & Bandis (1990) and Barton (1990). Where discontinuities contain infills, shearing may occur at the interface between the infill and the wall of the discontinuity. Such planes can exhibit much lower shear strength than that of intact infill material (Deere & Patton, 1971).

3.5.4 Mass Properties

General

The mass properties of weathered rocks are governed by the properties of the constituent materials and of the discontinuities therein. In some cases, the properties of the materials may dominate the overall behaviour of the ground mass. Coarse-grained igneous rocks with widely-spaced joints may give rise to corestones upon weathering (see Section 4.4.4). The effect of the presence of corestones on the mass properties of the soil has been studied (Irfan & Tang, 1993).

The properties of the discontinuities are more likely to dominate mass behaviour where the discontinuities have relatively low shear strength, are relatively closely spaced, relatively extensive and unfavourably orientated with respect to potential ground deformations or stability. Where a single discontinuity or a combination of discontinuities occurs which may cause direct sliding, the properties of the specific discontinuities are of far more concern than the mass properties.

Given the dominant role of discontinuities in affecting the geotechnical properties of weathered rock masses, this section primarily concentrates on the types of models and methods of classification that may be used to characterise the effect of discontinuities on the mass properties of the ground.

Owing to the uncertainties involved in assessing the properties of ground masses, it is good practice to derive a range of potential mass parameters that reflect these uncertainties for use in sensitivity analyses.

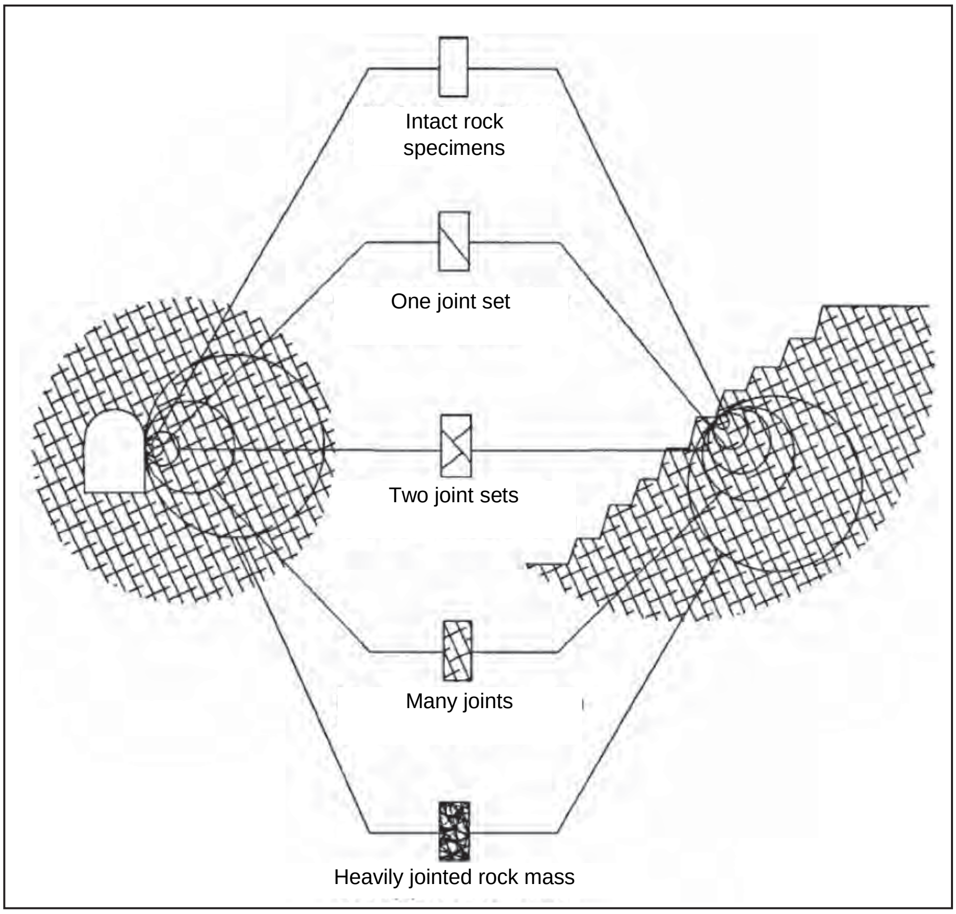


Figure 3.5.2 – Scale effect on the characteristics of discontinuous rock masses (Hoek, 2000)

Figure 3.5.3 – Candidate failure surface involving a number of different shear failure mechanisms (Hoek et al., 2000)

Effect of Discontinuities on the Mass Properties of Weathered Rocks

The mass shear strength and stiffness of weathered rocks are generally lower than the values indicated from laboratory tests on material samples, due to the presence of discontinuities (Figure 3.5.2). For example, Figure 3.5.3 illustrates some of the failure modes that may need to be considered in slope stability analysis. These may involve shear along a through-going major weakness, shear through the mass weakened by second order discontinuities and shear along stepped paths created by two or more discontinuity sets. The presence of steeply inclined, critically orientated discontinuities within the potential sliding mass may be an important factor that influences stability and the consequent brittleness and potential mobility of the mass at failure (Figure 3.5.4).

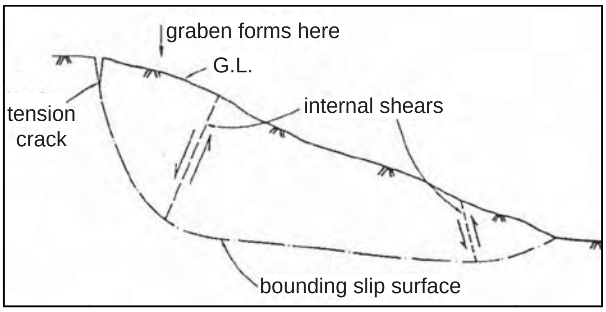


Figure 3.5.4 – Typical internal shears required to permit movement in a non-circular slide (Hutchinson, 1987)

The anisotropic effect of discontinuities on the mass shear strength of saprolite has been modelled using probabilistic methods (e.g. Koo, 1982), but these are difficult to apply in practice. Knowledge of past failures in similar ground conditions combined with a detailed knowledge of the engineering geology of the site can assist in determining the site-specific applicability of such models (Wong & Ho, 2000a).

An example of an alternative approach is given in Section 6.4.4 for the Ting Kau Cutting where a range of scenarios were incorporated into the ground model for sensitivity analysis. Different ranges of mass shear strength and discontinuity orientation, continuity, spacing, and location relative to the excavation were modelled based on the structural data from field investigations conducted for initial design, supplemented by detailed mapping of the excavation during construction.

GCO (1984), Hencher (1985) and GEO (2004c) contain guidance on the uses and limitations of stereographic projections and the influence of major discontinuities on the mass stability of rock slopes. Important considerations include:

- the recognition and presentation on stereoplots of single, through-going planes of weakness,
- the influence of discontinuities at a low angle with low strength (e.g. Fei Tsui Road – GEO, 1996a,b; Shek Kip Mei – FMSW 2000),
- the effect on mass stability of non-daylighting

discontinuities, and

- the role of groundwater in reducing the operative friction angle when considering the use of friction cones on stereonets to assist in the assessment of kinematic admissibility for failure.

Empirical Methods of Characterisation of Rock Masses

Empirical methods of rock mass classification are based largely on the spacing and condition of the discontinuities, and, in most cases, on the unconfined compressive strength (UCS) of the material. These classification systems do not give engineering parameters directly, and considerable experience and judgement are required in order to derive an appropriate range of parameters.

A listing of some rock mass classification systems that are commonly used in Hong Kong is given below Their main uses, input parameters and key references are also cited.

- *Rock Mass Rating (RMR) System* – (Bieniawski, 1989)
  - Uses – Rock mass classification system for design of support systems for underground excavation and for estimating rippability and dredgeability. Modified forms can also be used to provide estimates of rock mass deformability (GEO, 2006).
  - Main Input Parameters – UCS, Rock Quality Designation (RQD), discontinuity spacing, discontinuity condition, groundwater rating and discontinuity orientation rating.

- *Q System* – (Grimstad & Barton, 1993)
  - Uses – Rock mass classification system for design of underground excavation support systems. Also used for assessing TBM suitability. Modified forms can be used to provide estimates of rock mass deformability (Hoek *et al.*, 1995; Hoek, 2000, 2004; Barton, 2000).
  - Main Input Parameters – RQD, discontinuity set rating, discontinuity conditions, groundwater rating and stress reduction factor.

- *Geological Strength Index (GSI) and Hoek/Brown Strength Criterion* – (Hoek, 2000; Marinos & Hoek, 2000; Hoek, 2004; RocLab, 2004)
  - Uses – Rock mass classification system to derive estimates of deformability and Hoek/Brown non-linear strength parameters for the analysis of slopes, foundations and underground excavations.
  - Main Input Parameters for GSI – Rock type, rock structure/degree of interlocking of rock pieces, discontinuity condition.
  - Main Input Parameters for Deformation Modulus – GSI and UCS.
  - Main Input Parameters for Hoek/Brown Strength Criterion – rock type GSI, UCS and excavation disturbance factor.

- *IMS System* – (McFeat Smith, 1986)
  - Uses – Rock mass classification system for design of underground excavation support systems and for estimating performance of Tunnel Boring Machines (TBM).
  - Main Input Parameters – Weathering grade, spacing and orientation of discontinuities, and conditions of water inflow.

- *MQD Karst Classification* – (Chan, 1994; Chan & Pun, 1994)
  - Use – Classification of buried karst from drillhole data to facilitate identification of the extent of marble slightly affected and unaffected by dissolution. Used for zoning marble rock masses for estimating suitability for foundations in Scheduled Areas 2 and 4 in Hong Kong.
  - Main Input Parameters – RQD and percentage core recovery (see Section 5.5).

Considerable uncertainty in the resulting engineering parameters is likely to exist due to the indirect nature of classification systems. For this reason, a wide range of values may need to be considered for sensitivity analyses. An example of the translation of engineering geological data into a range of engineering parameters for sensitivity analysis using the Q-system and GSI-system is shown in the KCRC Tai Lam Tunnel example in Section 6.7.4. This example also illustrates the sensitivity of the possible range of ground deformations to the engineering geological characterisation of the rock mass.

3.5.5 Characterisation of Hydrogeological Properties

After completion of the main ground investigation works, a range of tested permeabilities will have been obtained, and actual or potential high permeability zones will have been identified. As monitoring continues, further piezometer readings and rainfall data will become available. In many applications further characterisation of hydrogeological properties can be achieved by monitoring the subsequent groundwater response and by back analysis. This allows further development of the geological model and refinement of the values assigned to the hydrogeological properties (see Section 4.6).

3.6 MODEL DEVELOPMENT DURING DESIGN AND CONSTRUCTION

3.6.1 Development During Design

In selecting the initial design methodology, judgement is required to assess the degree of adversity of the geotechnical conditions (Martin, 2003). This can be facilitated by consideration of the sensitivity of the proposed works to the range of possible conditions and potential mechanisms of deformation or shear failure indicated by the ground model. For example, in critical cases, appropriate numerical models or the Sarma limit equilibrium method (Sarma, 1979) may be used to assess the mass stability of jointed rocks or saprolite (Hoek *et al*, 2000; Tattersall, 2006). The application of such methods can give insight into the uncertainties of the effect of internal shear planes or release surfaces on the operational mass shear strength, failure mode and safety margin of jointed slopes that cannot be accommodated by the other more commonly used methods of limit equilibrium analysis.

3.6.2 Verification During Construction

Development and verification of the design should continue during construction when the ground is exposed and when it is subjected to temporary and permanent changes in loadings and changes in hydrogeological conditions. Depending on the nature of the works, engineering geological input can assist in the verification and updating of the geological, ground and design models. Typical tasks include the recording and reporting of exposed ground conditions, carrying out additional investigations and interpreting responses of the ground and of the groundwater in an engineering geological context.

Existing guidance and standards pertaining to the verification of design assumptions during construction works in Hong Kong are listed below:

- PNAPs 74, 83, 85, 161 and 274 - (BD, 1993, 1994, 1997, 1998, 2003)
- BD (2004a)
- TGNs 2, 4, 10, 11, 14 & 16 - (GEO, 2004b,c,d,e,i,j)
- Geotechnical Manual for Slopes - (GCO, 1984)
- Geoguide 2 - (GCO, 1987b)
- Highway Slope Manual - (GEO, 2000)

Much of the guidance contained in GEO (2004c) for rock slopes is equally applicable to saprolite slopes. During construction there are usually ample opportunities to map exposures in temporary excavations and on slopes before surfacing.

The value of engineering geological input during construction works for different types of engineering application is further discussed in Chapter 6.

4. GEOLOGICAL PROCESSES AND ENGINEERING IMPLICATIONS

4.1 INTRODUCTION

This chapter provides a brief introduction to the key geological processes that affect the engineering characteristics of most rocks and superficial deposits in Hong Kong, i.e. processes that are not limited to any one stratigraphical or lithological unit. These processes include:

- tectonics,
- metamorphism and hydrothermal alteration,
- weathering,
- geomorphological processes, and
- hydrogeological processes.

Understanding these processes, their evolution over geological time, their spatial relationships and their effect on the engineering properties of different lithological units (Chapter 5) is key to the development of geological models for engineering purposes.

4.2 TECTONICS AND TECTONIC STRUCTURES

4.2.1 Introduction

The solid geology of Hong Kong and the current seismic and *in situ* stress regimes are a result of plate tectonics. The current plate margin lies to the southeast of Hong Kong running from Taiwan to the Philippines. However, during the late Jurassic and early Cretaceous Periods the plate margin was much closer to Hong Kong (Sewell *et al.*, 2000). This resulted in a period of intense volcanic activity with associated granitic intrusions, and also in the main pattern of faults evident today (Figure 4.2.1).

Tectonic structures include faults, folds, metamorphic fabrics such as foliation and cleavage, and tectonic joints. These structures reflect the response of the rock mass to *in situ* stress over geological time. Most faults, metamorphic fabrics and joints are discontinuities that have much lower strength than the intact material. In the case of faults and joints that are not healed by mineralisation, the tensile strength is effectively zero. Therefore, discontinuities have a major effect on the engineering properties of rock masses.

Key engineering geological issues include:

- the effect of past and present regional tectonic settings on the formation of geological structures and *in situ* stress,
- zones of deep weathering along some of the major faults and their engineering implications,
- the geotechnical influences of different types of faults,
- the development and signficance of discontinuities, including the response to stress-relief from natural and man-made sources, and
- preferential groundwater flow.

A summary of types, occurrence and geotechnical significance of discontinuities, including those that are not of tectonic origin, is given in Hencher (2000). Further information on the characteristics of discontinuities associated with common rock types in Hong Kong is contained in Chapters 5 and 6.

4.2.2 Faults

Faults can adversely affect engineering works and need to be identified and characterised during site investigations and catered for during the detailed geotechnical design. Faults are often more deeply weathered than the surrounding rocks and may contain hydrothermally altered and sheared, relatively weak material at great depth. In addition, the tectonic movements which cause brittle faulting may increase the intensity of jointing and the hydraulic conductivity of the rock mass for considerable distances on either side of a fault (i.e. brittle tectonised rock masses). The effect of faults depends on their genesis and type (e.g. high pressure ductile, low pressure brittle, or multiple ductile and brittle type movements which have affected the same fault at different times and depths). Knowledge of the overall tectonic setting facilitates the anticipation and investigation of fault-related features.

The general pattern of the main known and inferred faults is shown in Figure 4.2.1. In most instances faults are poorly exposed and obscured by superficial deposits. Minor faults with displacements of a few metres or less are commonly observed in excavations and can be traced for several metres or sometimes tens of metres. Major faults, inferred to extend for distances varying from hundreds of metres up to tens of kilometres are less commonly exposed in excavations. Consequently, there are few detailed descriptions of the major fault zones in Hong Kong (Sewell *et al.*, 2000).

Some of the limitations of the published geological maps are discussed in Sections 3.2 and 6.7.

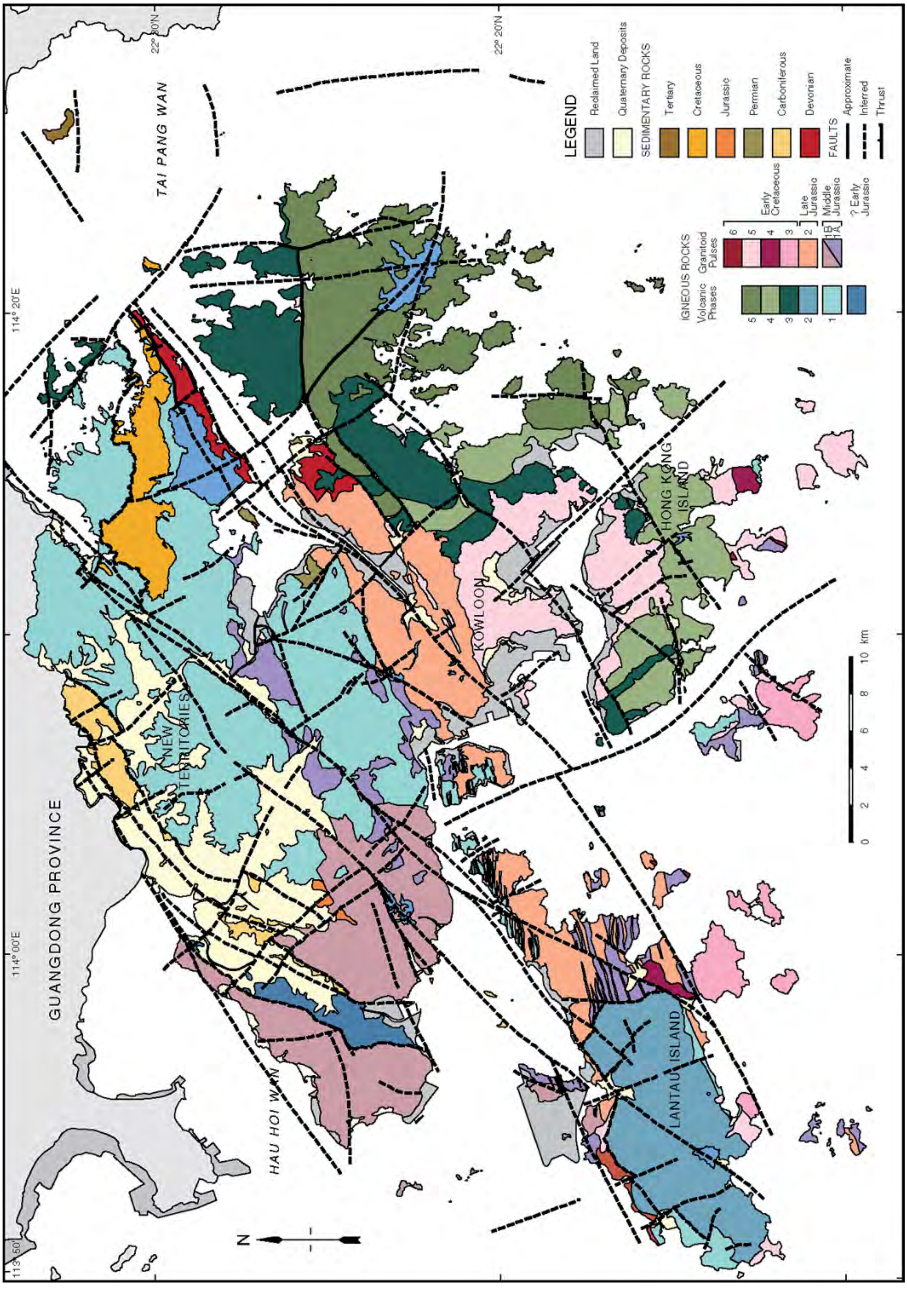


Figure 4.2.1 – Simplified geological map of Hong Kong (Fyfe et al., 2000)

Section 6.7 gives further details and examples of the assessment and influence of faulting on major tunnelling works. These discussions and examples demonstrate that an understanding of the structural geology of the area can facilitate the effective anticipation, investigation and characterisation of fault-related structures for an engineering project.

The main faults strike northeast and can be many tens of kilometres in length. These faults strongly influence the present day topography of Hong Kong. North-striking, northwest- and north-northwest-striking faults, whilst less continuous than the northeast-striking faults, can be up to 20 km in length. East-striking faults may be up to 12 km in length, but are not regularly developed in Hong Kong.

There are many different types of faults in Hong Kong. They range from slickensided or polished, relatively unaltered discontinuities to major zones of broken and sheared rock which are susceptible to weathering, and may form distinct, linear depressions in the topography. However, some faulted zones have been partially silicified by hydrothermal fluids and as a result are more resistant to weathering. This may give rise to intermittent ridges along the trace of the main fault zone.

Owing to the strike-slip and multi-phase nature of most of the faults in Hong Kong, the major fault zones can be many tens of metres in width and contain a number of smaller-scale, related faults, which can be of very different orientation to the main fault zone

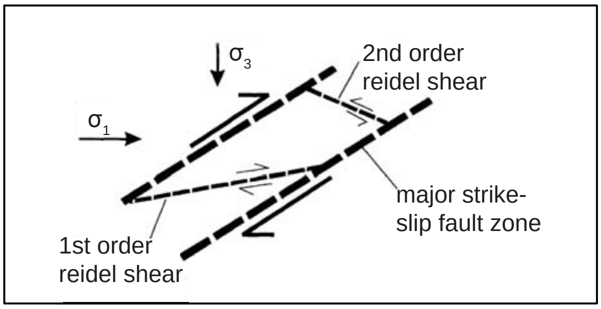


Figure 4.2.2 – Reidel shears within a major strike-slip fault zone (after Fookes et al., 2000 – based on Park, 1997)

(e.g. reidel shears - Figure 4.2.2). The discrete faults within the main zone may offset each other and form a complex arrangement of shear planes which separate relatively competent, though still partly deformed rock masses from each other (Section 6.7). The major northeast-striking fault zones are also commonly offset by northwest- and north-striking faults which can result in a structurally complex fault pattern.

The presence of individual faults or fault zones can be of major concern to engineering projects, due to influences on stability, deep weathering and the sharp contrast in engineering and hydrogeological properties between weak, faulted material and the host rock (see examples in Chapters 5 and 6). However, faults do not necessarily present insurmountable ground conditions if they are identified early in the project, particularly where they can be bridged by the foundations of surface structures or where ductile faults are intersected by tunnels where the potential

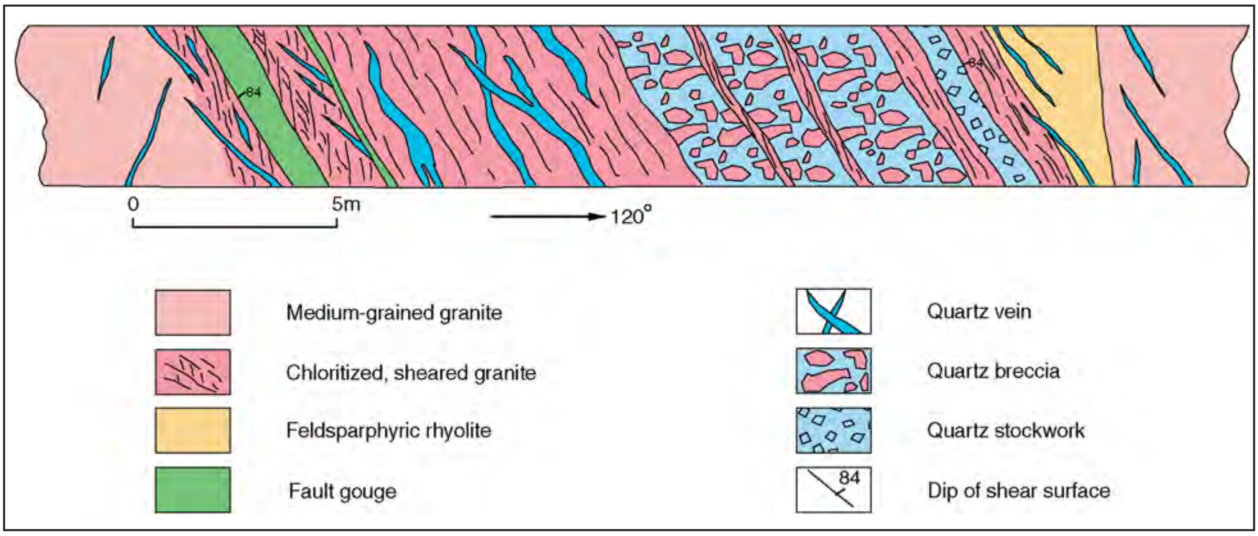


Figure 4.2.3 – Schematic section of a brittle-ductile fault zone ("Rambler Channel Fault") encountered in the Harbour Area Treatment Scheme (HATS) Tunnel 'F' between Tsing Yi and Stonecutters Island (Sewell et al., 2000)

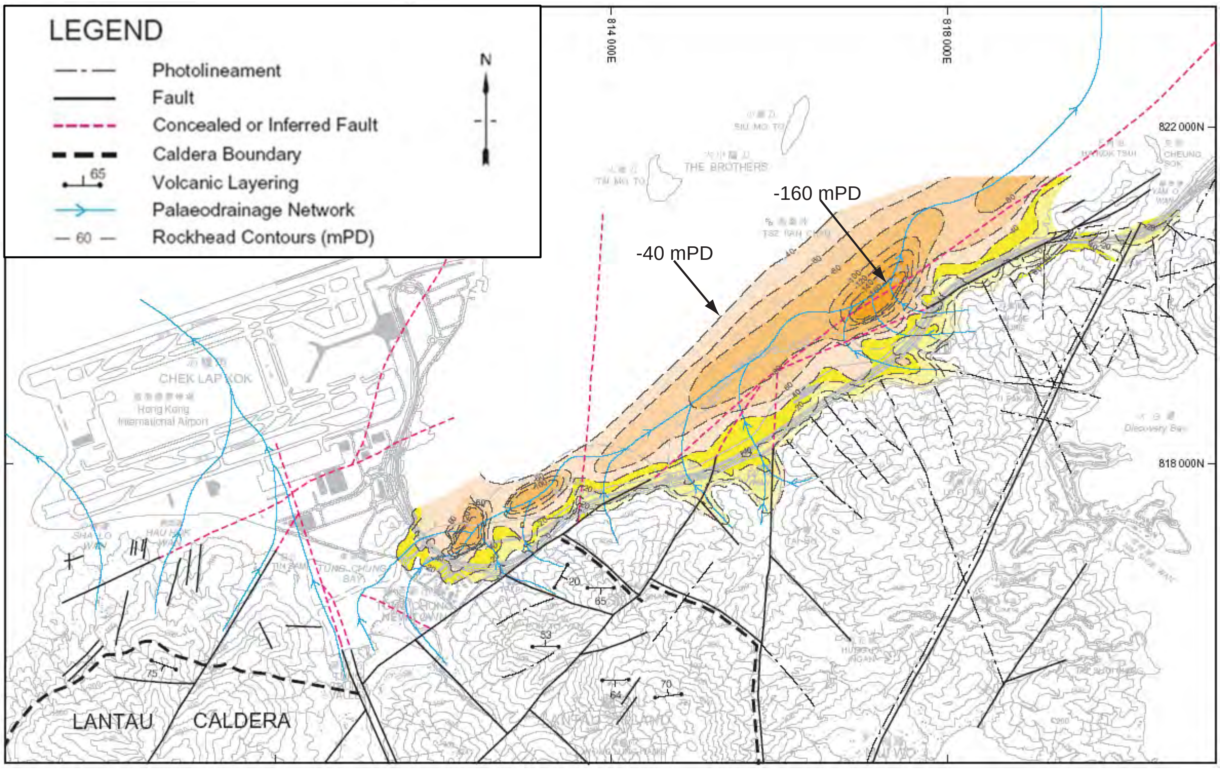


Figure 4.2.4 – Zone of deep weathering at Tung Chung (inferred from boreholes and gravity survey) in meta-sediment and marble xenolith-bearing granite. (Sewell & Kirk, 2000)

for sustained groundwater inflow is limited (see the KCRC Tai Lam Tunnel example in Section 6.7).

The style of deformation associated with the fault largely depends upon the depth of the fault zone at the time of movement. Mylonite and foliated zones are characteristic of ductile movement, while fault gouge, breccia and quartz stockwork/veining are characteristic of brittle movement. Reactivation of faults over geological time can result in both brittle and ductile styles now being evident (Figure 4.2.3).

The importance of distinguishing between individual features, such as zones of fault gouge and mylonite, and relatively less deformed zones composed of stronger rock fragments contained within the fault is illustrated by the KCRC Tai Lam Tunnel example in Section 6.7.

Some major fault zones are associated with dynamic metamorphism, and hydrothermal alteration may have taken place, resulting in a complex assemblage of variably altered and decomposed fault-slices. These can give rise to deep and steeply-sloping rockhead profiles (Figure 4.2.4). For instance, the existence and the full engineering implications of fault-related, deep and steeply-sloping rockhead profiles at Tung Chung were not recognised until investigations for the development of the area were already well advanced (see Sections 5.5 and 6.5 for further details).

4.2.3 Folds and Metamorphic Structures

Folding and the development of foliation and cleavage can be geotechnically significant due to the formation of additional discontinuities with preferred orientation, the development of bedding-plane shears, and small to large scale variations in geological structure across a site. Foliation may result in strength anisotropy which can give rise to difficulties regarding compliance with founding criteria for piles.

Most rocks have been tectonically deformed to some degree; however, it is most evident in the Palaeozoic sedimentary rocks (see Section 5.6). These are characterised by tight folding, and development of foliation and kink bands in argillaceous strata. An example of the control of foliation on the stability of a road cutting near Lok Ma Chau is given in Section 6.4.4.

4.2.4 *In Situ* Stress

In situ stresses in rock arise from the tectonic setting. Figure 4.2.5 summarises the results of selected *in situ*

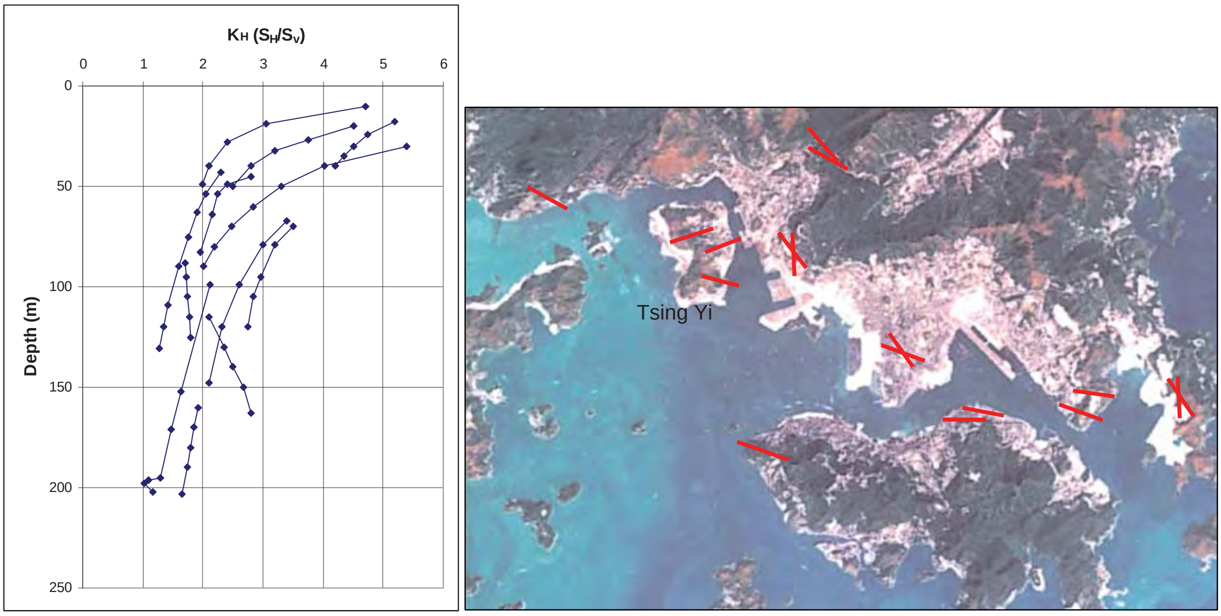


Figure 4.2.5 – Maximum horizontal stress ratio K_H (S_H/S_v) versus depth and orientations of S_H (alignment of red bars in photograph) in Hong Kong (selected data from Free et al., 2000 and Route 10 investigations)

stress measurements in Hong Kong. These show that the maximum horizontal stress (S_H) in rock is more than twice the vertical stress (S_v) at depths of less than 50 m. The scatter in stress ratio and orientation of maximum horizontal stress is due to factors such as the influence of topography at relatively shallow depth, locked-in stress from different stress regimes over geological time, and proximity to major geological fabrics and structures.

Although the principal horizontal stress directions shown in Figure 4.2.5 appear to be generally consistent with regional tectonics, the strong E-W preferred orientation of maximum horizontal stress measured on Tsing Yi (Figure 4.2.5), is consistent with N-S extension locally on the NE-trending faults (Li *et al.*, 2000) and the strong E-W -trending structure of the rock mass imparted by the intrusion of the E-W -trending dykes. When formulating geological models it is important to recognise that many different structural regimes may have existed over geological time and may have affected the directions and magnitudes of locked-in stresses.

The magnitude and direction of *in situ* stress can be important considerations in the design of underground structures in rock. For example, high horizontal stresses normal to the axis of a cavern or a tunnel are usually beneficial for stability of the roof while low horizontal stresses normal to a bridge anchorage can reduce the capacity of the anchorage to resist pull-out.

4.2.5 Tectonic Joints

Tectonic joints are formed as a result of stress in the Earth's crust and are common to most rock types. They often occur in distinctive sets, i.e. a series of parallel joints. The geometric relationship between sets may be interpreted with respect to a regional stress pattern or a geological structure such as a fault. However, such features may only be extrapolated with confidence where they are systematic and where the geological origin is understood. Tectonic joints may be formed under shear or tension. Joints formed under shear are commonly less rough than joints formed under tension and therefore might be expected to exhibit lower shear strengths (Hencher, 2006).

Figure 4.2.6 illustrates that similar jointing patterns in plutonic rocks can be widespread in areas of similar geology and stress-history. With appropriate knowledge of the regional geology and necessary caution, this principle can be used to formulate preliminary models of the potential structural geology of a site based on the mapping of nearby outcrops which are not necessarily within the site.

An investigation into the extent, orientation and distribution of discontinuities affecting engineering works will only be truly effective when the geological nature of the structure is taken into account. Recognition of type of discontinuity allows properties to be predicted and extrapolated with more confidence than could otherwise be justified. Too

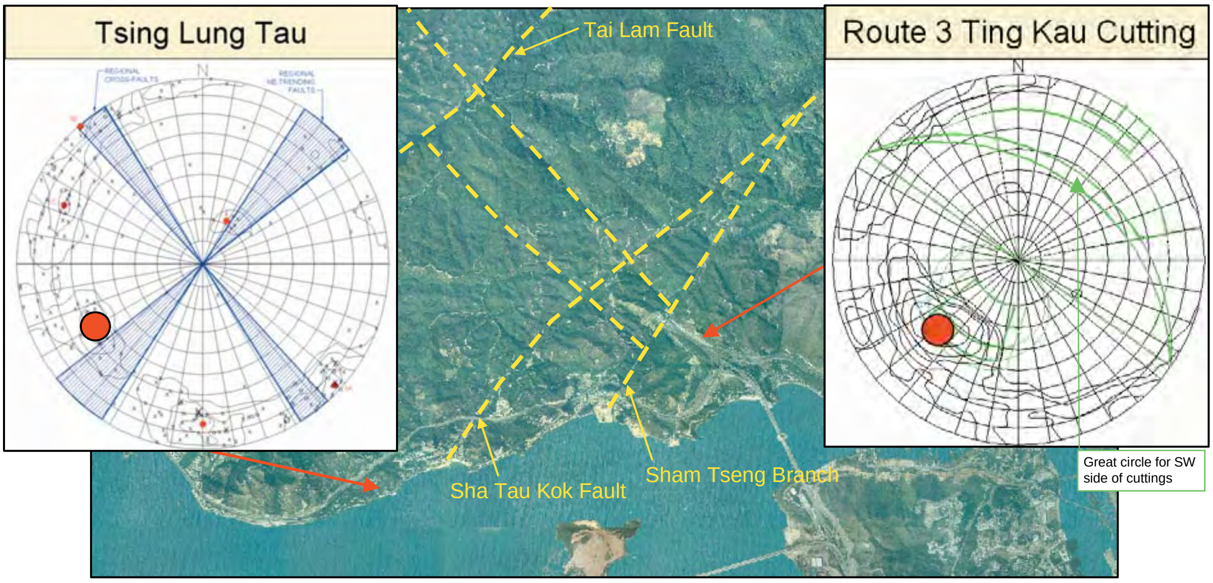


Figure 4.2.6 – Similarity in geological structure between Tsing Lung Tau and Ting Kau shown by contoured stereoplots of mapped discontinuities (major fault strikes shown on the Tsing Lung Tau plot for reference)

often the approach to the collection and processing of data is mostly statistical with only scant regard to the geological history of the site (Hencher, 1985).

Other types of joint that can affect the rock mass include cooling joints which are discussed in Sections 5.2 and 5.4, and stress-relief joints which are discussed in Section 5.2. Bedding planes are discussed in Section 5.6.

4.3 METAMORPHISM AND HYDROTHERMAL ALTERATION

4.3.1 Introduction

This section introduces the effects of metamorphism and hydrothermal alteration on the engineering properties of Hong Kong rock masses. Knowledge of these processes and skilled interpretation of their spatial relationships with other geological structures facilitate the development of realistic geological and ground models in areas where such altered rocks may be present.

Metamorphism in Hong Kong rocks can be divided into two broad classes:

- Contact metamorphism: related to temperature changes, e.g. associated with igneous intrusions.
- Dynamic metamorphism: related to pressure changes, e.g. associated with thrust faults in the northwest New Territories.

The type of metamorphic rock produced depends on the original rock material, the temperature and pressure conditions imposed and the effects of any mineral-charged fluids or gases at the time of metamorphism.

Metamorphism generally affects the engineering characteristics of rocks by:

- altering or replacing constituent minerals (typical in contact metamorphism), or
- aligning constituent minerals along a preferred orientation, i.e. foliation (typical in dynamic metamorphism).

Thus, metamorphic rocks can be broadly divided into two types, non-foliated, e.g. marble (see Section 5.5), hornfels and skarn, and foliated, e.g. phyllite and schist.

The location of igneous intrusions, and hence the potential areas of contact metamorphism are well documented, although the extent and effects of any associated metamorphism are less so. Dynamic metamorphic effects are widely found in the northwest and northern New Territories and are associated in part with fault movement (Sewell *et al.*, 2000; Figure 4.3.1).

The key engineering geological issues include:

- *Granular Recrystallisation*: hardening of the material whereby the resulting metamorphic rock has a stronger material structure. Examples of this

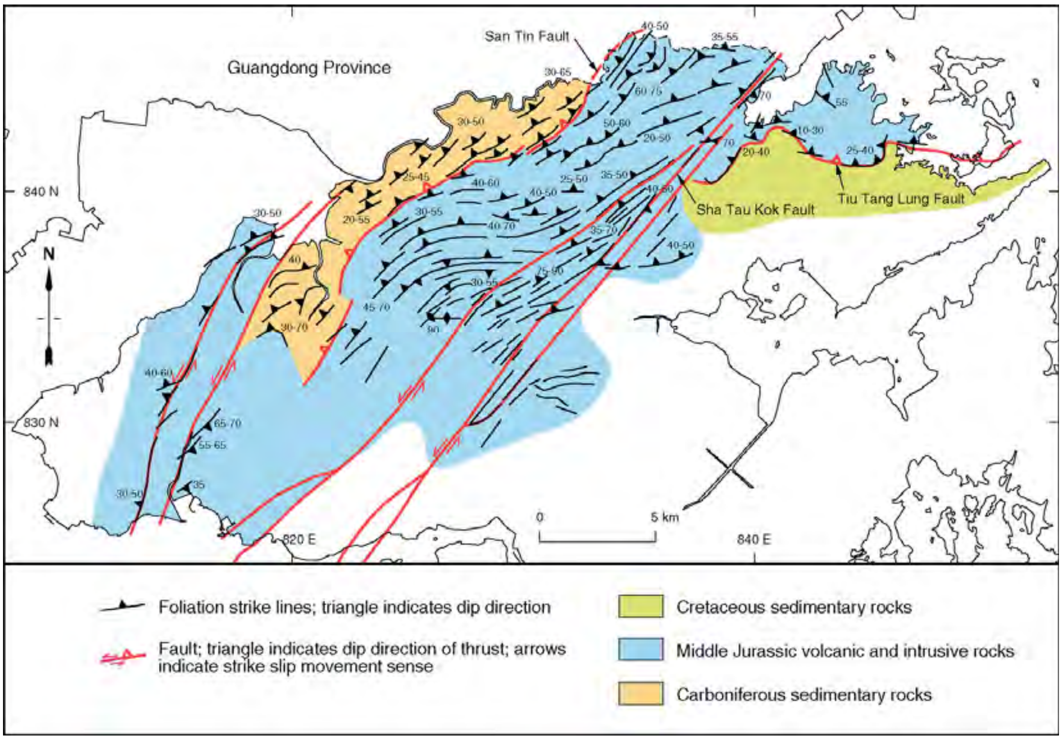


Figure 4.3.1 – Cleavage/foliation zones in the northern New Territories (Sewell et al., 2000)

type of metamorphic rock include hornfels, marble and quartzite.

- *Foliation*: alignment of minerals due to stress, forming a continuous or discontinuous penetrative planar fabric. This results in anisotropic material properties relative to the alignment of the fabric, and may represent potential failure surfaces.
- *Alteration or Concentration of Minerals:* due to heating or circulation of hot, mineral-charged fluids. These effects are generally associated with igneous intrusions and can result in a weaker rock mass such as with greisenisation which involves partial replacement of the rock with granular quartz and muscovite.

Hydrothermal alteration is mainly associated with the final stages of cooling of plutonic magma. It involves mineralisation, replacement or alteration of existing rocks by mineral-rich fluids which tended to concentrate near the boundaries of the plutons and within major joints and faults which intercept them.

The decomposition classification system used in Hong Kong for the rocks of plutonic and volcanic origin (GCO, 1988a) is not readily applicable to the metamorphic rocks and hydrothermally altered rocks as the strength of these rocks in the fresh state is not comparable with the granite and volcanic rocks which form the basis of the classification. This is especially true where anisotropic strength is developed due to foliation. Other classifications such as those described in BS 5930 (BSI, 1999) may be more appropriate for such rocks.

4.3.2 Dynamic Metamorphism

Dynamic metamorphism in the northwest and northern New Territories has affected the sedimentary rocks and areas of adjacent tuff by varying degrees (Figure 4.3.1) depending on their location and original composition.

The key engineering geological effect of regional metamorphism is the development of foliation. Typically, foliation in the New Territories is inclined to the north or northwest (Sewell *et al.*, 2000), and where this foliation orientation coincides with an unfavourable slope aspect and angle, instability can result (see the case studies in Section 5.7).

4.3.3 Contact Metamorphism

Contact metamorphism results in mineralogical changes, which vary depending on temperature. As a result, the metamorphic effect reduces with distance from the igneous body. For example, at Victoria Peak (Figure 4.3.2) the effects of metamorphism are evident over 500 m from the contact (Strange & Shaw, 1986), but often only when subjected to microscopic examination. However, closer to the

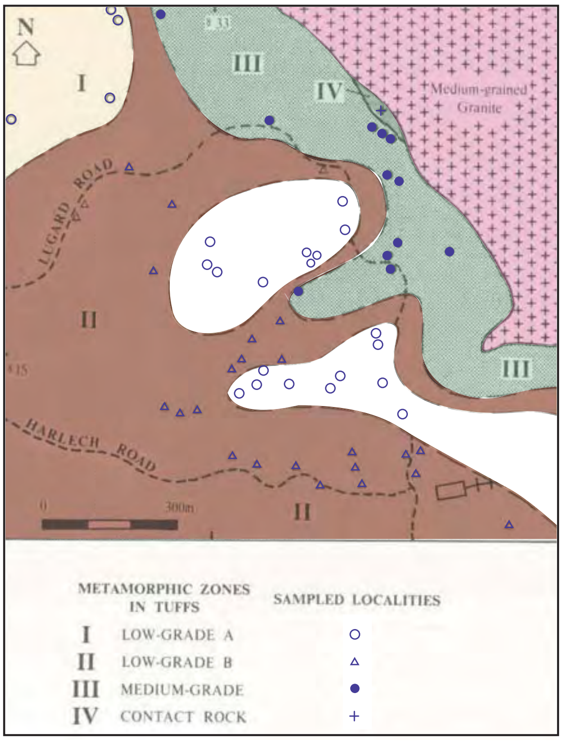


Figure 4.3.2 – Development of contact metamorphic effects along a granite/tuff contact zone near Victoria Peak, Hong Kong Island (Strange & Shaw, 1986)

contact hornfels occurs resulting in an increase in rock strength (Figure 4.3.3).

Thermal metamorphism associated with volcanism and plutonism has resulted in the alteration of limestone and dolomite into marble (see Section5.5), with recrystallisation resulting in an increase in rock material strength.

4.3.4 Hydrothermal Alteration

Hydrothermal alteration is the alteration of rocks by high temperature fluids. In Hong Kong, this may result in chloritisation, kaolinisation or silicification. Within the pluton itself, greisenisation may occur.

Replacement of ferromagnesian minerals by chlorite is relatively common near contact zones, and may lead to a reduction in overall strength of the rock and give rise to low-friction discontinuities coated with chlorite which can have implications for slope stability. Hencher (2000) reports a failure in Aberdeen where the chlorite-coated discontinuity had very low shear strength. In addition, hydrothermal alteration can result in economic mineral deposits which have been mined in the past (Section 6.10).

Kaolinisation results from the alteration of feldspars and has a similar occurrence to, but more pronounced effect than, chloritisation. Figure 4.3.4 shows a granite core sample which contains many pits and voids resulting from kaolinisation and dissolution of the feldspar crystals. Until the mid-1990s it was considered that any significant, laterally-persistent concentrations of kaolin were formed as a result of hydrothermal alteration. More recently, Campbell & Parry (2002), GEO (2004i) and Parry *et al.* (2004a) suggest most near-surface, kaolin-rich zones are related to weathering. However, hydrothermal kaolin

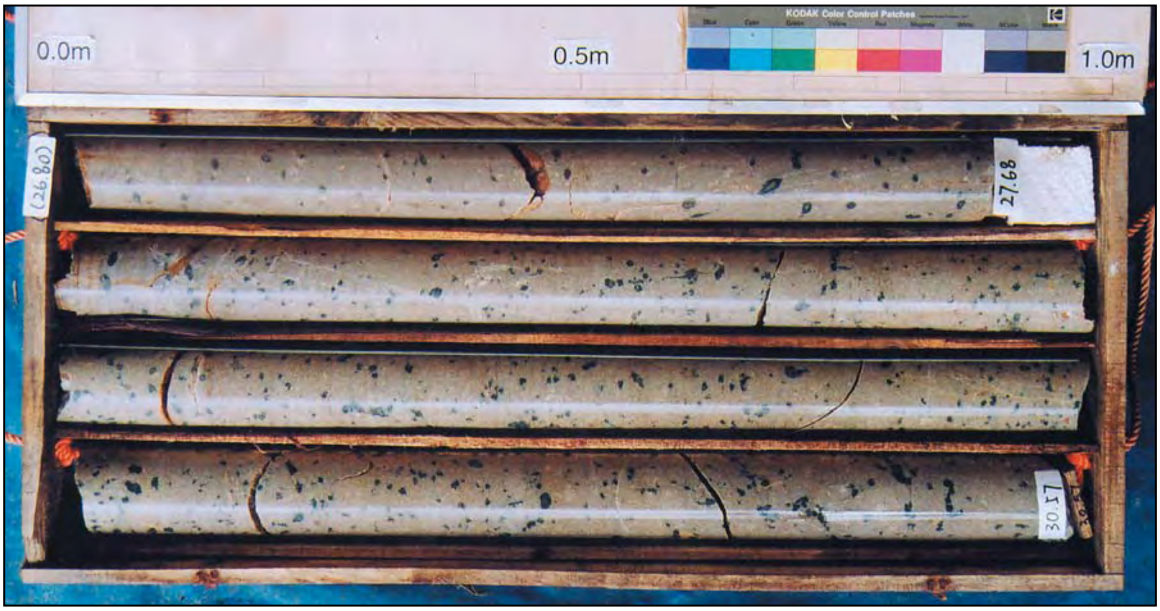


Figure 4.3.3 – Hornfels close to the contact between granite and tuff near Victoria Peak, Hong Kong Island (note extensive re-crystallisation and the development of spotting)

Figure 4.3.4 – Kaolinised granite core from the 1997 Ville de Cascade landslide site at Fo Tan (HAPL, 1998a)

is locally important and can affect the ground to a considerable depth below the soil to rock interface.

Silicification by replacement of much of the rock-forming minerals by quartz can occur in the vicinity of fault and plutonic contact zones. Crystallisation of quartz along discontinuities can also occur . These processes can give rise to veins or zones with extremely high compressive strength which are resistant to weathering and can also result in the healing of discontinuities, including faults (see Figure 4.2.3).

Greisenisation involves partial replacement of the rock with granular quartz and muscovite. Greisenisation can result in complete loss of crystal bonding in granites, giving a friable, granular texture.

4.4 WEATHERING

4.4.1 Introduction

This section reviews the processes of weathering *in situ* that are relevant to the development of typical ground models and classification systems in the igneous rocks of Hong Kong. The discussion is predominantly based on data from plutonic and volcanic rocks. Although the main processes and effects of weathering in clastic sedimentary and metamorphic rocks are similar to the igneous rocks, alternative classification systems as outlined in BS 5930 (BSI, 1999) may be more applicable due to their generally lower strength when fresh and closer spacing of the discontinuities. The weathering of carbonate rocks is mainly due to solution and removal of calcium carbonate by groundwater (see Section 5.5).

The two main components of weathering are mechanical disintegration and chemical decomposition. In view of the dominance of chemical weathering in Hong Kong, material weathering grades are classified using the term 'decomposed' rather than the more general term 'weathered'.

Key engineering geological issues include:

- decomposition of the original minerals to low strength clay minerals,
- growth of pore spaces and an increase in void ratio, causing increases in porosity and possibly in permeability, and with reduction in grain bonding, thereby decreasing material strength,
- growth of microfractures,
- retention of geological structure and fabric in saprolite, which may result in heterogeneous variations in mass shear strength and permeability,
- concentration of clay minerals along discontinuities, particularly in saprolite close to interfaces between rock and soil,
- variations in weathering intensity and depths giving rise to difficulties in defining rockhead, and

- the presence of corestones and heterogeneous masses giving rise to difficulties in estimating mass shear strength, deformability and permeability.

All of the issues listed above can give rise to complex weathering profiles, which may require a thorough understanding of the weathering processes and considerable engineering geological input to enable realistic models to be formed.

Material and mass weathering classification systems have been developed to characterise the variability of weathered *in situ* rock masses for geotechnical design purposes. They are most effective when used within a well-planned investigation and design framework. This facilitates the development of ground models from the results of investigations, including *in situ* tests and laboratory tests, which have been conducted on volumes of ground that are normally several orders of magnitude smaller than the mass affecting or affected by the proposed engineering works.

Accounts of weathering and the development of weathering classification systems that are relevant to rocks in Hong Kong can be found in GCO (1988a), Martin & Hencher (1988), Anon. (1995), Irfan (1996a,b & 1998a), Fookes (1997b) and BSI (1999).

4.4.2 Mechanical Disintegration

Disintegration is caused by physical processes such as absorption and release of water, changes in temperature and stress, and frost action (minor in Hong Kong).

Microfractures (i.e. fractures only visible using a microscope) are evident even in fresh rock and are associated with tectonic or cooling stresses. As stress-relief occurs due to the removal of overburden over geological time, microfracturing develops further, either extending pre-existing microfractures or by the development of new microfractures generally parallel to joint planes. Most microfractures caused by chemical weathering probably arise from the de-stressing of quartz and feldspars as the rock-forming minerals decompose and change in volume, strength and stiffness (Irfan, 1996a).

Fractures that are visible to the naked eye occur in certain settings and rock types, especially coarse grained, plutonic rocks (Figure 4.4.1). These are most likely due to the effects of stress-relief. This may lead to increases in frequency, aperture and inter-connectivity of pre-existing microfractures. Finer grained plutonic and extrusive volcanic rocks which have crystallised at lower pressures appear to be much less susceptible to this type of disintegration.

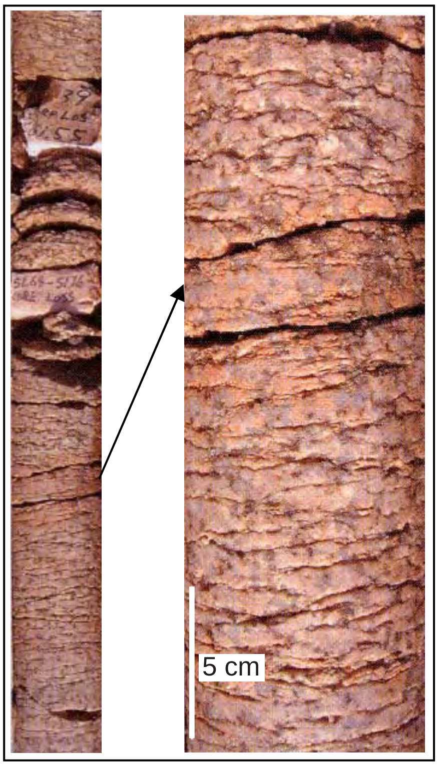


Figure 4.4.1 – Sub-horizontal fractures in a granite core due to stress relief (Fletcher, 2004)

4.4.3 Chemical Weathering

Chemical weathering results in variable decomposition and solution by hydrolysis of the rock-forming minerals to more chemically stable components. This process is primarily promoted by the circulation of groundwater in pre-existing discontinuities. Both the discontinuities and the mineralogical variations within the rock mass are not uniform. As a result, the degree of chemical decomposition varies in three dimensions. This can give rise to complex assemblages of materials with different engineering properties (mass weathering).

The six-fold grade system used for the material description of igneous rocks in Hong Kong is shown in Figure 4.4.2, along with a schematic depiction of the main processes and effects. Using granite as an example, a model of variation in mineralogy with chemical decomposition is shown in Figure 4.4.3, while Figure 4.4.4 provides a detailed breakdown of the processes and diagnostic characteristics of each

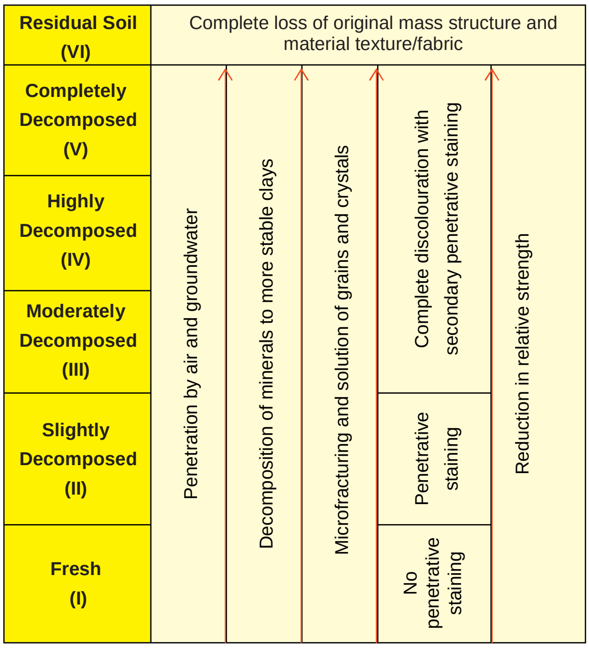


Figure 4.4.2 – Six-fold classification of material decomposition, processes and effects in a sub-tropical environment

decomposition grade. However, it should be noted that the ratio of the different minerals with decomposition may vary from that shown in the figures due to local environmental factors (Irfan, 1996b; Campbell & Parry, 2002). Table 4.4.1 provides a definition of the terms used for the description of the degree of decomposition of feldspars.

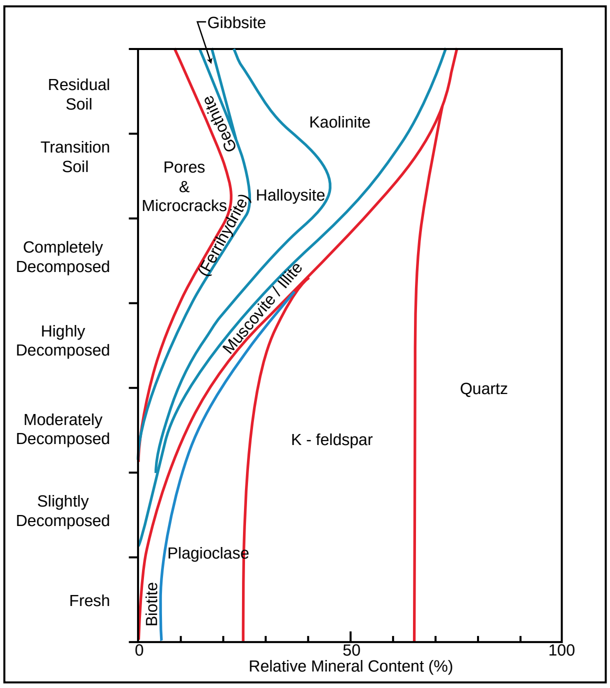


Figure 4.4.3 – Variation in mineralogical and pore composition with decomposition in a typical granite (Irfan, 1996b)

Grittiness Term	Description
Hard	Cannot be cut by knife; cannot be grooved with a pin.
Gritty	Can be cut with a knife or grooved with a pin under heavy pressure.
Powdery	Can be crushed to silt sized fragments by finger pressure.
Soft	Can be moulded very easily with finger pressure.

Table 4.4.1 – Terms for describing decomposition of feldspars (Irfan, 1996b)

As the system is intended for the classification of relatively homogeneous materials, without consideration of mass characteristics, it is mainly applicable to the description of drillhole samples, laboratory test specimens and material blocks in the field.

Although each grade of decomposition may represent a likely range of strength, such relationships are not definitive. Processes such as microfracturing and disintegration caused by tectonic stresses and stress relief, or loss of crystal bonding and alteration due to hydrothermal action, can affect strength significantly. For example, granite of chemical decomposition Grade III or IV may be so mechanically altered that it can be broken down into its constituent grains by finger pressure. Similarly, foliation fabric can result in anisotropic strength. In such cases, it would be normal practice to record the state of disintegration and to supplement the main rock description with a separate soil description if applicable.

Whilst the six-fold grading system is generally applicable to all the igneous rocks, it should only be extended to other types of rock that exhibit similar gradational weathering characteristics. Where this is not the case the alternative approaches given in BS 5930 (BSI, 1999) may be applicable. Marble weathers by dissolution with little transition between the fresh rock and soil, and the classification system reported in Chan (1994) and summarised in Section 5.5 is commonly used.

Because the whole purpose of description and

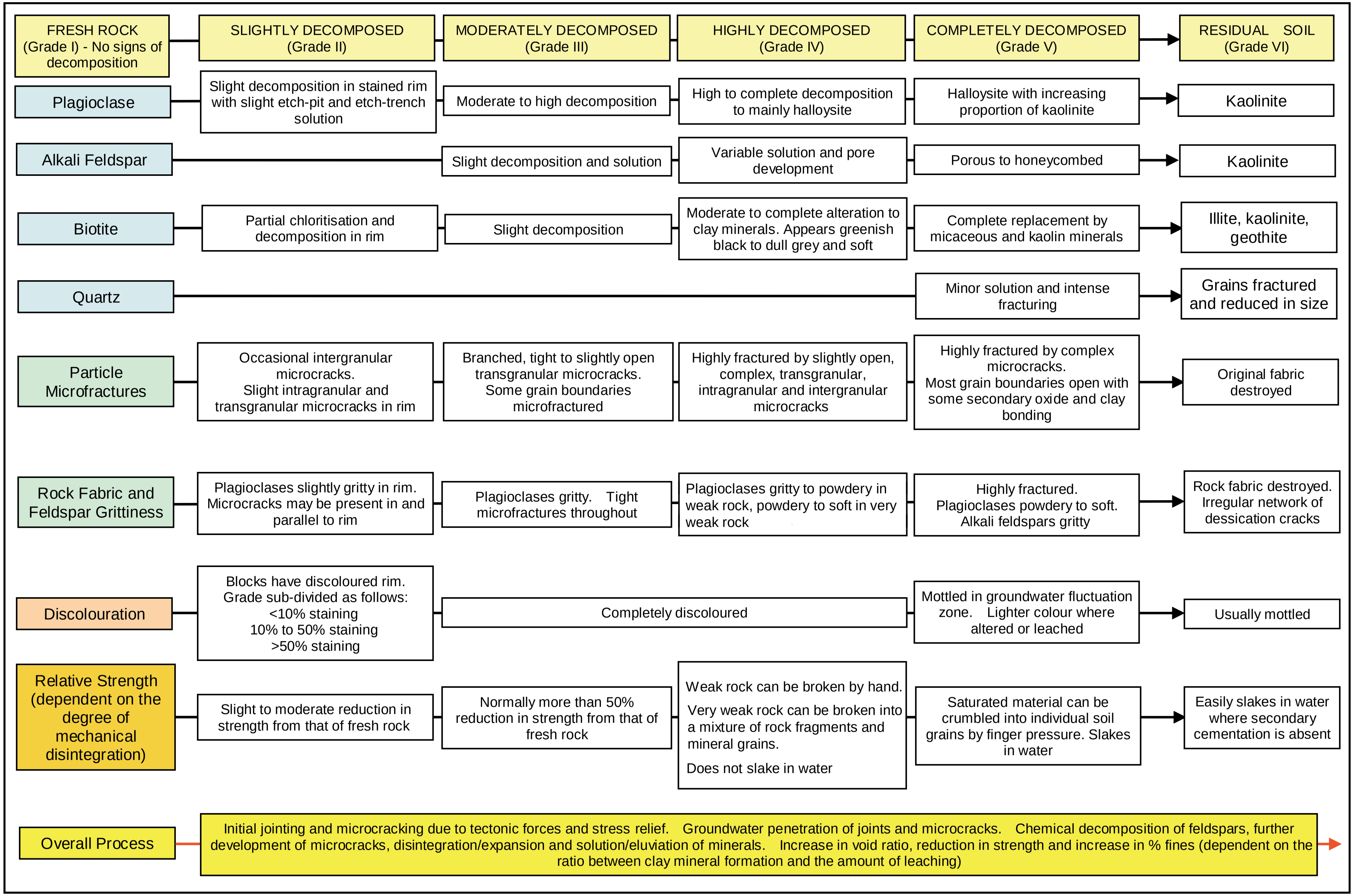


Figure 4.4.4 – Grades and characteristics of granitic rock material subject to primarily chemical decomposition processes (after Irfan, 1996b)

"CDG" End Member	Fabric and Mineralogy D=dominant, SD=sub- Tr=trace	Grading (on remoulding)	Range of Engineering	
'Strong' End – bordering HDG	Little alteration to quartz (D). Alkali feldspars some pores, still gritty (SD). Plagioclase feldspars partially to completely decomposed => halloysite (Tr). Clay mineral content 10-25% by weight.	Silty/clayey very sandy GRAVEL or very gravelly SAND (Gravel 30-50%, Sand 30-50%, Silt + Clay 10-20%)	SPT-N γ_d(Mg/m^3) e c′(kPa) Ø′ (deg.)	60-120 1.6-1.8 0.4-0.6 0-10 38-44
'Weak' End – bordering transition to Residual Soil	Mircofracturing, some solution of quartz (D, but lower %). Alkali feldspars porous, honeycombed => kaolinite (A). Plagioclase feldspars virtually absent => halloysite and kaolinite (Tr - Nil). Clay mineral content 30-50% by weight.	Slightly gravelly, sandy SILT/CLAY (Gravel 10-30%, Sand 30-50%, Silt + Clay 30-45%)	SPT-N γ_d (Mg/m^3) e c′ (kPa) Ø′ (deg.)	10-40 1.2-1.5 0.7-1.1 2-6 33-36
Note: Grading/properties are for free-draining, coarse-grained granite on a sloping site.				

Table 4.4.2 – Range of engineering properties within a completely decomposed granite weathering profile (Martin, 2003)

categorisation of material is to facilitate the engineering of a project, the development of more detailed or specific descriptive weathering systems can be beneficial where they result in better definition and understanding of the ground conditions (GCO, 1988a).

The different grades of decomposed rock, particularly moderately, highly and completely decomposed, may exhibit a wide range of engineering properties. Table 4.4.2 shows a range of engineering properties for a profile of completely decomposed, coarse-grained granite on well drained sloping terrain. The variations in engineering properties are primarily due to differences in void ratio and microfracturing which may be indirectly correlated with the ranges of SPT-N values (Pun & Ho, 1996). However, the example given in Table 4.4.2 is only intended to illustrate the ranges in properties that can occur within one decomposition grade. Although similar trends may be found at different sites, the actual parameter ranges are likely to be different due to local variations in lithology, alteration, and drainage and weathering environments over geological time. Hence, site-specific investigations and laboratory testing are required to establish representative design parameters.

As noted in Martin (2003), it may be appropriate to sub-divide thick zones of saprolite where a large range in strengths is clearly evident, provided that the boundaries between the different zones can be reliably depicted in the geological model.

Correlations between grades of decomposition with field index tests using the Schmidt hammer , for rock, and hand penetrometer , for soil, have been developed to aid in distinguishing between highly and completely decomposed materials (Martin, 1986), and have been used to define sub-grades at specific sites (Irfan, 1996a,b). Although comparison of results from different studies may show variations due to different moisture conditions, soil grading and operator technique (Irfan, 1996a,b), these index tests can be useful in providing additional means to determine decomposition grades or sub-divisions on a site-specific basis.

4.4.4 Mass Effects

Chemical decomposition in rock masses is concentrated along the discontinuities which form the boundaries of rock blocks, and proceeds inwards via microfractures. The degree of penetrative weathering is also influenced by the rate of circulation of groundwater over geological time and the zone of wetting and drying resulting from seasonal changes in the groundwater table. These effects can result in a complex assemblage of variably decomposed soil and rock blocks (corestones). Where the transition between 'soil' and 'rock' masses is gradational, problems in defining rockhead for foundation design can occur . Corestone development is common in widely-jointed, coarser grained rocks, whereas in finer grained rocks with relatively close discontinuity spacing, corestones are relatively rare

Figure 4.4.5 – Sharply-defined rockhead with little corestone development in volcanic rock. A possible kaolin-rich zone lies directly above rockhead (Fyfe et al., 2000)

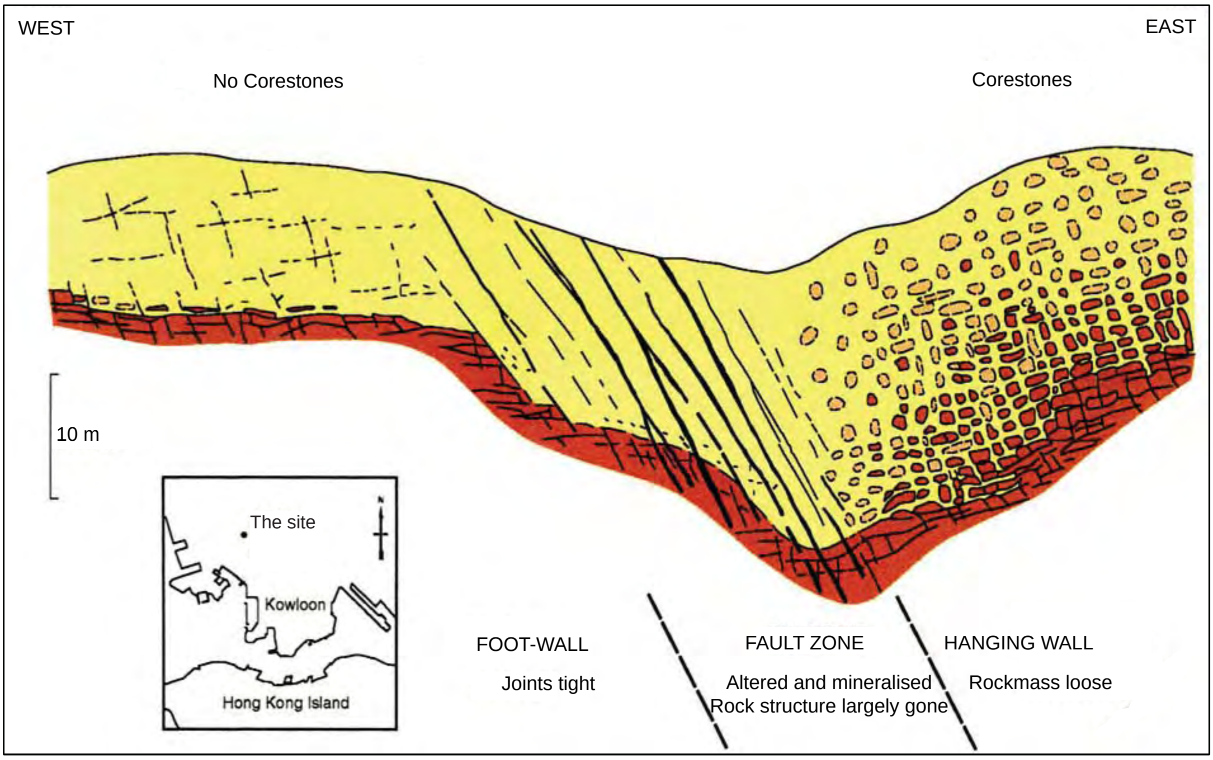


Figure 4.4.6 – Corestone development influenced by looseness of the rock mass on the hanging-wall side of a fault in northwest Kowloon (Whiteside, 1988)

and the soil to rock interface is often sharply defined (Figure 4.4.5).

In some situations, particularly where steeply dipping joints are relatively widely spaced, or where the flow of groundwater is concentrated in low-angle stress-relief joints (see Section 5.2.4), the weathering may bypass large tabular blocks of rock, resulting in coreslabs being left within the weathering profile (Fyfe *et al.*, 2000).

Figure 4.4.6 shows an actual weathering profile in northwest Kowloon, based on the logs from over 100 hand-dug caissons and mapping records of temporary foundation excavations. The overall rockhead profile is influenced by a major fault zone, joints sub-parallel to the fault creating sharp steps up to 8 m in height, and sub-horizontal sheeting joints. On the footwall side of the fault, the joints were very tight, and few corestones were encountered. The development of a thick zone of corestones, resulting in a much more gradational weathering profile on the hanging-wall side of the fault, was attributed to the looser condition of the rock mass and preferential groundwaterflow in the open discontinuities (Whiteside, 1988).

Where corestone-rich masses become exposed at the ground surface, the additional resistance of the corestones to erosion causes them to weather proud of the soil profile, leading to the development of tors of bare rock and surface boulders.

The main scheme that is used in Hong Kong for the classification of rock mass weathering (Partial Weathering or 'PW' scheme) is shown in Table 4.4.3 (GCO, 1988a). An older scheme based on Ruxton & Berry (1957) is included in the Geotechnical Manual for Slopes (GCO, 1984). The PW scheme is based on the percentage volume of rock within the mass and whether or not the soil matrix retains the mass structure, material texture and fabric of the parent rock (i.e. saprolite). In most cases information on the three dimensional extent of the mass is limited and it is usually only possible to make a rough estimate of the percentages. Although GCO (1988a) defines saprolite as the non-rock material within the partially weathered (PW 90/100 to PW 0/30) mass, the term is also commonly used when referring in general terms to the PW 0/30 and PW 30/50 masses.

For some engineering applications, in addition to the PW scheme, more detailed descriptions may be required. For example, the ground may need to be defined in terms of the proportions of material with different weathering grade, rock strength, percentage core recovery, joint spacing and joint condition, and

Geoguide 3 - GCO (1988a)				GCO (1984) – based on Ruxton & Berry (1957)	
Description		Symbol	Characteristics	Zone	Characteristics
Residual Soil		RS	Residual soil derived from insitu weathering; mass structure and material texture/fabric completely destroyed.	A	Structureless sand, silt and clay. May have boulder concentration at the surface.
Partially Weathered Rock	0 / 30% Rock	PW 0 / 30	Less than 30% rock. Soil retains original mass structure and material texture/fabric (i.e. saprolite). Rock content does not affect shear behaviour of mass, but relict discontinuities in soil may do so.	B	Residual material with corestones. Rock percentage is less than 50% corestones are rounded and not interlocked.
	30 / 50% Rock	PW 30 / 50	30% to 50% rock. Both rock content and relict discontinuities may affect shear behaviour of mass.		
	50/ 90% Rock	PW 50 / 90	50% to 90% rock. Interlocked structure.	C	Corestones with residual material. Rock percentage is 50% to 90% and corestones are rectangular and interlocked.
	90 / 100% Rock	PW 90 / 100	Greater than 90%. rock Small amount of the material converted to soil along discontinuities.	D	More than 90% rock. Minor residual material along major discontinuities which may be considerably iron stained.
Unweathered Rock		UW	100% rock. May show slight discolouration along discontinuities.		

Table 4.4.3 – Comparison of Geoguide 3 (GCO, 1988a) and Geotechnical Manual for Slopes (GCO, 1984) schemes for classifying weathered rock masses

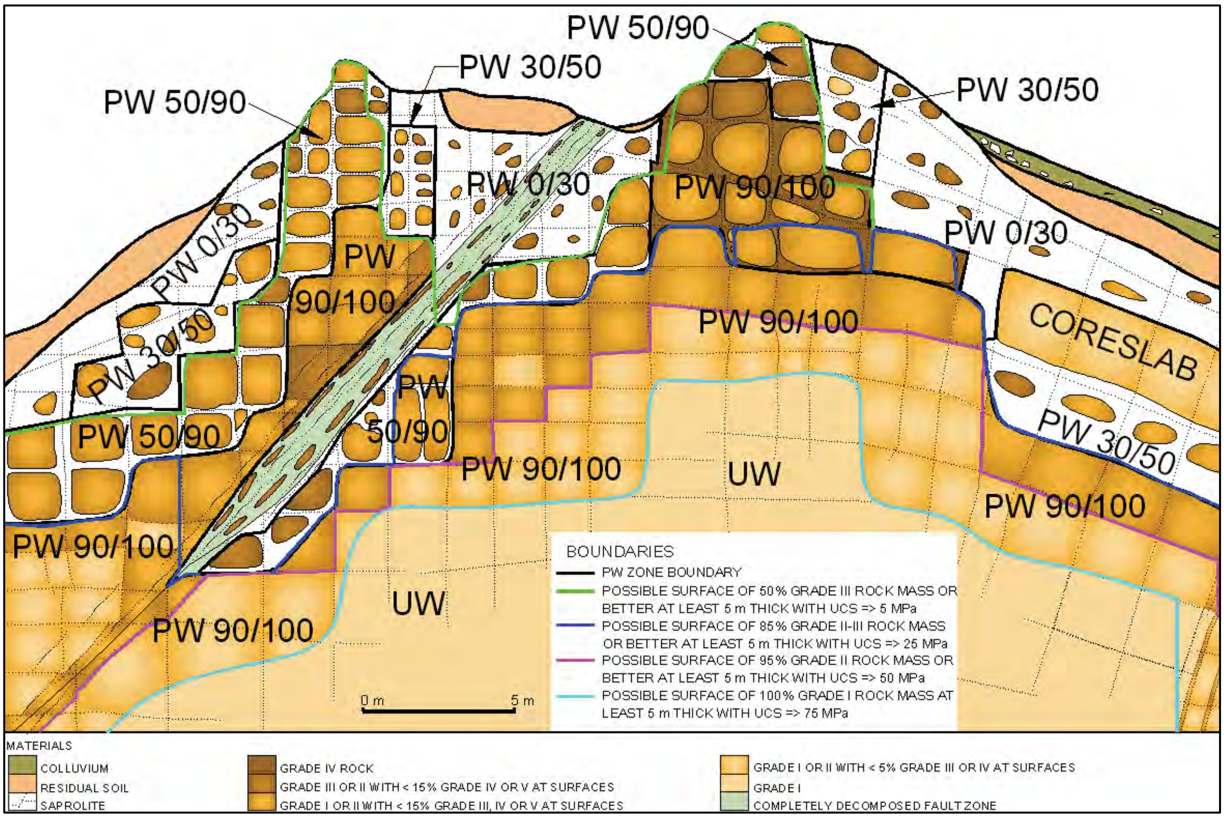


Figure 4.4.7 – Schematic PW weathering scheme applied to a mass exposure and other possible boundaries based on engineering requirements (after GCO, 1988a)

in some cases, a rock mass classification system. Where descriptive terms such as 'rockhead' are used, their engineering geological characteristics and scope of applicability should be well defined to avoid ambiguity.

Figure 4.4.7 shows a schematic weathering profile which demonstrates the PW scheme applied to a mass exposure and shows boundaries based on other possible engineering requirements.

Figure 4.4.8 shows a schematic depiction of rock volume percentages, some possible correlations with rock core recovery, and an illustration of the potential effect of the ratio between block size and excavation dimensions on the relative ease of excavation. The correlations with rock core recovery demonstrate that the rock volume percentage can be much lower than the percentage of rock recovered, particularly where the blocks are evenly spaced and equi-dimensional (see Figure 4.4.8 c, d). However, the differences will tend to be smaller where the blocks are more randomly arranged, or where they are markedly anisotropic in dimensions (see Figure 4.4.8 e).

Figure 4.4.8 c, d also demonstrate that the percentage of rock recovered in a drillhole may vary depending on its location relative to the vertical columns of blocks. The chance of encountering no rock at all in differently positioned vertical drillholes would vary (compare Figure 4.4.8 c, d). Although the block arrangement is highly stylised, similar situations can occur where development of corestones is strongly influenced by sub-vertical jointing. A thorough understanding of the development and associated distribution of corestones and their likely variations needs to be exercised to plan any necessary investigations to better define the actual ground conditions.

Figure 4.4.8 also highlights possible differences between area measurements in 2-dimensional exposures and equivalent PW percentage by volume. With care, similar diagrams can be constructed to aid visual assessments of PW percentages in outcrops or excavations.

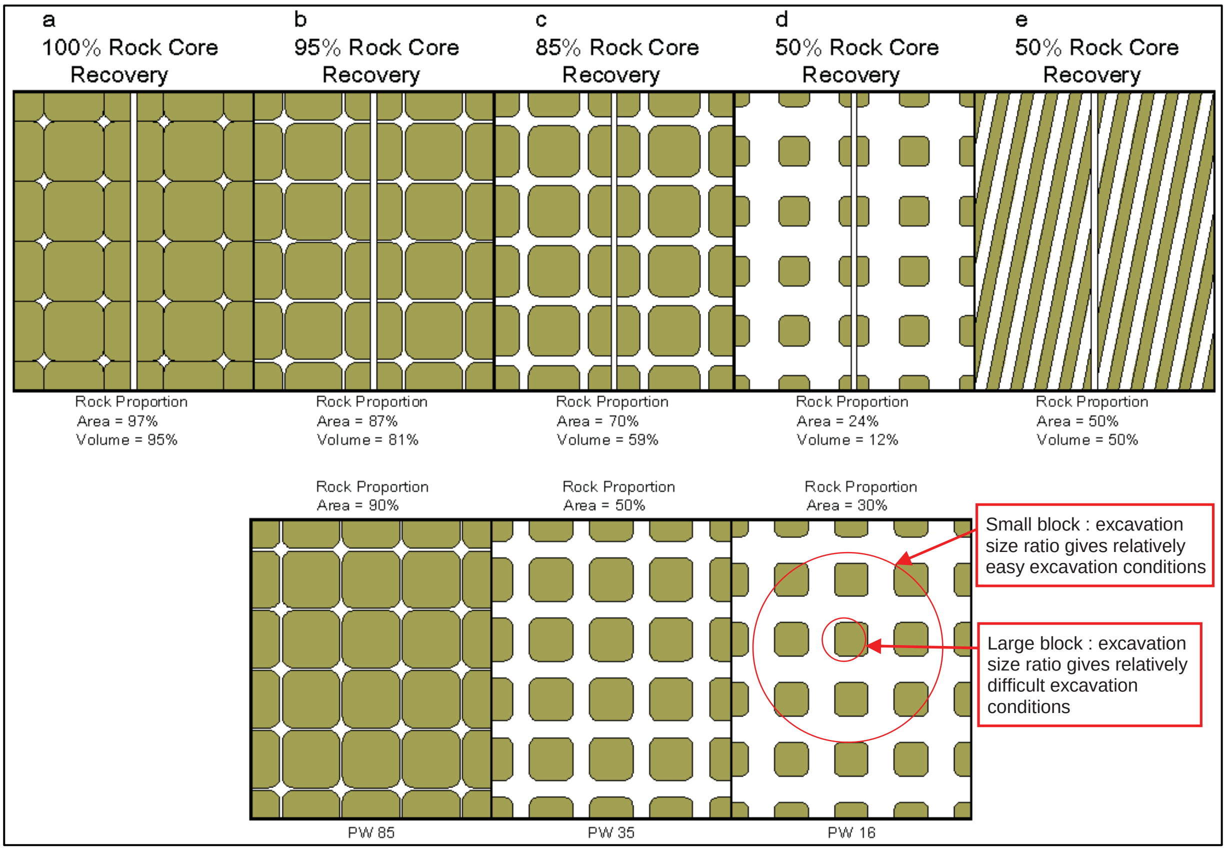


Figure 4.4.8 – Schematic depiction of rock percentage and correlation with rock core recovery (assuming recovery of 100% of the rock material). Very idealised jointing patterns and equi-dimensional blocks are shown. The percentage of rock in the ground in proportion to the percentage of rock core recovered will be larger with increasing randomness in block arrangement and increasingly poor drilling practice.

Although the presence of corestones can enhance the stability of slopes due to deflection of any potential shear surfaces through the soil mass, an important caveat is that the presence and orientation of any discontinuities within the saprolite or within the corestones that could reduce shear strength or lead to adverse groundwater conditions must also be investigated and their effect on stability should be assessed (Irfan & Tang, 1993).

There is evidence that some ground movement takes place during weathering to accommodate changes in stress. Such movement may result in the generation of slickensides on relict joints resulting in a reduction in shear strength (Parry *et al.*, 2000). Hencher (2006) suggests that additional joints may be generated by such processes.

4.4.5 Variation in Engineering Rockhead

The term 'rockhead' as used in engineering is the level at which the engineering parameters of the rock mass satisfy the design parameters for the project. These requirements vary considerably, for example rockhead can signify the depth to which the ground can be excavated mechanically without blasting, or it can signify the top of rock with a required bearing capacity. As such, engineering rockhead is project and site specific and its determination can be one of the most critical issues for construction purposes (Figure 4.4.7).

Given the complex inter-relationship between lithology, fabric, structure and weathering, the level of engineering rockhead across a site is usually subject to considerable variation. However, engineering geological skills and experience can be applied to reduce the uncertainty.

Major variations in engineering rockhead level are commonly caused by geological structures (see Section 4.2). On a small to medium scale, the presence of corestones and steep steps in engineering rockhead along individual discontinuities may lead to irregularities in the rockhead profile which can be much more pronounced than data from widely-spaced drillholes may indicate (Shaw, 1997). The probability of a widely-spaced drilling pattern intersecting the lowest and highest points of an irregular rockhead profile is small.

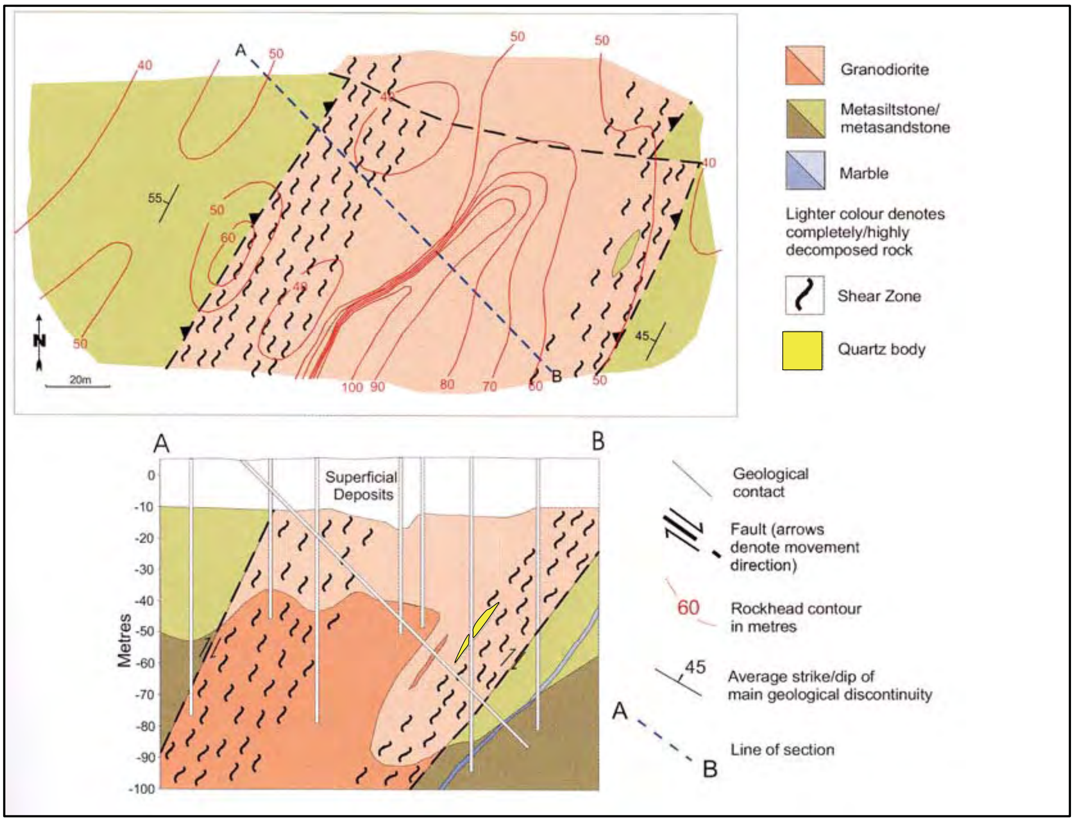


Figure 4.4.9 − Schematic geological map and section developed from borehole data and local geological knowledge showing highly variable rockhead surface (Fletcher, 2004)

Figure 4.4.9 shows a schematic geological map and section for a site with a highly variable engineering rockhead profile caused by a granodiorite intrusion which is sheared along its flanks. Whilst some initial drillhole data was available, knowledge of the regional and local structural geology was necessary to produce a representative geological model of the site. An inclined drillhole later confirmed the model. Smooth rockhead profiles drawn between the drillhole locations can only reflect the general profile, since weathering progresses preferentially along the discontinuities. In this example, the inclined rock structure could lead to overhanging rock or quartz veins with intervening saprolite which could prove problematic for bored pile foundations.

4.4.6 Subsurface Processes

General

Subsurface erosion and transportation (eluviation) of the fine materials and solutes produced by weathering *in situ* is an important process which leads to the creation of interconnected voids in saprolite and transported soils. This increases the void ratio and permeability of the soil. Where sufficient hydraulic gradient exists in saprolite and transported soils, through-flow of groundwater may cause larger scale internal erosion leading to the development of soil pipes (Nash & Dale, 1984). These can be of major hydrogeological importance (Section 4.6) and have also been implicated in many slope failures (Section 6.4) where they have become blocked or constricted due to collapse or sedimentation.

Deposition (illuviation) of material, for instance in joints, can lead to decreases in permeability. High concentrations of low strength clay minerals such as kaolin have been implicated in some large scale slope failures in Hong Kong, e.g. Campbell & Parry (2002).

Solution was the major process in the formation of the buried karst in Hong Kong. Solution of pure marble leaves only minor amounts of residual material which may be deposited in the cavities along with other, in-washed detrital material (see Sections 5.5 and 6.5 for further details).

Eluviation and Development of Soil Pipes

Eluviation involves the transportation of solutes and fines produced as a by-product of *in situ* weathering of rock masses (Section 4.4.3) by intergranular flow of groundwater through the soil mass, and also via pervasive, interconnected pores, open joints and soil pipes which may range in aperture from less than 0.5 mm to more than 1 m. This process is also common in superficial deposits such as colluvium where groundwater flow occurs (Figure 4.4.10).

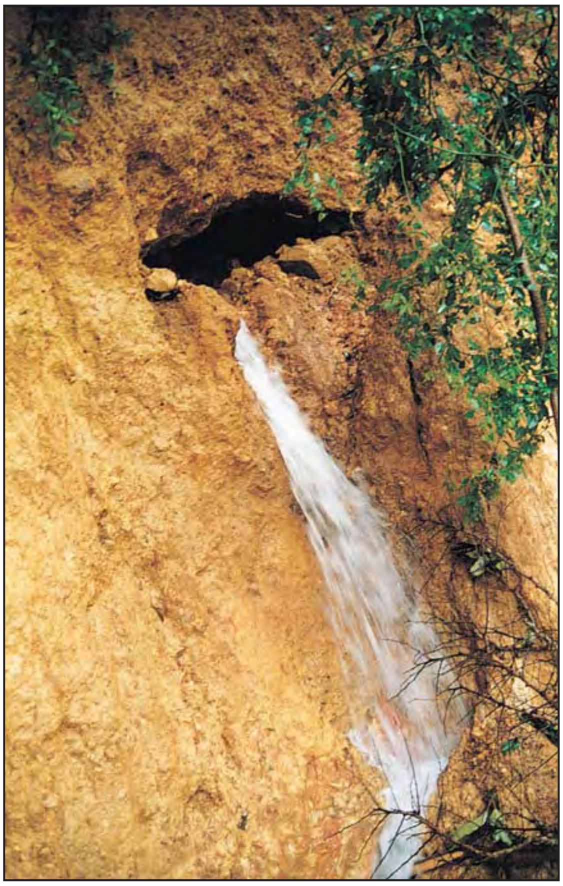


Figure 4.4.10 – Soil pipe about 1 m across, near the colluvium/saprolite interface at Lai Ping Road Landslide (Koor & Campbell, 2005)

Development of soil pipes is an important hydrogeological process which influences hillslope drainage, eluviation and slope instability. A model for development of soil pipes is given in Nash & Dale (1984), and the key diagrams are shown in Figures 4.4.11 and 4.4.12. Soil pipes evolve progressively, leading to interconnection of voids and further erosion and expansion of the pipe. Although most reported examples have been in connection with the investigation of shallow landslides, generally involving less than 2 m thickness of regolith, a pipe network at the base of saprolite more than 10 m thick was described by Whiteside (1996), and infilled pipes were identified at greater depth in granite saprolite in northeast Kowloon (HAPL, 1998b). It is estimated that 95% of all groundwater flow in

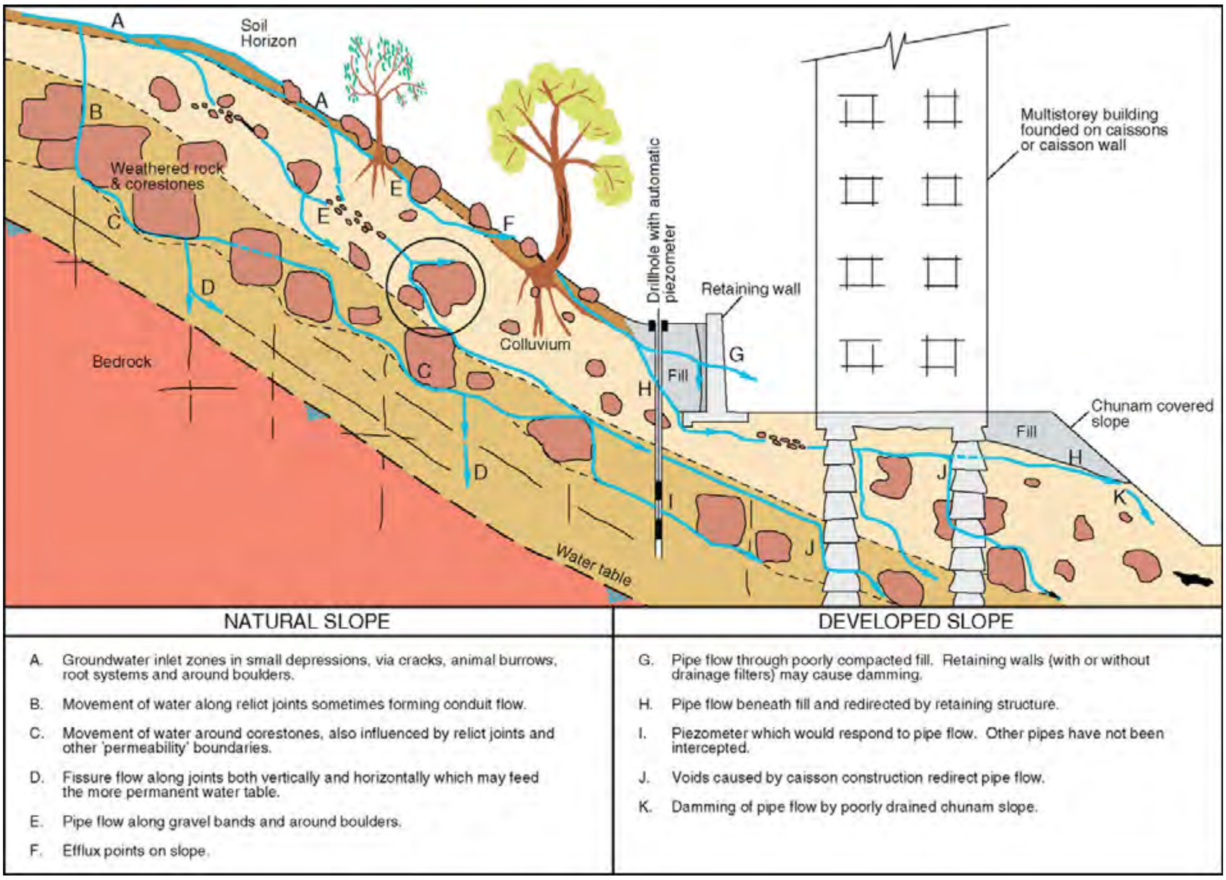


Figure 4.4.11 – Soil pipe development and its effects on groundwater regimes (Nash & Dale, 1984)

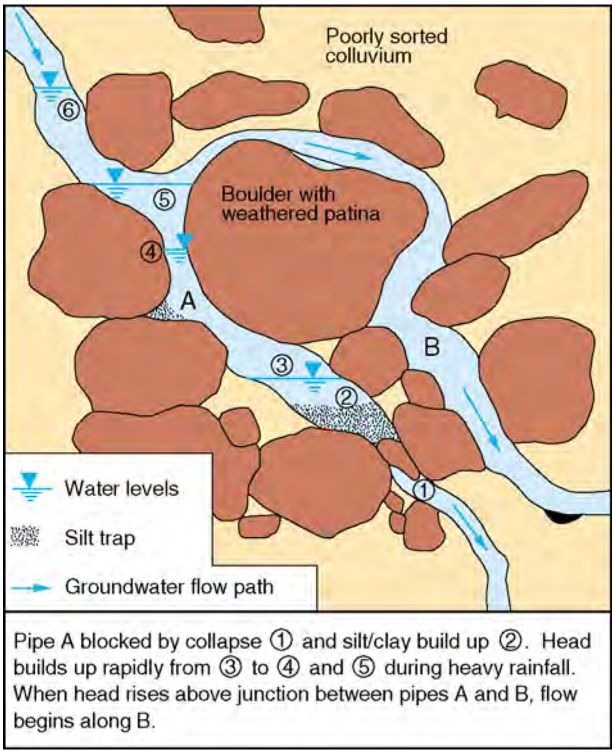


Figure 4.4.12 – Schematic representation of re-direction of pipe flow in a blocked system (Nash & Dale, 1984)

the granitic terrain of Japan is through such pipes (Ziemer & Albright, 1987).

Soil pipes commonly occur at permeability boundaries within the regolith (Figure 4.4.11). Shallow pipes may coalesce and feed into deeper-seated, more substantial pipes in saprolite (Nash & Dale, 1984). These are influenced in their development by relict joints. Soil pipes are also commonly associated with landslides and areas of ground deformation, e.g. forming along their flanks, where they occur within tension cracks and the displaced mass in general (Figure 4.4.13).

Where they are free draining, soil pipes can provide efficient drainage in the slope. However, soil pipes may be closed, or become blocked, or have their capacity exceeded during periods of extreme rainfall. In such circumstances, they have been interpreted at several locations as having caused high water pressures to develop, contributing to slope failure (e.g. Nash & Dale, 1984; HAPL, 1998b; Whiteside, 1996; Koor & Campbell, 2005). Sediment is often seen on the floors of soil pipes, and clay may encrust the sides and roof. In some cases open joints allow groundwater

Figure 4.4.13 – Voids/soil pipes at the colluvium/bedrock interface in a landslide scar

flow to carry coarser material (Figure 4.4.14). Many examples of infills of laminated sand were identified in granitic saprolite at a landslide site in northeast Kowloon (HAPL, 1998b). Erosion within shallow pipes may eventually lead to collapse of the pipes and to the initiation of gully erosion. A model of pipe erosion has been used to explain shallow landslides and surface collapses in a large volcanic saprolite slope at Pun Shan Tsuen (see Section 6.4). Large pipes have also been recorded where surface collapses occurred, for example at Yee King Road (see Section 6.4), and have resulted in difficulties when grouting soil nails (see Section 5.9).

Illuviated Kaolin

Landslide studies and research into the occurrence

Figure 4.4.14 – Bedded granular material infilling sheeting joint

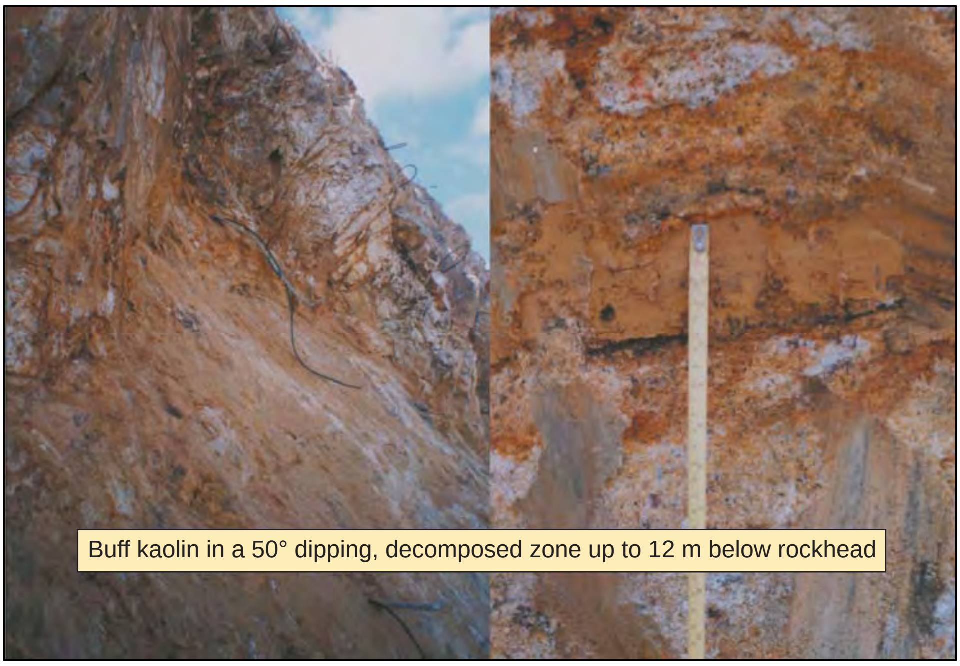


Figure 4.4.15 – Typical appearance of a thick kaolin infill in granite

and properties of kaolin-rich zones in Hong Kong suggest that much of the kaolin accumulation in discontinuities in Hong Kong is primarily a weathering product which has been transported by eluviation, mainly in solution, and deposited by illuviation. As such it is more likely to occur along discontinuities close to, and above the soil to rock interface or where there are other such similar hydrogeological boundary conditions causing preferential groundwater flow (Campbell & Parry, 2002). For example, these conditions can occur along the interface between extensive coreslabs and saprolite in plutonic rocks.

Kaolin infilling of discontinuities is relatively common especially within stress-relief joints. Clay infilling may be located along several sets of joints within the weathering profile depending on the variability of past groundwater regimes, weathering and joint development history. Some examples of failures along kaolin-infilled discontinuities and

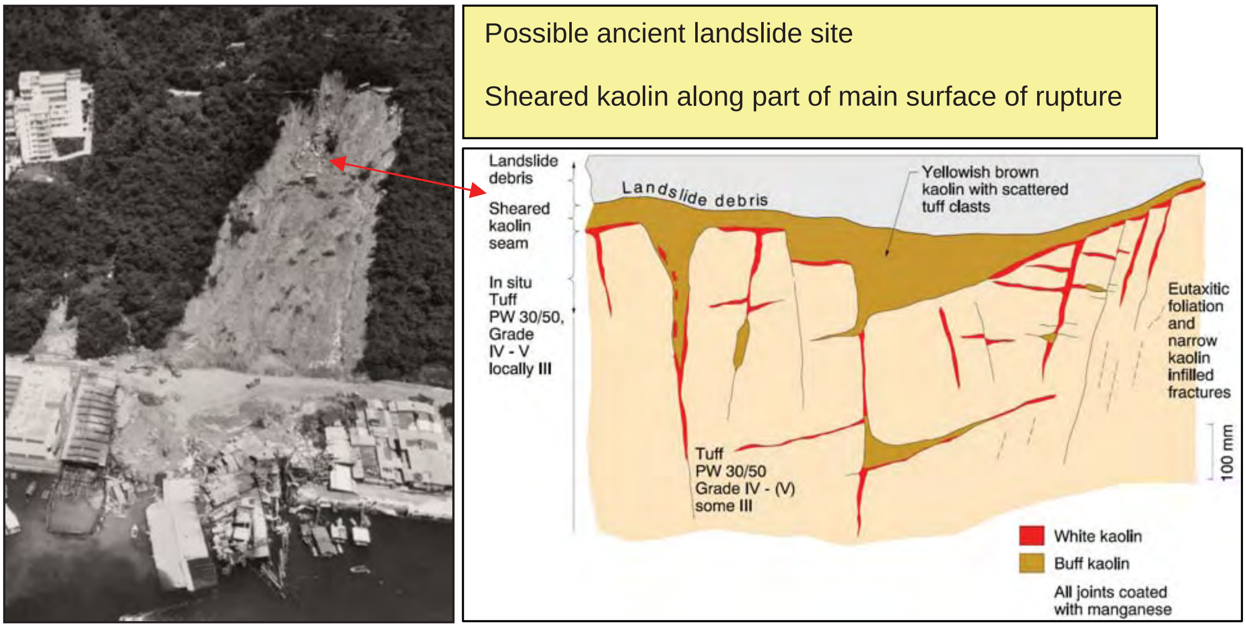


Figure 4.4.16 – August 1995 Shum Wan Road Landslide (GEO, 1996b and Kirk et al., 1997)

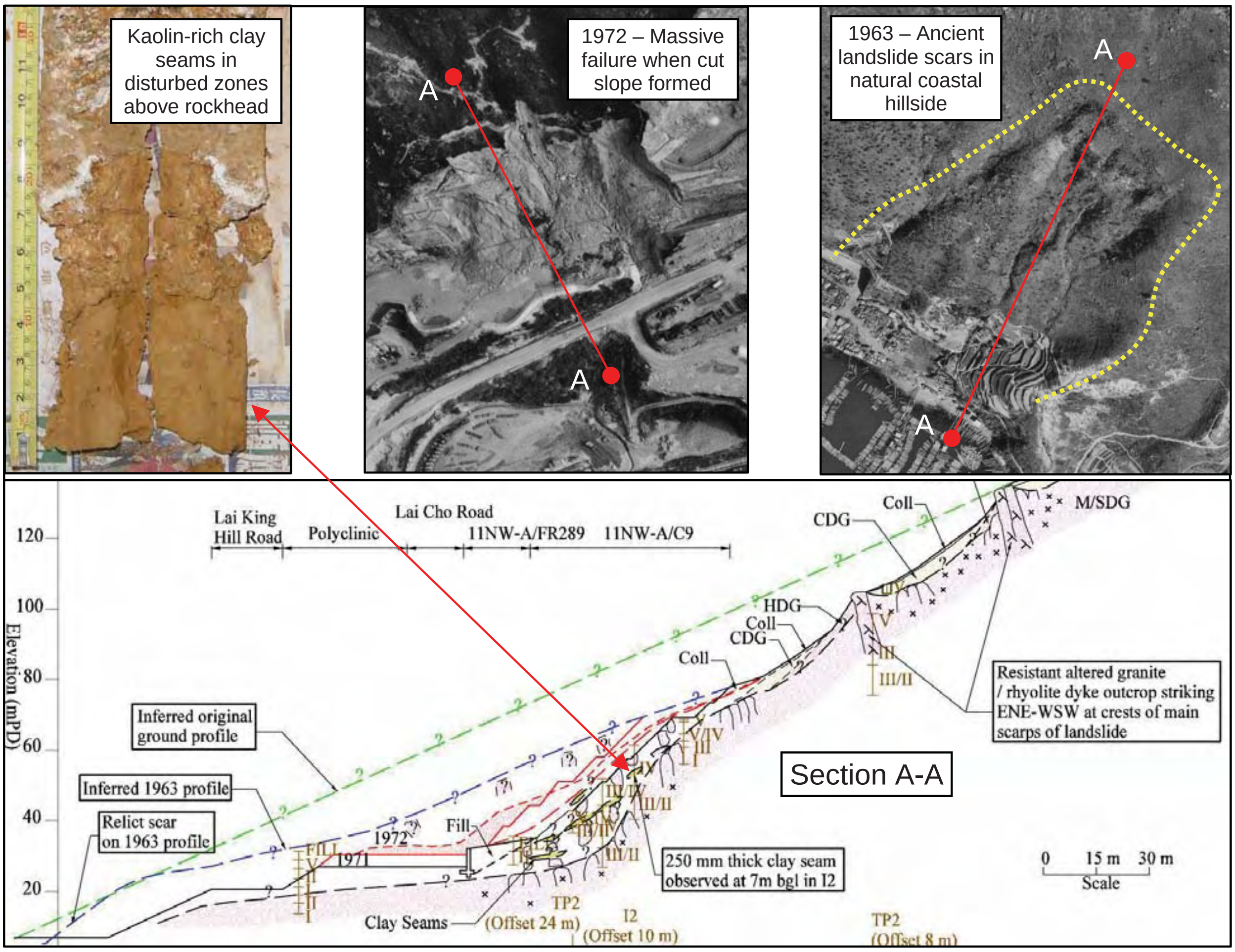


Figure 4.4.17 – 1972 Lai Cho Road Landslide (MGSL, 2002 and Thorn et al., 2003)

their engineering implications are given in Campbell & Parry (2002).

Kaolin infills have also been recorded within shear zones and dilated discontinuities associated with past landsliding. These infills are commonly light buff to dark brown in colour, due to the inclusion and weathering of rock fragments (Campbell & Parry, 2002). Because the kaolin can be difficult to recover and identify using standard methods of drillhole investigation, understanding its genesis and potential distribution is key to the identification of kaolin-rich zones at the site investigation and design stages.

Figure 4.4.15 shows a thick kaolin zone in granite, where the buff colour suggests movement during its formation, while Figures 4.4.16 and 4.4.17 show prominent landslide sites involving kaolin-rich seams. The 1995 Shum Wan Road landslide (GEO 1996c,d; Kirk *et al.*, 1997) and the 1972 Lai Cho Road landslide (MGSL, 2002; Thorn *et al.*, 2003) are coastal sites with evidence of ancient landslides.

Examples of other prominent landslides involving kaolin-rich infills such as the 1995 Fei Tsui Road (GEO 1996a,b) and 1999 Shek Kip Mei (FMSW 2000) landslides are given in Section 6.4.

4.5 GEOMORPHOLOGICAL PROCESSES

4.5.1 Introduction

Geomorphological processes encompass all forms of surface erosion and deposition including colluvial, fluvial and coastal processes. These processes have shaped the present-day topography and are of fundamental importance in understanding the engineering geological characteristics of the Hong Kong landscape.

Engineering geological issues include:

- identifying the various processes currently active and those which have affected the terrain in the past, and

- assessing if the results of these processes could affect the engineering project in question.

4.5.2 Geomorphology

Anon (1982) notes that there are two main types of geomorphological approach to landscape evaluation for engineering purposes: (a) land classification involving identification of landscape patterns, and (b) land surface (geomorphological) mapping involving demarcation of small areas of similar terrain, the nature and properties of their materials, and the characteristics of the processes currently active on the land surface. Although each approach can complement the other, the land surface mapping approach is the most suitable to facilitate the development of geological models given the scale required for most applications in Hong Kong.

The Geotechnical Areas Study Programme (GASP) carried out in the 1980s used a land classification approach (Styles & Hansen, 1989) based on API. The associated maps were designed for use at a scale of 1:20,000. In addition, 13 GAS (Geotechnical Area Studies) at 1:2,500-scale were carried out for areas considered to have extensive bodies of colluvium (Styles & Hansen, 1989).

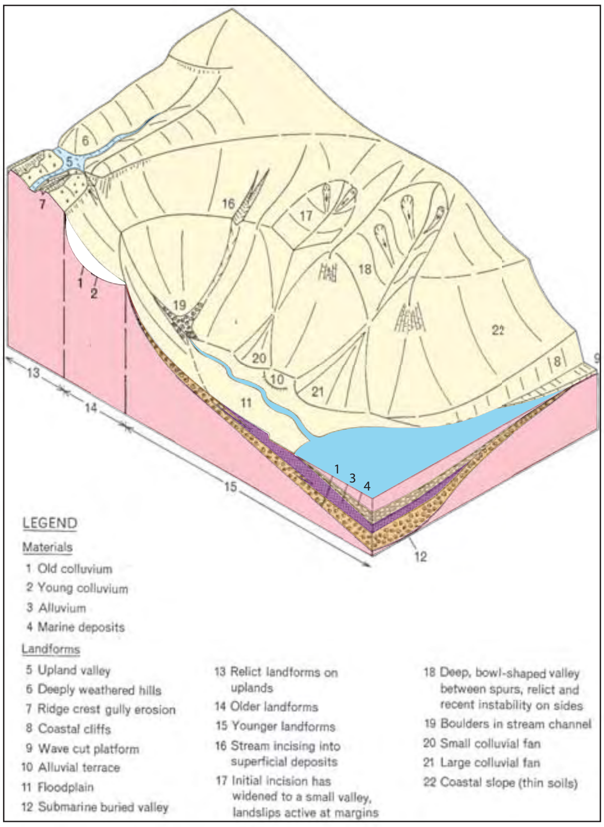


Figure 4.5.1 – Geological model for landscape evolution in Hong Kong (Hansen, 1984)

Based on the GASP work, Hansen (1984) presented a geological model for landscape evolution in Hong Kong based on geomorphological principles. He proposed a simple two-form model with an older and younger landform assembly (Figure 4.5.1). The upper, 'older' assembly contains deep weathering profiles and the oldest colluvial sediments. The 'younger' assembly is a product of stream rejuvenation as a consequence of Pleistocene sea level fall. Both assemblages are subject to different types and rates of processes, with the greatest potential for erosion at the boundary between the two.

Geomorphological mapping places the site and its surroundings in a hierarchical framework that integrates morphology (form), process, materials and age (GEO, 2004g). As such it helps the practitioner to interpret the influence of lithology, structure, materials and processes on past and current landform development, thus allowing the formulation of geological models to predict future behaviour.

Morphology

The morphology or shape of a hillslope arises from the interaction of hillslope processes, mass and materials over time. Slopes commonly exhibit overall an upper area of shallow slope gradients, a mid-slope area of steeper gradients that is predominantly erosional and a lower area of shallow gradients that is predominantly depositional (Figure 4.5.2). This simple pattern varies greatly within a particular site. Such variations are not random features in the landscape but generally reflect features such as lithological contacts, faults or shear zones, with associated contrasts in material strength and weathering characteristics. They may also represent landform assemblages of different ages and provide evidence of previous instabilities.

Process

It is often useful to classify the terrain in accordance with active process. These processes include:

- runoff and surface erosion,
- net sediment transportation and mass movement,
- fluvial, including debris flow development, and
- deposition.

Several different geomorphological processes may be active within the same terrain. For example, transitional environments such as hillsides of moderate gradient and medium to low energy fluvial valleys may, at varying times, be affected by erosion, transport or deposition in response to

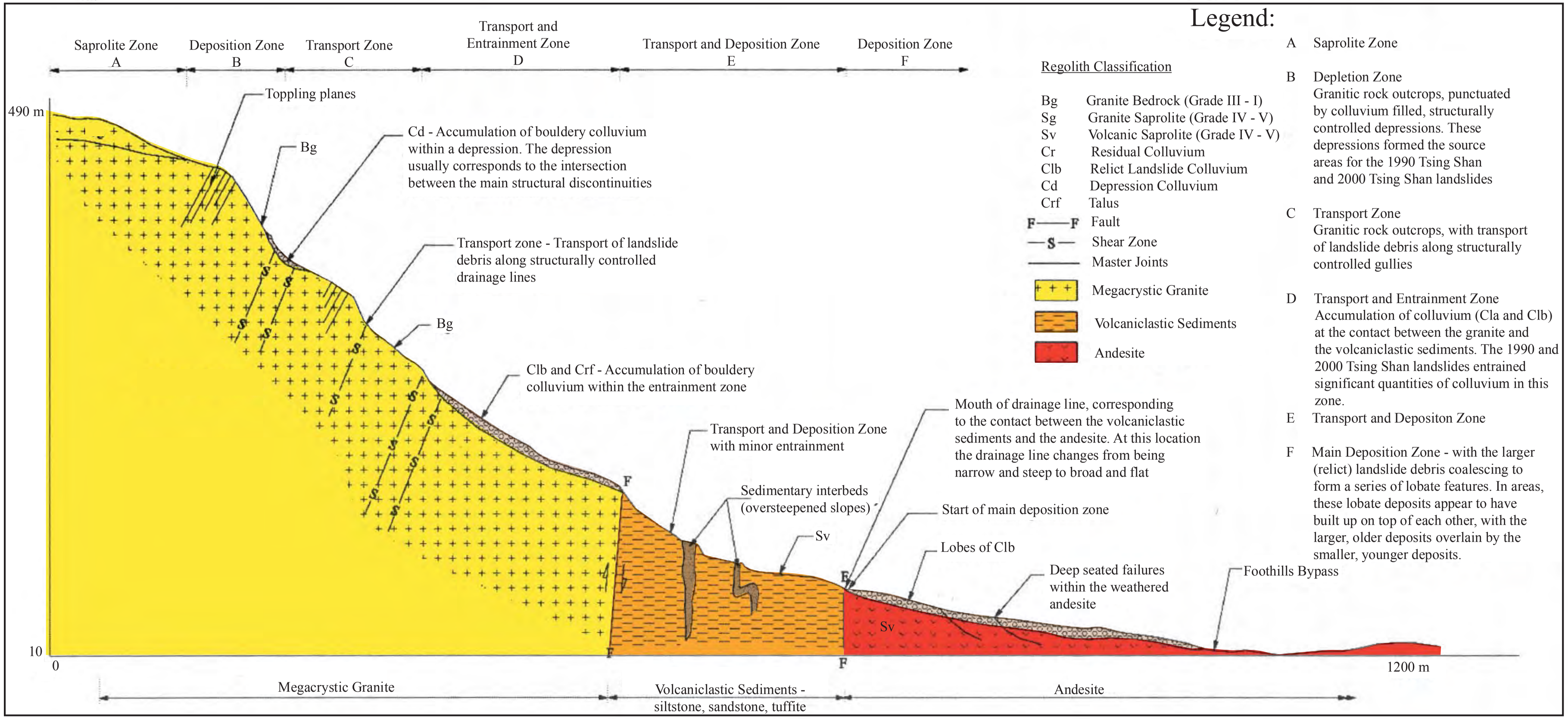


Figure 4.5.2 – Geological model of the eastern flank of Tsing Shan showing the interaction of geology, topography and geomorphology on landslide initiation, transport and deposition processes (Fletcher et al., 2002)

rainfall, landslides, flooding or drought. An example of the interaction of topography, geology, and geomorphology on landslide initiation, transport and deposition is shown schematically in Figure 4.5.2. Such models can be used for understanding the dynamics of evolution of the local terrain and associated geological hazards.

The recognition of active processes related to progressive deterioration, such as weathering, changes in the hydrogeological regime and slope movements, can be facilitated by geomorphological mapping. Progressive deterioration can lead to increased water ingress and modify subsurface water flow conditions in soil pipes and joints, thus changing the potential for hazard on a local scale.

Materials

Geomorphological processes are generally restricted to the regolith, and exposed rock masses. Regolith comprises superficial deposits and saprolite. Mapping of the regolith sub-divides saprolite and transported superficial material according to their different properties and behaviour (Figure 3.2.3). Site-specific classes of regolith have been developed for some individual studies (MFJV, 2002b).

Mapping the regolith can be a useful interpretative tool as information can be obtained about several other terrain characteristics such as relative age, morphology and processes. However, for some sites, there may not be sufficient contrast or diversity in the regolith units mapped for these to be of key importance in susceptibility analysis (OAP, 2004a). Consideration of potential differences in the geotechnical behaviour of the slope-forming materials in different hillside settings can help to increase the usefulness of regolith mapping (see Section 6.2.4).

In addition to regolith, the possible influence of lithology and structure should be considered in geomorphological mapping. Site evaluation at larger scales may reveal site-specific lithological or structural factors that influence both the geomorphology and the hazards, e.g. Fletcher *et al.* (2002).

Age

The evaluation of relative age of units of terrain and previous instability of natural terrain allows a degree of understanding of the past and present activity of the site in terms of instability or mass wasting. Although geomorphological mapping allows only relative dating, a prediction of future activity is possible. While intense rainfall is the main trigger of natural terrain landslides in Hong Kong, longer duration environmental changes have an important influence on stability of terrain. Significant climatic variations during the Quaternary Period have influenced the rates of hillslope processes such as weathering, erosion and landsliding. In particular, episodic changes in sea level promote fluvial downcutting during low sea level stands, and deposition of unconsolidated material (potential for entrainment) on lower slope areas during high sea level stands. An understanding of such time-dependent variations in the landscape can help to interpret zones of relative hazard activity.

An example is the evolutionary model developed by Hansen (1984); see Figure 4.5.1. Although this regional model may not be directly applicable to every site, it is based on local experience and generally accepted geomorphological principles. The concept of upper and lower landform assemblages with relatively high rates of erosion and mass movement near the boundary between the two landform types

Figure 4.5.3 – The 1990 Tsing Shan debris flow

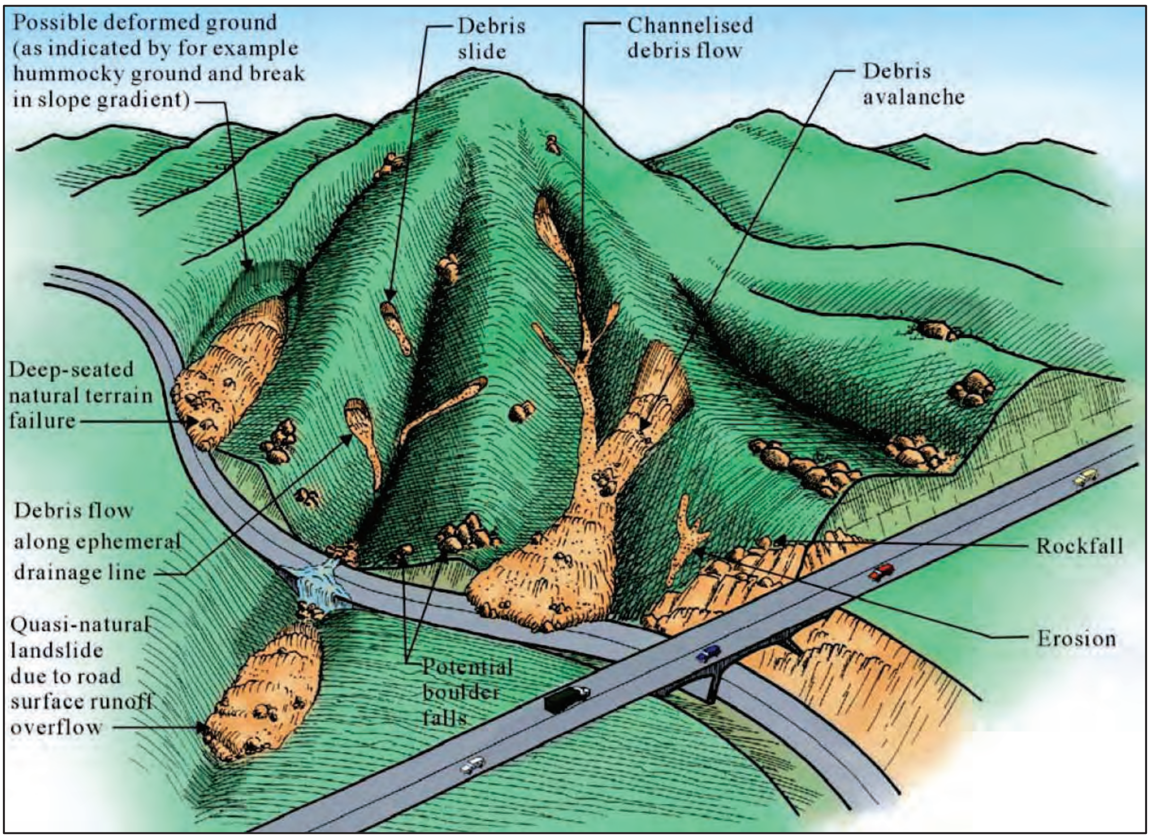


Figure 4.5.4 – Typical natural terrain hazards in Hong Kong (GEO, 2000)

has been shown to be useful in identifying areas with relatively high landslide intensity (see the Cloudy Hill example in Section 6.2).

Dating techniques can provide more precise absolute age information than geomorphological interpretation. Sewell & Campbell (2005) report on a suite of techniques for dating natural terrain landslides and rock surfaces in Hong Kong. Their results suggest that some large relict landslides are tens of thousands of years old, and therefore the landslides may have occurred under different environmental conditions than those pertaining in more recent times. Ages of events are important for magnitude and frequency analyses for hazard assessment (Section 6.2). However, the time-constraints of most engineering projects limit the use of dating techniques which often require considerable time to complete.

4.5.3 Mass Movement

Mass movements have played a significant part in forming the present-day landscape of Hong Kong, including the formation of extensive colluvial deposits which can reach a thickness of about 25 m in the Mid-levels area of Hong Kong Island. Figure 4.5.3 shows large-scale mass movement and Figure 4.5.4 shows a schematic representation of typical natural terrain hazards.

Types of mass movement include:

- deep-seated landslides, commonly associated with thick or deeply weathered, weak regolith, and high groundwater levels,
- debris slides, avalanches and channelised debris

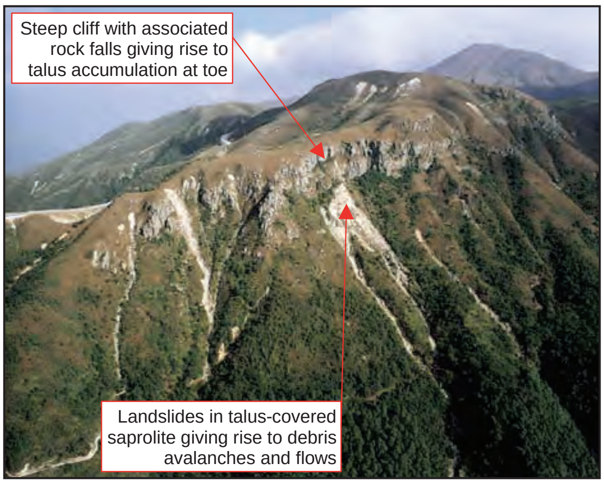


Figure 4.5.5 – Debris avalanches and debris flows below steep cliff near Ngong Ping, Lantau

flows, typically originating from over-steepened terrain,

- rockfall from cliffs (Figure 4.5.5) caused by dilation, toppling or sliding promoted by cleft-water pressure build-up or root wedging, and
- boulder falls caused by erosion of the saprolite or colluvium matrix with consequent undermining on steep slopes.

Evidence of mass movement such as degraded, amphitheatre-shaped depressions in hillsides and large colluvial lobes near the base of hillsides can be seen in many places in Hong Kong. However, in many cases debris may be absent or it may not be possible to link the debris present with the source area. Consequently considerable skill, and often detailed mapping, is required to determine whether such features are degraded large landslides or the result of the coalescence of a number of smaller landslides or erosional features. Even if the failures resulted from a single event, this may have occurred under very different environmental conditions from those pertaining in more recent times. The application of geomorphological knowledge and skills is therefore required to identify and interpret the potential implications of the occurrence of debris lobes and hillside depressions on potential landslide activity given the current geomorphological conditions.

4.5.4 Fluvial Processes

Surface erosion primarily takes place by gully erosion, sheet erosion and stream bank erosion (Fyfe *et al.*, 2000). Gully erosion is most active on saprolite in upland areas with sparse vegetation and is especially prevalent on well-drained granitic soils, particularly at breaks in slope. Gullies tend to coalesce and form dendritic patterns in granitic terrain and are relatively common in areas west of Tsing Shan and between Siu Lam and Tai Lam in the New Territories.

Most valleys are short in length, with a sharp change in gradient at the foot of the bordering hillsides. Low alluvial terraces are formed of generally well-sorted sand and gravel deposits which are laid down and re-worked as the alluvial channels meander. These deposits may inter-digitate with colluvium near the foot of the hillsides. Finer-grained lagoon deposits may also develop behind beach bars where the valleys drain into sheltered bays.

The Yuen Long floodplain is the most extensive area of flat-lying ground in Hong Kong and is formed from alluvial deposits overlying Holocene marine deposits and Pleistocene alluvium with buried channels. The main streams and rivers typically meander, but the development of fish ponds, flood protection works and fill platforms have largely arrested their natural migration.

4.5.5 Coastal and Offshore Processes

Much of the eastern and southern coastline of Hong Kong is exposed to the prevailing wind and waves, and is generally a high-energy erosive environment, characterised by the development of extensive sections of crenulated, rocky cliffs, with beaches and other depositional features being confined to the more sheltered bays. The western coastline is generally more sheltered, and depositional processes prevail. The influences of the sediment-laden Pearl River and the Yuen Long floodplain have given rise to the mudflats, mangroves and intricate tidal channels of Deep Bay. In the wet season, the effect of the Pearl River discharge is greatest. At this time, the discharge penetrates into western and central waters and can carry a high level of suspended sediment. Eastern waters are far less influenced by the Pearl River.

The depth of Hong Kong waters generally increases from northwest to southeast. The depth of water is generally less than ten metres in the northwest, near the Pearl River estuary. It becomes ten to twenty metres deep in the central harbour area, and it is about thirty metres deep in south-eastern waters. Tidal flows are the dominant influence in inshore areas with current speeds in excess of 2 m/s in constricted channels but 0.5 m/s or less in sheltered waters. The pattern of tidal currents is complex but the strongest currents and residual currents are in a generally southeast-northwest direction and this is reflected by two main tidal channel networks which are fifteen to twenty metres deeper than the surrounding sea bed.

The pattern of distribution of sea bed sediment reflects both the hydraulic conditions and the topography of the sea bed. The sea bed comprises mostly soft clayey silt with associated layers of suspended mud. Coarser sediments such as sands, gravels and cobbles occur close to the coast, islands, submarine rock outcrops and in constrained channels, where current speeds prevent sedimentation of fine material. This is a reflection of increased wave action in shallow water and increased currents around shoals, headlands and in channels (Fyfe *et al.*, 2000).

4.5.6 Influence of Quaternary Fluctuations in Sea Level

During the Quaternary Period, the lowest sea level was about 120 m to 130 m below the present sea level, and the shoreline was approximately 120 km south of the current position. This resulted in the formation of an extensive network of streams and rivers in which the predominantly fluvial materials of the Chek Lap Kok Formation were deposited (see Section 5.8). These include complex palaeochannel deposits of gravel and cobbles and local desiccation crusts in marine and estuarine deposits which have been mapped during investigations for reclamations such as Chek Lap Kok airport and Tseung Kwan O New Town (Fyfe *et al.*, 2000). Sub-aerial weathering during low sea levels has also led to the development of saprolite and karst solution features extending more than 100 m below the present sea level.

Approximately 6,000 years ago, the sea level was up to 2 m higher than at present, leading to the development of raised beaches and stranded sea cliffs which are now best preserved in the Tuen Mun valley and between Yuen Long and Lo Wu (Fyfe *et al.*, 2000).

4.6 HYDROGEOLOGICAL PROCESSES

4.6.1 Introduction

Hydrogeology is of major geotechnical importance in Hong Kong, with uncertainty regarding the groundwater regime often being a key issue in many types of engineering applications, such as slope stability, deep excavations and tunnels.

Engineering geological issues include:

- Heterogeneous and discontinuous geological materials with complex contrasts in permeability, particularly at the site-scale including:
  - o highly transmissive pathways such as soil pipes, open discontinuities and coarse-grained superficial deposits, and
  - o perching, confinement or damming of groundwater due to relatively impermeable barriers at interfaces between materials of contrasting permeability, e.g. lithological boundaries, decomposed dykes, faults and deep foundations such as diaphragm walls and rows of closely-spaced piles.
- Settlement of unconsolidated deposits (e.g. new reclamation) in response to groundwater abstraction or flow into deep foundation and tunnel excavations during construction.

Many of these issues are discussed in the context of the engineering geological characteristics of Hong Kong rocks and soils in Chapter 5 and specific engineering applications in Chapter 6. This section is primarily intended to provide an introduction to the main hydrogeological processes, and illustrations of hydrogeological complexity with particular reference to groundwater in slopes and tunnels.

4.6.2 Hydrogeological Environments

The main processes involved in the hydrological cycle are precipitation, evaporation, transpiration, surface flow, infiltration and groundwater flow. At very large or very small scales, where relative homogeneity is often assumed, the effect of these processes can be demonstrated using simple physical models. Figure 4.6.1 shows the material variability within the Chek Lap Kok Formation. On a large scale, the ground could be modelled for engineering design purposes as one material which has a representative

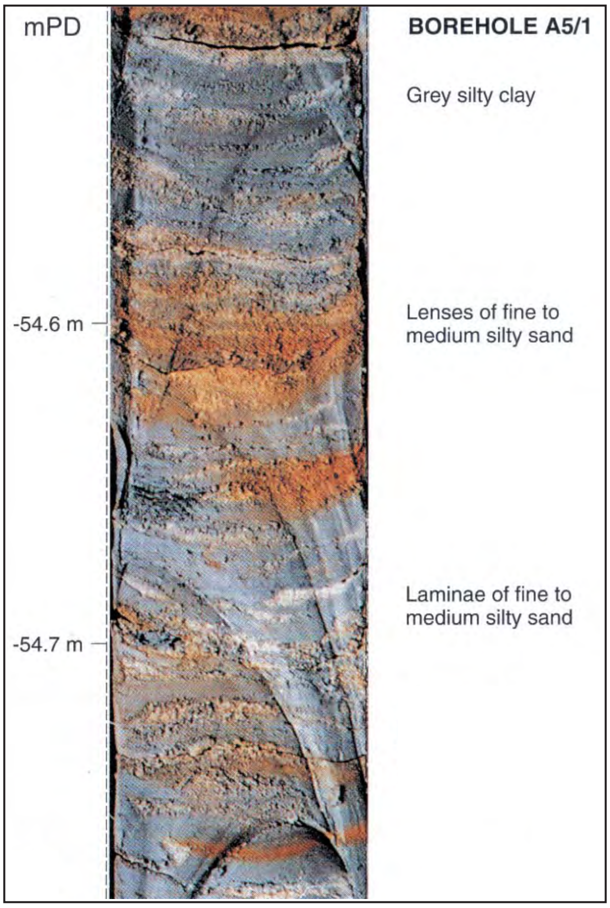


Figure 4.6.1 – Silty clay with sand laminae and lenses from the Chek Lap Kok Formation (Fyfe et al., 2000)

set of anisotropic compressibility and permeability characteristics, even though on a small scale the sample is composed of distinct layers of clay and sand with vastly different properties. Similarly, the numerical modelling for a major hillslope in the Mid-levels Study (GCO, 1982) considered all types of rock as a single aquifer. However, the discussion on the effects of tunnelling on the hydrogeological regime in Section 6.7 illustrates that zones of relatively high transmissivity can occur along fractured zones associated with faulting and stress-relief.

In many cases, a large amount of uncertainty may exist due to the heterogeneous nature of the ground, the impracticality of defining it in detail and potential future changes in environment. The variability of hydrogeological characteristics is primarily due to the geological origins and the subsequent effects of the processes described in Sections 4.2 to 4.5.

In addition, the groundwater regime is affected by environmental influences which include:

- long term variations in precipitation and infiltration due to changes in climate and vegetation cover,
- annual variations in precipitation,
- seasonal rainfall, frequently resulting in large and complex variations in transient groundwater levels in response to individual rainstorms, antecedent rainfall and overall seasonal variations,
- variations in infiltration due to construction, cultivation, hill-fires, bioturbation and opening-up of fissures due to desiccation, movement or stress-relief,
- coastal processes such as tides and wave action, and
- long term sea level variations.

4.6.3 General Hydrogeological Characteristics

Soils

Groundwater in soils is characteristically inter-granular. The hydraulic properties of soils range very widely. Open textured soils (e.g. some bouldery colluvium) have high permeability, large storage capacity and change little in volume when dewatered. By contrast, clays (e.g. marine clay) have low permeability and graded soils (e.g. dense saprolite) have low storage capacity. Soft clays compress when dewatered. Soils can be subjected to eluviation that can result in mass leaching or internal erosion leading to the creation of soil pipes. Generally, volcanic saprolite has a higher content of fines and is less permeable than granitic saprolite.

Conductivity of soil masses depends on connectivity. For example, fine grained alluvial soils deposited in broad expanses such as floodplain deposits can provide extensive strata of low permeability. In contrast, coarser alluvial soils found in stream beds can behave as channelised, sometimes interconnected or braided, extensive aquifers. Alternatively, confined aquifers can occur where lenses of coarse soils lie within less permeable materials.

Perched water tables can develop above aquicludes, and temporary perched water tables can develop above contacts with less permeable layers. Upward hydraulic gradients giving rise to sub-artesian or even artesian conditions can develop in some hillsides due to confinement, or partial confinement by a less permeable layer (see Figure 4.6.5). Examples of the influence of the hydrogeological properties of stratified sediments on groundwater modelling for deep excavations and tunnelling are discussed in Sections 6.6 and 6.7 respectively.

Rock

Generally, rock material is of low permeability and groundwater flow is dominated by discontinuities. Rock mass permeability may increase towards the soil to rock interface due to stress relief and correspondingly higher intensity of joints. Faults and dykes may be more transmissive in their plane, and act as aquitards normal to their plane. The influence of groundwater inflow on tunnelling and the extensive drawdown and settlement that this can cause in overlying soils are discussed in Sections 4.6.5 and 6.7.

Partially Weathered Rock Mass

Owing to the presence of relict geological structure, saprolite can have a combination of hydrogeological characteristics of soil and of rock, with added complexity due to the variations between the properties of the soil material and those of the relict joints and any soil pipes. This can lead to complex hydrogeological regimes. A schematic model showing typical hydrogeological processes in a cut slope above the interface between soil and rock is shown in Figure 4.6.2.

4.6.4 Groundwater in Slopes

A number of groundwater studies in Hong Kong (GCO, 1982; Insley & McNicholl, 1982; Li *et al.*, 1995; Evans & Lam, 2003a,b; MFJV, 2004) have demonstrated that the piezometric response time to individual rainstorms generally increases with depth,

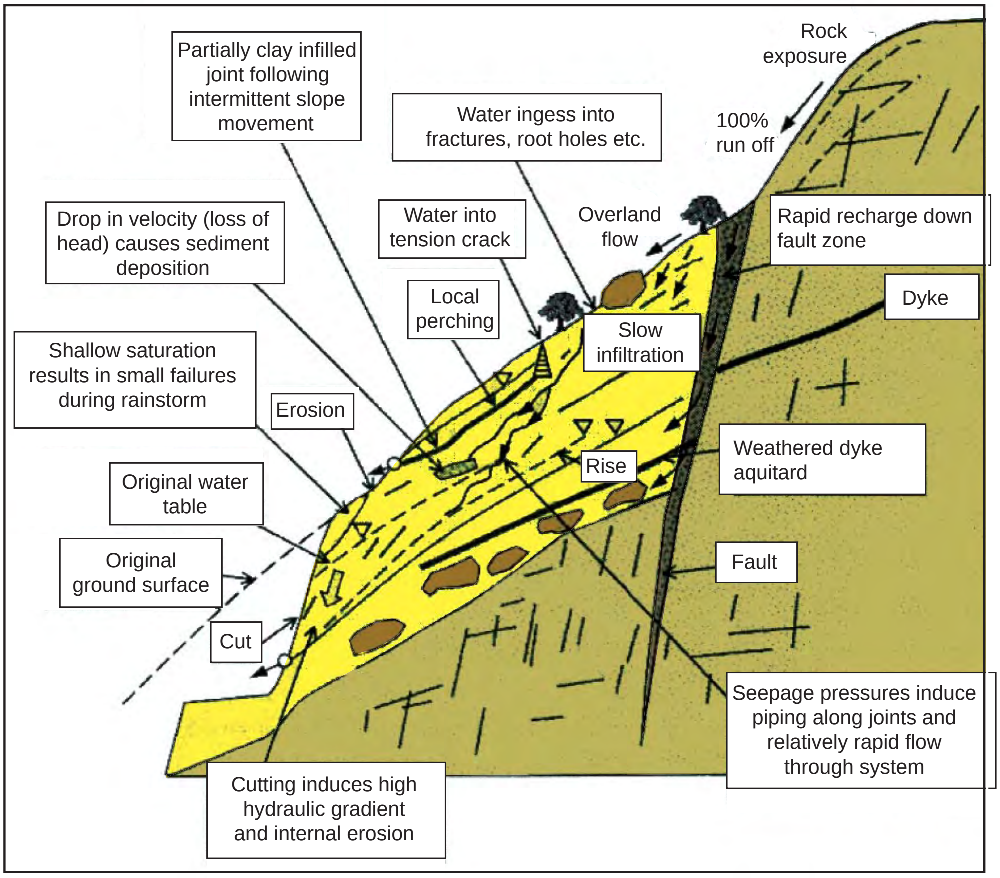


Figure 4.6.2 – Schematic hydrogeological processes above the rockhead in a cut slope (after Hencher, 2000)

with sharper responses of shallow perched water tables in colluvium or thin saprolite overlying shallow rock being common. Examples of slopes affected by groundwater and complex hydrogeological conditions are included in Section 6.4.

In the case of shallow , perched water tables, the response curve is usually asymmetric with a sharp response and slower rate of dissipation, though still relatively rapid. Examples of typical responses are shown in Figure 4.6.3 which depicts monitoring results of shallow piezometers installed in mainly superficial material. Similar responses are also

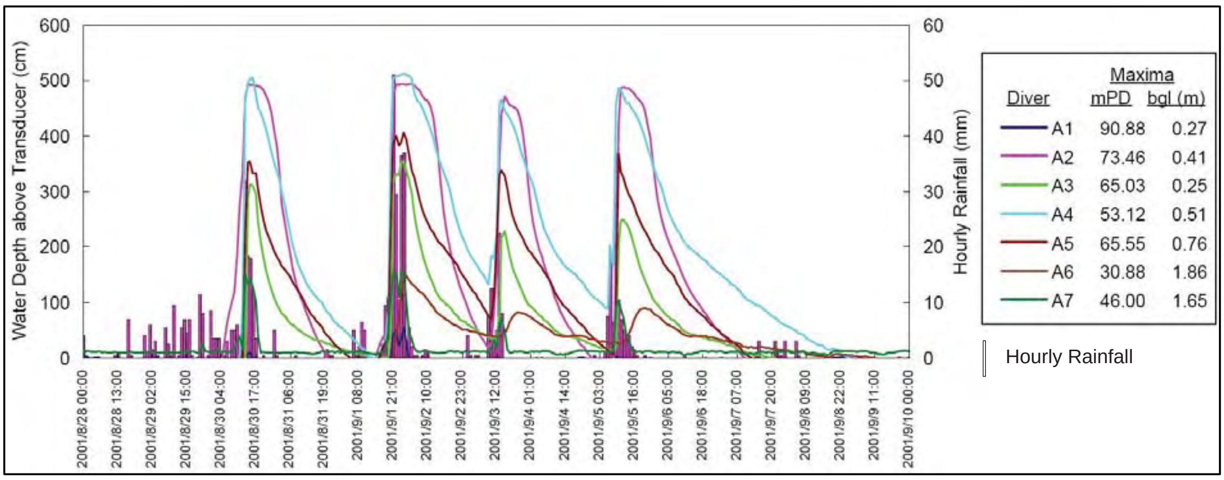


Figure 4.6.3 – Results of automatic groundwater monitoring at shallow depth in natural terrain (Evans & Lam, 2003b)

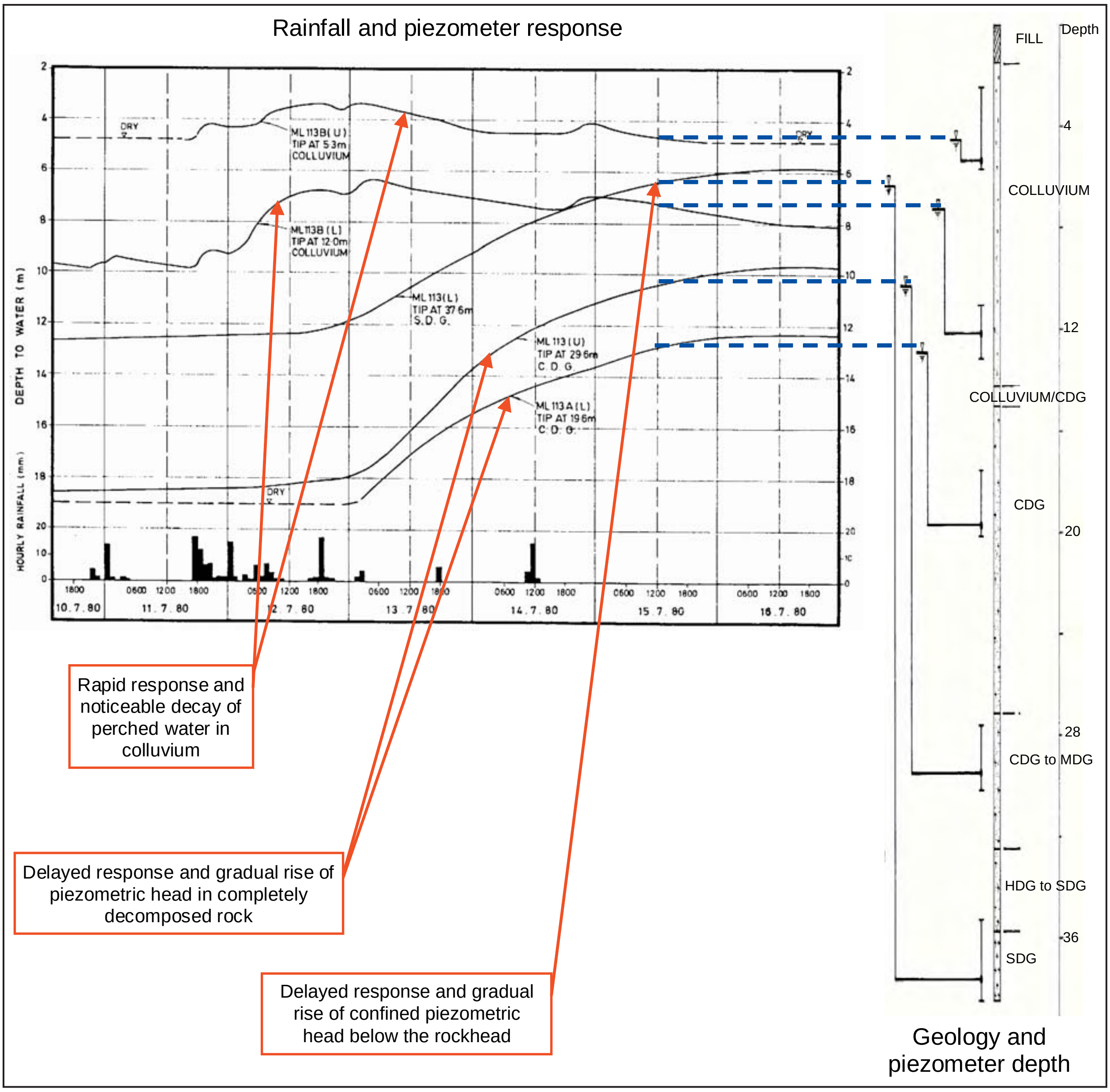


Figure 4.6.4 – Groundwater responses in colluvium and a weathered granite profile in the Mid-levels study area of Hong Kong Island (after GCO, 1982)

shown for two piezometers installed in colluvium in Figure 4.6.4.

Delayed but often large responses with relatively slow dissipation are commonly recorded in thick saprolite with increasing depth (Insley & McNicholl, 1982). In such cases, the base groundwater table may show a gradual rise throughout the wet season, with a less marked response to individual rainstorms. Examples of typical responses are shown in Figure 4.6.4 for two piezometers installed in granitic saprolite. Relatively rapid and large responses in thick weathering profiles and colluvium can also occur where a network of relatively open joints, fissures or soil pipes allow rapid infiltration and conduct flow towards zones of lower mass permeability (Sun & Campbell, 1999).

Rock mass is often regarded as being less permeable than saprolite, but this may not be always the case. Figure 4.6.4 shows evidence of a zone of more permeable rock close to the interface between soil and rock, with a higher piezometric head in the lowest piezometer being partially confined by less permeable, moderately to completely decomposed rock mass above it. The upwards hydraulic gradient implied by the lowest three piezometers has been interpreted as evidence of upwards flow from a partially confined aquifer in the rock mass

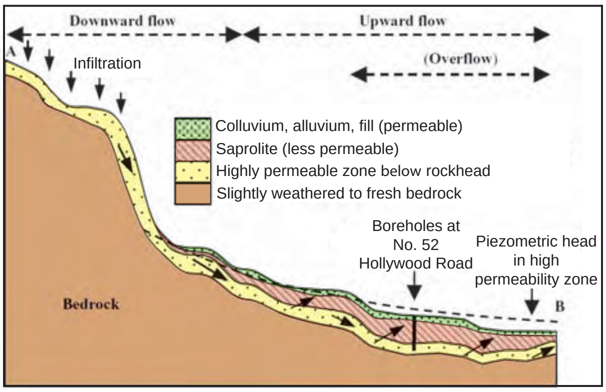


Figure 4.6.5 – Conceptual model of high permeability zone and confined groundwater below rockhead (after Jiao et al., 2003)

(GCO, 1982). Figure 4.6.5 shows a conceptual hydrogeological model of this environment which was developed to explain the occurrence of artesian water in drillholes at Hollywood Road. Jiao (2000a) has also raised the possibility of partially con fined groundwater having contributed to the delayed response and deep-seated failure of some lar ge cut slopes in Hong Kong (Figure 4.6.6).

An example of an investigation using automatic monitoring of piezometers in joints in a rock slope is given by Richards & Cowland (1986). A section through the slope and the groundwater responses for a number of rainstorms are shown in Figure 4.6.7. The monitoring showed a high variability in response times and magnitude to different rainstorms, with no single piezometer responding to all the rainstorms. The monitoring also showed that transient groundwater pressures were not observed to occur simultaneously over the whole surface of an individual stress-relief joint, and that the groundwater pressures were much less than predicted by typical empirical equations.

As noted above, the groundwater regime in saprolite can be complex, with primary porosity (soil material) and secondary porosity systems comprising networks of relict discontinuities, fissures and soil pipes. The secondary porosity may result in a transmissivity much higher than the primary system. Conversely , geological features such as clay-infilled relict discontinuities may result in lower permeability and lead to local perching or retardation of slope drainage. A schematic

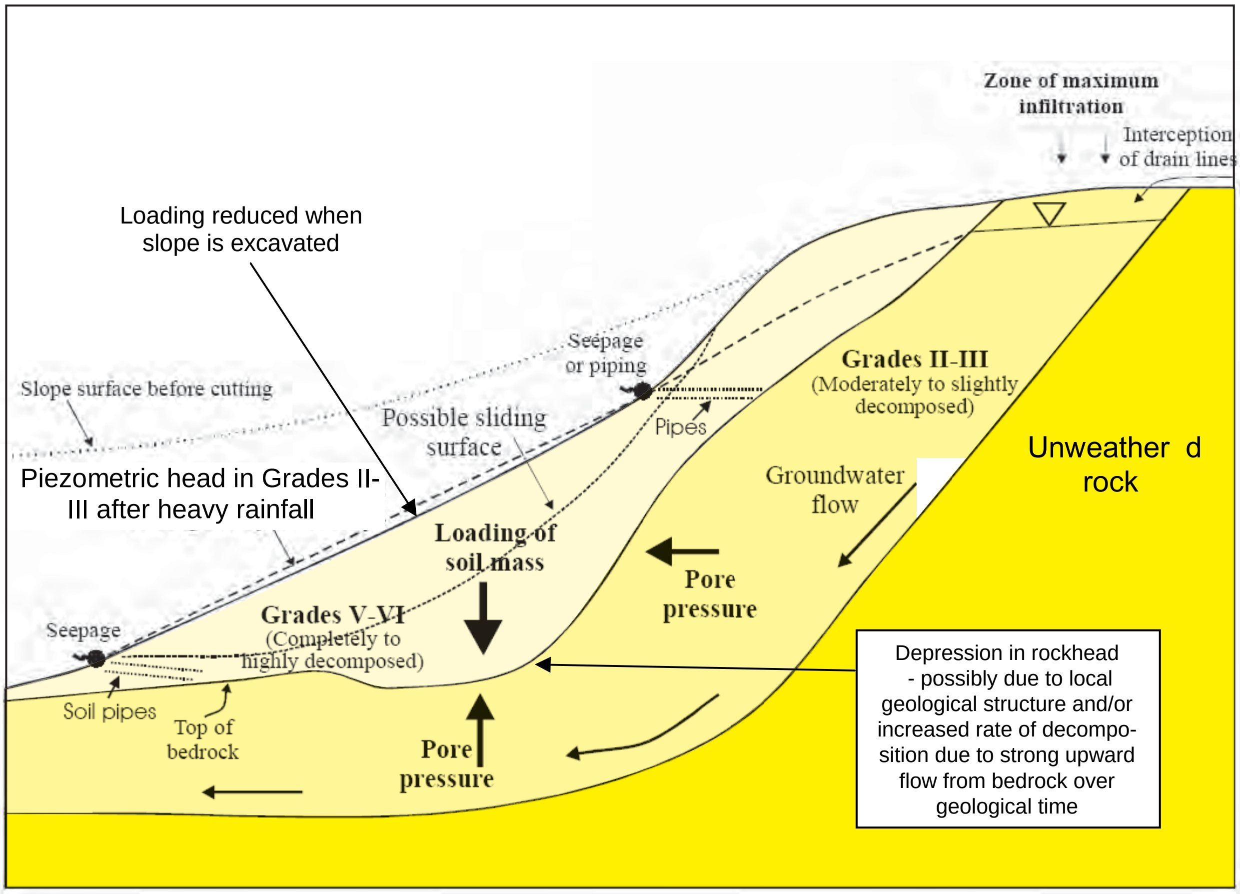


Figure 4.6.6 – Conceptual model of possible hydrogeological influence on deep-seated failure of some cut slopes in Hong Kong (after Jiao, 2000a)

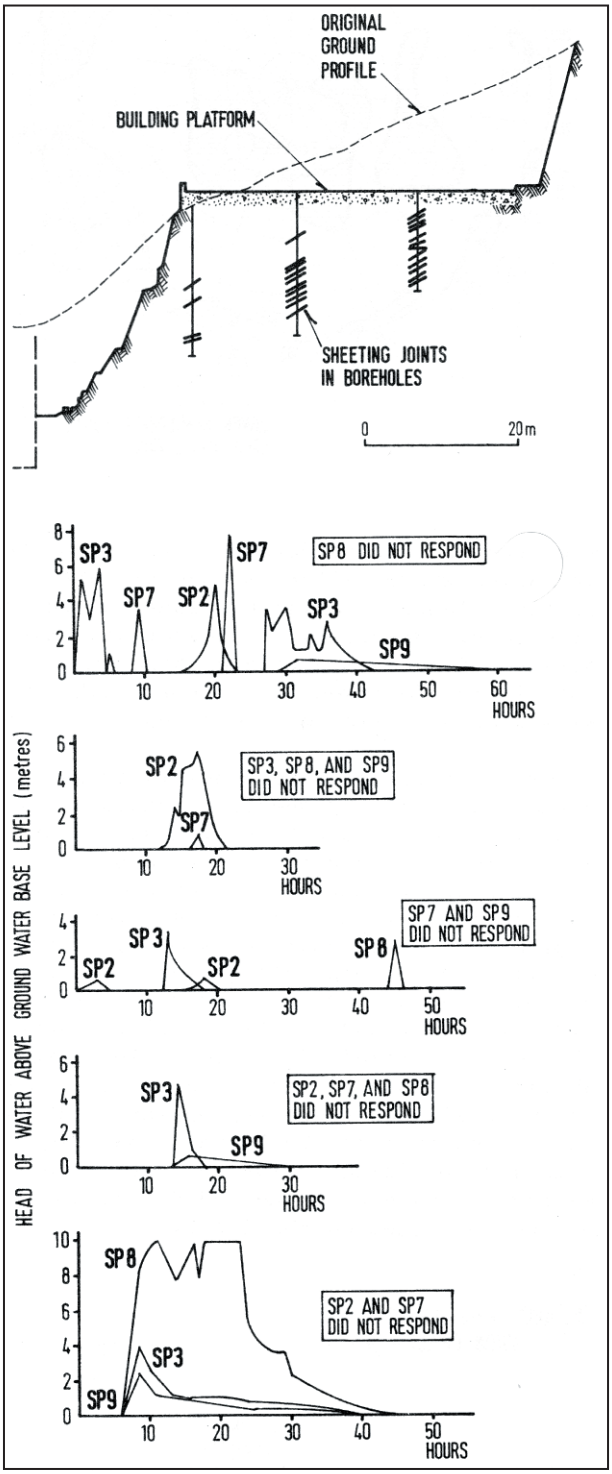


Figure 4.6.7 – Automatic piezometer monitoring of rock slope sheeting joints for different rainstorms (Richards & Cowland, 1986)

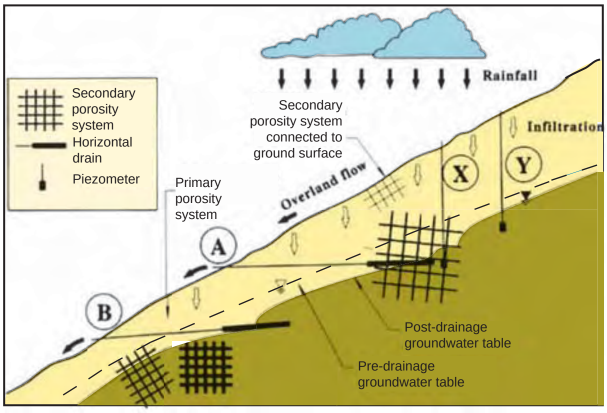


Figure 4.6.8 – Schematic model of primary and secondary porosity systems and groundwater compartmentalisation in a slope (Au, 1990)

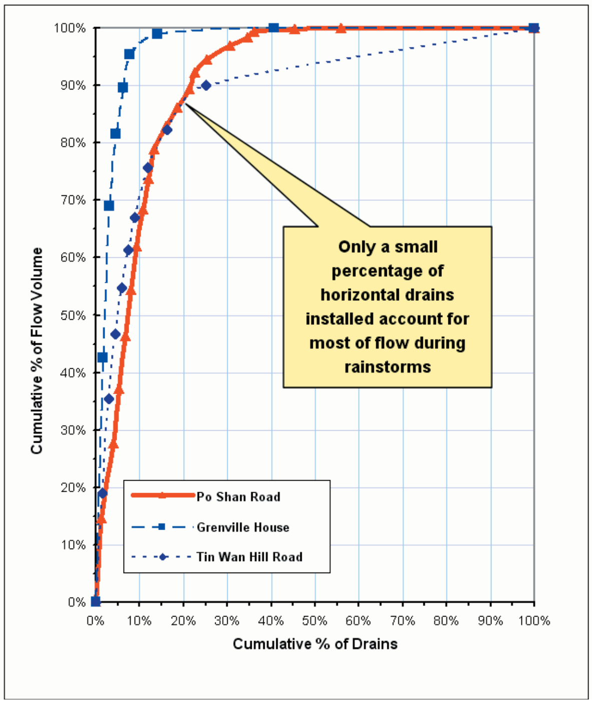


Figure 4.6.9 – Evidence of groundwater compartmentalisation from horizontal drain flow measurements (after Martin et al., 1995)

model of primary and secondary porosity systems developed by Au (1990) to explain dif ferences in responses of piezometers and of horizontal drains is shown in Figure 4.6.8. In this model, piezometer 'X' is likely to be more responsive to rainstorms than piezometer 'Y' because it has intersected a network of discontinuities. Horizontal drain 'A' is also likely to be more responsive than 'B' for the same reason.

Much of the evidence of preferential drainage paths in soil slopes in Hong Kong comes from flow measurements of horizontal drains during rainstorms (Martin *et al.*, 1995; Whiteside, 1996). Figure 4.6.9 shows the results of horizontal drain monitoring at three different locations. Only a small proportion of the installed drains accounts for most of the flow volume, indicating that only a few drains in each case intersected the more transmissive groundwater pathways. The relationship between geological structure and the drains with high flow rates at Grenville House is described by Kwong *et al.* (1988).

A staged observational method approach to the installation of batches of horizontal drains has been recommended (Au, 1990; Whiteside, 1996), whereas Martin & Siu (1996) stress the value of obtaining as much information as possible on hydrogeological conditions during the investigation stage, and note that an '*inquisitive approach*' to the understanding of the ground conditions during the investigation process has been of great value in finalising the layout of groundwater control measures during construction.

4.6.5 Groundwater Affected by Tunnelling

The key concerns about groundwater for tunnels are the ingress of groundwater during construction, the draw-down of groundwater outside the tunnels and any associated settlement of the ground. Ingress of water into tunnels can hamper or, in the case of large flows, even render tunnelling impossible. Groundwater draw-down and associated settlement can result in damage to property (Morton *et al.*, 1980).

Section 6.7.6 gives examples of ingress of water into tunnels and the effects of tunnelling on groundwater levels and settlement. These illustrate the concentration of groundwater ingress in relatively continuous and extensive, open-jointed zones associated with faults and dykes and the observation of draw-down as far as 2 km from tunnel construction. Experience has shown that prediction of zones of high groundwater inflows into tunnels in rock can be developed, based on a geological model of the rock structure in the vicinity of the tunnel and the identification of zones of poor rock along the alignment (MCAL, 2000). However, prediction of rates of ingress is not feasible within an order of magnitude due to the vast range of transmissivity of the ground and the variety of sources of recharge.

Draw-down of groundwater outside a tunnel can be modelled numerically given an adequate geological model including identification of transmissive pathways and characterisation of the aquifers (MCAL, 2000). Data from extensive monitoring of ingress of water into deep tunnels in rock, as illustrated in Section 6.7.6, can be used to calibrate and refine the ground model to anticipate the ground conditions and to assess the sensitivity of the design to variations.

4.6.6 Hydrogeological Uncertainty

Hydrogeological uncertainty can have major effects on the reliability of geotechnical designs and engineering performance both during and after construction. Sections 4.6.4 and 4.6.5 indicate that hydrogeological uncertainty can be reduced if models created during the investigation are used to target further investigations and if they are calibrated and updated during the design and construction stages.

Other measures to reduce hydrogeological uncertainty or counter its effects, where applicable, include:

- a representative period of groundwater monitoring before finalisation of the design,
- installation of automatic piezometers or 'Halcrow buckets' at appropriate locations and in representative hydrogeological units,
- monitoring of groundwater levels and seepage mapping during construction, and
- adoption of robust designs, including installation of prescriptive drains.

These measures should take into account the geological and ground models in three dimensions, and with respect to engineering time-scales to be fully effective.

5. ENGINEERING GEOLOGY OF HONG KONG ROCKS AND SOILS

5.1 INTRODUCTION

A detailed knowledge of the engineering geological characteristics of the rocks and soils in Hong Kong facilitates better prediction of site-specific ground conditions, improves appreciation of the potential range of ground behaviour and helps manage geotechnical risk with respect to specific engineering applications. This section considers the engineering geological characteristics of the rocks and soils in Hong Kong which, for the purposes of this document, are combined under the following broad groups:

- Plutonic Rocks
- Volcanic Rocks
- Dyke Rocks
- Marble and Marble-bearing Rocks
- Sedimentary Rocks
- Metamorphic Rocks
- Superficial Deposits
- Made Ground.

Detailed geological descriptions and background for each of the groups are given in Sewell *et al.* (2000) and Fyfe *et al.* (2000). In general terms, the mass characteristics of rocks and soils are controlled by the following engineering geological factors:

- geological history,
- intact material properties,
- properties of discontinuities, and
- groundwater within pores and discontinuities.

For each group, the key engineering geological issues are highlighted, together with material and mass characteristics and relevant cases histories to illustrate these characteristics. However, the broad nature of the lithological groups means that each group contains considerable variations. A thorough understanding of the geological environment of formation, and subsequent modifying processes such as faulting and weathering (Chapter 4), provides the necessary framework for the characterisation of the ground for specific engineering situations (Chapter 6).

5.2 PLUTONIC ROCKS

5.2.1 Introduction

Plutonic rocks originate from the crystallization of large intrusions of magma at depth (plutons) or from tabular sheet-like bodies of magma within faults or fissures at more shallow depth (dykes and sills). Although dykes and sills are also plutonic, given their smaller scale and differing engineering geological considerations, these are discussed separately in Section 5.4 (Dyke Rocks). In Hong Kong, the intruding magma was generally acidic (i.e. dominated by felsic minerals such as feldspar and quartz). The plutons cooled slowly, allowing the formation of distinctive interlocking crystal aggregates which generally result in 'very strong' rocks in the fresh state. The distribution of plutonic rocks in Hong Kong is shown in Figure 5.2.1.

For the purpose of describing their engineering

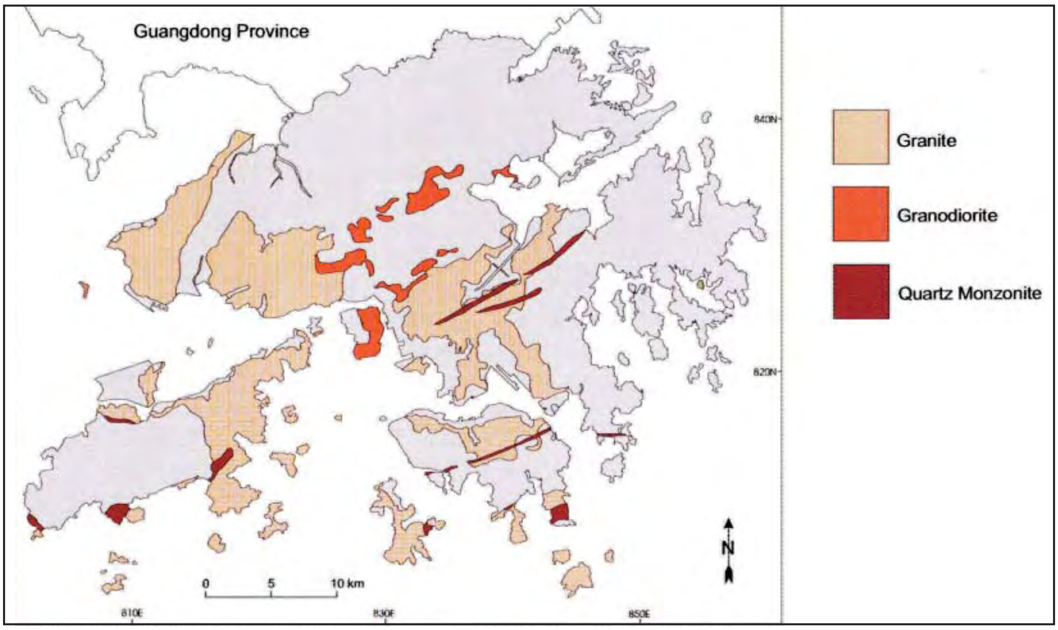


Figure 5.2.1 – Distribution of the plutonic rocks in Hong Kong (Sewell et al., 2000)

geological characteristics, the plutonic rocks can be divided into three main rock types based on their mineralogy (Figure 5.2.2). They collectively occupy about 30% of the land surface area of Hong Kong, but form about 80% of the developed area. The approximate percentages of each of the plutonic rock types in terms of the total area of land formed by plutonic rocks (Figure 5.2.1) are:

- Granite (80%)
- Granodiorite (15%)
- Quartz monzonite (5%).

Granite tends to form circular or ellipsoidal bodies many kilometres across. Granodiorite is more irregular, forming subvertical plutons and laterally persistent sills. Quartz monzonite typically forms smaller stocks and structurally controlled dykes which may be of the order of 50-100 m across.

In general terms, engineering geological considerations largely relate to:

- Nature of plutonic rock formation: relatively little post-formation deformation results in relatively uniform material characteristics over large areas (with the caveats listed below) and interpolation of drillhole information can generally be made with a reasonable degree of confidence.
- Weathering: this is the dominant process which controls the engineering characteristics of plutonic rocks. It is initiated at the surface and penetrates the rock via discontinuities. In the unweathered state, plutonic rocks are very strong to extremely strong and the mass characteristics are controlled by discontinuities.
- Discontinuities: faults, shears, tectonic joints and stress-relief joints weaken the rock mass and promote irregular weathering where groundwater penetration has occurred.

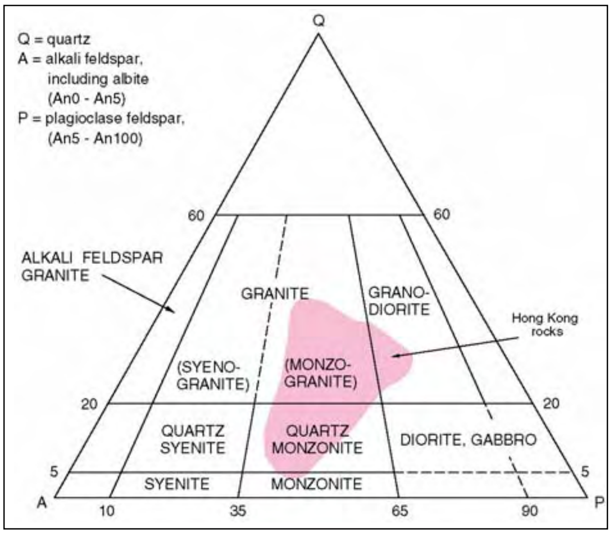


Figure 5.2.2 – Composition of the plutonic rocks (Sewell et al., 2000)

In specific terms, the key geological factors that may have an adverse influence on the engineering properties of plutonic rocks include:

- Contact margin and cooling effects
  - o heterogeneous and variable material properties (along irregular contact surfaces)
  - o local variations in material strength
  - o additional discontinuities, i.e. cooling joints
- Material weathering effects
  - o variations in material weathering effects (depth/ rate of chemical weathering and resulting soil properties vary according to mineralogy)
  - o disintegration (variations in material strength for same weathering grade)
- Mass weathering effects
  - o stress-relief joint development (potentially affecting slope stability)
  - o development of corestones, coreslabs and irregular weathering below rockhead (prevalent in coarse-grained rocks, potentially affecting foundations and tunnelling operations)

Table 5.2.1 summarises some material characteristics and properties of the three main plutonic rock types.

5.2.2 Engineering Geological Considerations

When magma is emplaced, the surrounding country rocks are partially displaced and partially assimilated into the magma. As a result, blocks of country rock (xenoliths) may occur resulting in irregular contacts and variable material properties.

Concentrations of residual fluids near the margins of cooling plutons can result in greisenisation (alteration, replacement and enrichment of the granite resulting in granular quartz and concentrations of mica and other minerals), and hydrothermal fluids may also penetrate and alter the plutonic and country rocks (e.g. chloritisation). These processes partially alter or replace existing minerals, typically resulting in a reduction in strength through the material fabric or along joint surfaces (Section 4.4). Associated with the final phases of cooling are pegmatites, which are very coarse-grained dykes or veins representing residual portions of the magma. In addition, fine-grained veins and dykes of granitic composition (aplite) also occur (see Section 6.7).

	Granite (Generic)	Granodiorite	Quartz Monzonite
Exposed area in Hong Kong (%)	24	5	1
Occurrence	Large circular or ellipsoidal plutons	Tabular sills/dykes or small plutons	Tabular sills
Typical Composition: (%)* Quartz Alkali Feldspar Plagioclase Feldspar Biotite mica/Hornblende	35 → Abundant quartz 25 } Equal feldspars 25 <10	30 → Abundant quartz 10 40 → Plagioclase dominant >10 → Relatively abundant	<20 → Little quartz 35 } Equal feldspars 35 <10
Texture and Grain Size General: Reasonably uniform texture, changes may occur especially close to the contact margins.	Crystalline, with interlocking crystal mosaic. Grain size very fine (aplite) to coarse (pegmatite).	Crystalline, with interlocking crystal mosaic. Generally coarse-grained.	Crystalline, with characteristic fabric anisotropy resulting from alignment of tabular alkali feldspar megacrysts.
Material Weathering Properties General: Weathering of granitic rock is generally deeper than volcanic rocks.	Variations in weathering due to mineralogy, grain size etc. Corestones may occur in saprolite depending on grain size and structure.	More susceptible than granite to weathering due to higher proportion of plagioclase feldspar which is less resistant to chemical weathering than alkali feldspar.	Most susceptible to weathering due to lower quartz content.
Aggregate and Roadstone Properties	Range (Acceptable Limit)		
ACV AIV LAAV TFV (10% Fines Value) (kN)	21-29 (<30) 15-31 (<45) 28-44 (27-44) 100-200 (>50 or>150**)	No specific data	18 12 21.5 No specific data
Suitability for Use in Construction	Good dimension, decorative and armour stone. Good aggregate when fresh. General fill for saprolite.	Good dimension, decorative and armour stone when fresh. General fill for saprolite.	May be used for aggregate (see text). General fill for saprolite.
Excavatability	Uniaxial compressive strength (UCS) can be over 250 MPa for fresh rock.	The strength properties for fresh to slightly decomposed rock are similar to granite.	
	Rock generally requires blasting. Saprolite can be excavated easily by machine although large corestones may require splitting.		

Notes: * Some % values are minima
** for heavy duty concrete use

ACV: The aggregate crushing value indicates the ability of an aggregate to resist crushing. The lower the figure the stronger the aggregate, i.e. the greater its ability to resist crushing (BSI,1990a).
AIV: The aggregate impact value indicates the strength value of an aggregate as determined by performing the aggregate impact test (BSI, 1990b).
LAAV: The Los Angeles abrasion value test is carried out to determine the susceptibility of an aggregate to abrasion (ASTM, 2003).
TFV: The ten percent fines value test determines the crushing force in kN at which 10% of the weight of aggregate is reduced to fine material (BSI, 1990c).

Table 5.2.1 – Summary of material characteristics and properties for plutonic rocks

Material weathering of plutonic rock is largely a result of chemical decomposition and this in turn is largely a function of the variable stability of the constituent minerals under the physical conditions to which they have been subjected to over time.

In the fresh state all plutonic rocks are competent, but as weathering increases, differences in material properties become more apparent. The development of fractures by mechanical disintegration can also reduce the strength of the material (Section 4.4).

Mass weathering is largely controlled by the discontinuity characteristics. Weathering along joints leads to an irregular rockhead profile. Plutonic rocks (especially granite) can develop extensive low-angle, undulating stress-relief joints which may be dilated and clay in filled (see Section 5.2.4). Weathering along sub-horizontal joints can result in seams of decomposed rock below the general rockhead and in the formation of coreslabs. Weathering on three or more joint sets can result in corestones within the saprolite matrix and in tors where the saprolite has been eroded. Corestones are more common in coarser grained and more widely jointed plutonic rocks.

5.2.3 Material Characteristics

Rock Composition, Texture and Fabric

Plutonic rocks are classified by their composition according to their relative percentages of alkali feldspar, plagioclase feldspar and quartz (Figure 5.2.2). Lesser amounts of other accessory minerals such as biotite and hornblende are also found in these rocks. Differences in the relative proportions of the main component minerals affect the rate of weathering and the properties of the weathered material. Table 5.2.1 shows these differences for the main plutonic rock types and associated material weathering implications.

The mineralogical, textural and fabric changes associated with weathering in some plutonic rocks of Hong Kong have been assessed by Irfan (1996a and 1996b). The key elements and description of the grades of weathering are summarised in Section 4.4 and illustrated in Figure 4.4.4. Visible fractures resulting from weathering appear to be more prevalent within plutonic rocks than in volcanic rocks and are found mostly in Grade IV and (to a lesser extent) Grade V material (Hencher & Martin, 1982).

Granite contains about 35% quartz and roughly equal amounts of plagioclase and alkali feldspar, with minor biotite (Table 5.2.1). This composition together with the interlocking crystal mosaic gives granite a high strength when fresh and a generally sandy soil when fully weathered. The grain size of plutonic rocks is generally uniform over large areas. However, the grain size can vary abruptly, especially near contact margins and this may affect the material properties. Additional fabrics can develop due to flow prior to cooling. This results in preferred orientations of mineral grains (schlieren).

Granodiorite also has a high strength when fresh, and contains about the same or slightly lower quartz content as granite but has a much higher proportion of plagioclase feldspar. This can result in a higher fines content upon weathering, relative to granite, as plagioclase is more susceptible to chemical weathering than alkali feldspar. Consequently, there is a more pronounced reduction in strength with decomposition, and a generally greater depth of weathering when compared to granite.

Quartz monzonite has significantly less quartz than granite or granodiorite (<20%) and is, therefore, the most susceptible to chemical weathering. Compared to completely decomposed granite, this may result in a material with a relatively high clay content, greater extent of penetrative weathering and with different material properties. These characteristics can have adverse engineering implications, e.g. at the Aberdeen Tunnel South Portal (Twist & Tonge, 1979). However, in the fresh state the rock is a competent material.

Material Properties

There is a significantly larger amount of numerical test data available for granite in comparison to granodiorite and quartz monzonite due to the greater surface exposure of granite within Hong Kong, especially in the urban areas. Plutonic rocks generally have very good material properties for engineering purposes, when in the fresh state. However, the degree, depth and rate of weathering of plutonic rock material is strongly influenced by the mineralogy as indicated above.

Plutonic rock masses can also be locally weakened at depth by hydrothermal alteration, especially near the margins of plutons and along fault zones (Section 4.3). This generally has more significance to deep foundations and tunnel excavations (Chapter 6), but also may affect slope stability.

Radon occurs naturally in many geological environments and is particularly associated with granitic rocks as a result of their relatively high uranium content (Sewell, 1999). Radon can be a radiation hazard if concentrations of the gas and its decay products exceed safety limits (Ball *et al.*, 1991). Further information and references relating to the occurrence and control of radon gas in tunnels and caverns are given in Section 6.7.

Rock Properties

Typically fresh granite, Grade I, is very strong to extremely strong, with uniaxial compressive strength (UCS) about 200 MPa. Grade II granite, slightly decomposed, is very strong, with UCS about 100 to 150 MPa. Grade III granite, moderately decomposed, is moderately weak to strong, with UCS about 10 to 80 MPa. Grade IV granite is weak when it is intact but is classified as a soil when highly fractured and composed of loosely interlocking fragments. However, within these decomposition grades wide variations in strength occur due to the gradational nature of rock decomposition and variability in the degree of microfracturing.

There are fewer test results readily available for granodiorite and quartz monzonite rocks. However, Irfan & Powell (1984) indicate a range of UCS between 150 MPa and 200 MPa for fresh Tai Po Granodiorite, and Irfan (1987) indicates a maximum UCS of about 300 MPa for quartz monzonite from Turret Hill quarry. These results indicate that in the fresh and slightly decomposed state the material strength properties of the plutonic rocks are similar (i.e. very strong to extremely strong).

Soil Properties

Saprolite is a soil derived from the *in situ* weathering of rock. It retains the relict structure and texture of the rock mass and typically comprises decomposition Grade V but may include Grade IV where it is disintegrated to gravel or sand. As a result of the breakdown of feldspars and the abundant more resistant quartz, granite saprolite typically forms a sandy silt when completely decomposed. However, variations do occur. Figure 4.4.4 in Section 4.4.3 summarises the chemical decomposition process in granitic rock. This demonstrates how variations in the proportions of the main component minerals affect the derived saprolite material. Saprolites derived from granodiorite and quartz monzonite tend to result in sandy clay and slightly sandy silty clay respectively. However, there can be considerable variability in density and strength, even within the same rock type and the same decomposition grade.

Table 4.4.2 (see Section 4.4.3) gives ranges of engineering properties within a completely decomposed granite profile at the 'strong' and 'weak' ends of the completely decomposed grade. There are significant variations in engineering properties due to differences in lithology, alteration, and moisture conditions at different sites. Therefore, representative engineering properties can only be obtained through site-specific investigations. However, the example serves to highlight that where a thick zone of saprolite occurs, it may be feasible to sub-divide it for the purposes of geotechnical characterisation, provided that the boundaries between the different zones can be reliably depicted in the geological model.

The plasticity of granite saprolite is usually in the low to intermediate range. However, saprolites derived from granodiorite, monzonite and altered granite may develop a higher plasticity, falling within the intermediate and occasionally high plasticity zones, due to mineralogical variations. Similarly, the permeability of saprolite soils will be affected by the mineralogy and the fines/sand content.

5.2.4 Mass Characteristics

The mass characteristics of plutonic rocks are largely dependent on the nature, persistence and density of discontinuities and the degree of weathering.

Discontinuities

General

Discontinuities allow inelastic deformation of the rock mass and can reduce mass strength by more than an order of magnitude, depending on confining stress (Hoek, 2004). The following discontinuity types are especially pertinent to plutonic rocks:

- Cooling joints
- Tectonic joints
- Stress-relief joints

In addition to their effect on the rock mass, these joints can result in variable and steeply sloping rockhead and allow weathering below general rockhead. Variable and steeply sloping rockhead can be problematic for piling (see Section 6.5), and weathering below rockhead can be problematic for tunnelling (see Section 6.7).

In general, plutonic rocks tend to have wider spaced discontinuities than volcanic rocks. On a site-specific scale, the plutonic rocks typically contain a low-angle joint set and two orthogonal joint sets that are normally steeply-inclined. However, additional sets are commonly present, which may be inclined in the range of about 30° to 90° (see Figure 4.2.6). These sets can be important for the stability of steep rock slopes, since they may form potential failure planes that are too steeply-inclined for joint roughness to provide adequate shearing resistance (see the Ting Kau Cutting case study in Section 6.4.4 for example). Different plutonic intrusions may have a different joint pattern to the adjoining plutons.

Cooling Joints

Due to the nature of plutonic rocks, primary discontinuities usually form during the late cooling stages of the magma. Four main types occur and are defined in terms of their relation to flow structures (flow lines sub-parallel to the edge of the pluton occurring during emplacement), namely cross-joints, longitudinal joints, diagonal joints and flat lying joints (Price, 1966). The identification of such features can be problematic, especially when flow lines are absent. However, veins and mineralization may be associated with these joints. The stress systems that formed these structures may influence the formation of later tectonic and stress-relief discontinuities. Gamon & Finn (1984) identified an additional steeply dipping cooling joint set within granite occurring close to (within 150 m) and parallel to a geological contact, during a major site formation.

Tectonic Joints

Tectonic structures including tectonic joints and faults reflect responses of the rock mass to changes in stress regimes due to tectonic processes. These processes are discussed in Section 4.2. Faults and fault-related joints can be very persistent and can promote deep weathering resulting in linear rockhead depressions.

Stress-relief Joints

Stress-relief joints form when a pluton is unloaded due to erosion. Moderately inclined stress-relief joints, sub-parallel to the ground surface are also commonly known as sheeting joints. Adjacent to steep terrain with relatively rapid rates of erosion such as active coastal settings or fault controlled valleys (or large man-made excavations), stress relief can also act laterally, thereby inducing formation and subsequent dilation of relatively steeply-dipping joints.

The spacing of stress-relief joints within plutonic rocks is variable. On a small scale, stress-relief and microfracturing can result in extremely closely spaced joints (Figure 4.4.1). Persistent stress-relief joints are typically widely spaced, but with increasing depth the spacing may increase to extremely widely-spaced as the influence of the stress-relief effects diminish. The persistence of stress-relief joints within the plutonic rocks has been recorded extending over hundreds of metres at some coastal sites.

The aperture of stress-relief joints also tends to decrease with depth due to a reduction in stress-relief effects. Observations made after the Lei Pui Street landslide (MGSL, 2004), which failed along a stress-relief joint, revealed apertures of over 100mm exposed along the flanks of the landslide source area. Drillholes at the same site also indicated that the stress-relief joint apertures were <10 mm below 10 m depth at this particular location.

At low stresses, given the wavy nature of stress-relief joints, the roughness angle of the discontinuity is important due to its effect of increasing the overall friction angle. However, extreme waviness can result in localised steep inclinations, which can induce failure, (e.g. at Hiu Ming Street – see Section 6.4.3).

Richards & Cowland (1982) report that the roughness angles of stress-relief joints at North Point varied from 16° to 6°. For design, two roughness angles were adopted: 16° where stress-relief joints were persistent and potentially affected the entire lower slope, and 8° for small potential failures affected by near surface joints. Richards & Cowland (1982) note that these are site-specific measurements which should not be adopted elsewhere without field verification.

Where shear box testing of a joint is undertaken the natural roughness of the surfaces should be taken into account by normalising the data to account for dilation during testing (Hencher & Richards, 1982). A basic friction angle of 40° has been proposed by Hencher & Richards (1982) regardless of the decomposition grades. The roughness of stress-relief joints and their effect on friction angle has been assessed at a number of landslide failures within granite rock masses including at Lei Pui Street (MGSL, 2004) and Leung King Estate (HCL, 2001).

Dilation of stress-relief joints can reduce rock-to-rock contact and thus shear strength. Water pressure build-up in near-surface stress-relief joints can also adversely affect stability. Failure along wavy surfaces characteristic of stress-relief joints requires considerable dilation of the rock mass. In some cases, incremental movement and dilation of the rock mass may occur due to transient build-up of water pressure without immediate failure occurring. Subsequent infilling of discontinuities may impair groundwater flow, increasing basal water pressures during heavy rainfall and thus promoting further downslope movement. The cycle is repeated until the system stabilises or failure occurs (Hencher, 2006).

Cowland & Richards (1985) discuss the contribution of persistent sheeting joints to rapid transient groundwater rise in the granite hillside above North Point. They also conclude that at this particular site transient groundwater rises did not occur simultaneously over the entire surface area of the joints, thus producing less severe groundwater conditions than normal assumptions of a rising groundwater table would indicate (see Section 4.6.4). This phenomenon may have been influenced by a combination of partial infilling, constrictions and free-draining pathways along the joint surfaces.

Case Study - Leung King Estate Landslide
Sheeting joints which were wavy, dilated and infilled were observed to form the surface of rupture of a landslide during the investigation of the natural terrain landslides above Leung King Estate in 2000 (HCL, 2001). The nature of the sheeting joints, block displacements and sediment infilling are shown in Figure 5.2.3. These observations indicate that progressive movements of the rock slabs above the joint surfaces had occurred. Theoretical back analysis of the failure indicated that the range of operative friction angle for the wavy sheeting joint was between 33 ° and 60 ° for a groundwater head range of between 0 m and 3 m.

Rock Mass Weathering
Rock mass weathering processes in granitic and volcanic rocks are described in Section 4.4. In comparison with the volcanic rocks, the depth of the weathered profile of the plutonic rocks is typically greater.

Material Uses
Fresh granite is a source of concrete aggregate and roadstone, decorative and armour stone and rock fill. Most aggregates used in Hong Kong are sourced from granite due to its abundance and its material properties which are within the acceptable limits for aggregates used in concrete and roadstone production (Table 5.2.1). These favourable material properties are related to the interlocking, crystalline nature of the fresh rock with abundant resistant quartz and feldspar. Some medium- and coarse-grained granites have an Aggregate Impact Value (AIV), Aggregate Crushing Value (ACV) or Abrasion Value on the higher side of acceptable limits, making them less desirable compared to fine-grained granites for

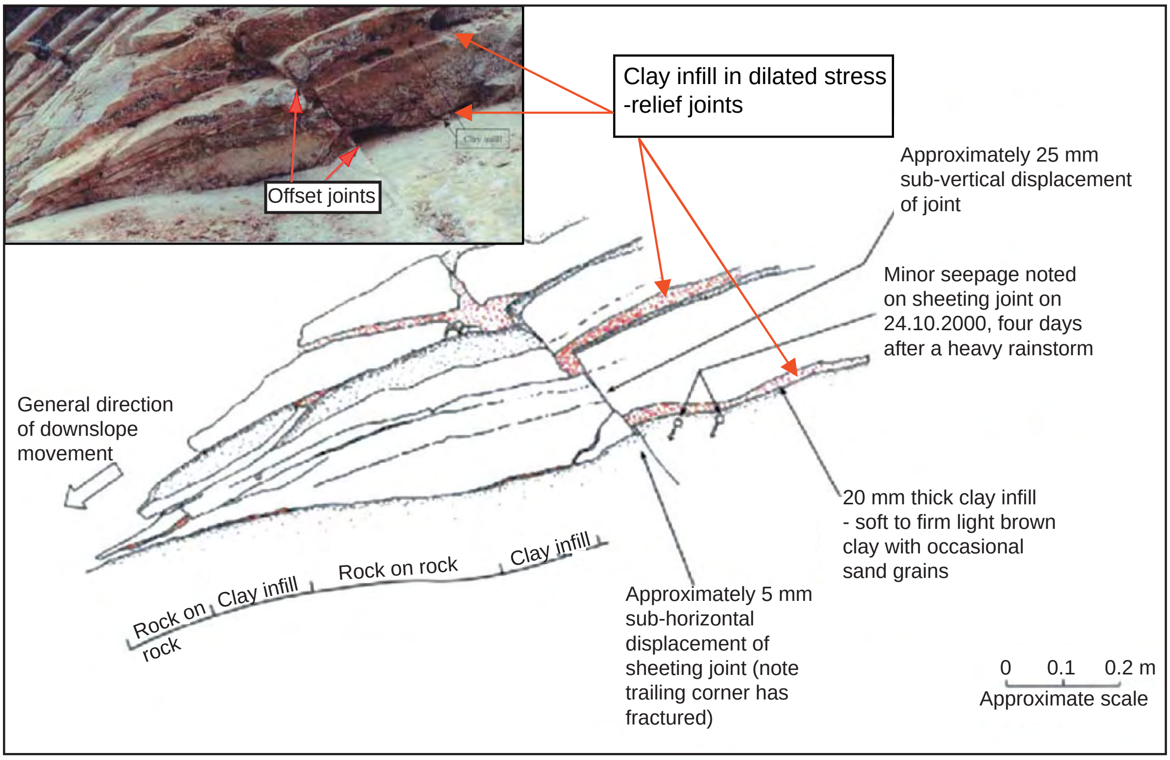


Figure 5.2.3 – Failure along wavy stress relief joint surface in Tsing Shan granite (after HCL, 2001)

wearing courses and some special uses such as heavy duty concrete floors. Low crushing strength may arise due to fracturing along the cleavage planes of coarse crystalline constituents. Problematic materials can also occur in shear zones, hydrothermally altered veins and aplite dykes.

Quartz monzonite when fresh has comparable mechanical and physical aggregate properties to granite and is generally within the acceptable limits for use in concrete and roadstone production (Irfan, 1987). However, quartz monzonite aggregates may have a lower resistance to abrasion due to lower free silica (quartz) content. Quartz monzonite also contains a higher percentage of feldspars compared to granites and is likely to be more susceptible to further decomposition and disintegration if the rock has already undergone some weathering. In addition, the alkali feldspars often display a preferred orientation, with less crystal inter growth or interlocking, which may affect the mechanical properties parallel to the preferred orientation of the crystal fabric.

Granodiorite has not been exploited as a source of aggregate in Hong Kong. Although aggregate test data are lacking, fresh granodiorite is probably suitable for concrete aggregate and roadstone. Fresh quartz monzonite and granodiorite are also suitable for armour stone and rock fill. Alkali aggregate reaction is generally not problematic except where the rock has been altered.

Saprolite from plutonic rocks is generally suitable as earth fill material.

5.3 VOLCANIC ROCKS

5.3.1 Introduction

During the Middle Jurassic to Early Cretaceous periods, intense regional volcanic activity affected the Hong Kong area. This resulted in deposits of pyroclastic rocks (mostly rhyolitic tuff) up to several thousand metres thick, and lesser lava flows. The volcanic rocks collectively occupy about 50% of the land surface area, much of it forming hilly terrain.

During periods of volcanic activity, some of the volcanic material was re-worked by water to form sedimentary rocks. Consequently, the volcanic rocks contain beds of sedimentary rock which vary in grain size, thickness and extent. Furthermore, the volcanic activity was often contemporaneous with the intrusion of the plutonic rocks (see Section 5.2). As a result, metamorphism and deformation are evident in some volcanic rocks and may affect the material properties (see Sections 4.2.3 and 4.3.2).

The key engineering geological considerations for volcanic rocks relate to their origin and post-depositional deformation, leading to potential variability in the following characteristics:

- Composition
- Grain size
- Fabric
- Discontinuities
- Strength

Compared to the plutonic rocks, the material and mass properties of the volcanic rocks are generally more variable and this variability is exacerbated by weathering.

A detailed account of the volcanic rocks is given in Sewell *et al.* (2000). Most volcanic rocks in Hong Kong comprise tuffs of varying age, which forms the basis for their stratigraphical grouping. However, for the purposes of describing their engineering geological characteristics the tuffs are considered as a single rock type in this document.

In addition to tuff, subordinate lavas occur. These vary from rhyolite to dacite and trachydacite to andesite in composition. With the exception of the andesite lava, which has some unique engineering geological characteristics, there is little engineering information on the lavas given their geographical locations in relatively remote areas. Other volcanic rocks which are much less extensive, but which also justify their separate consideration due to distinctive engineering geological characteristics, are:

- Marble-bearing volcaniclastic rock.
- Tuffaceous sedimentary rocks.

The distribution of the volcanic rocks is shown in Figure 5.3.1. Details of the material and mass properties of these rocks are given in Sections 5.3.3 and 5.3.4, and summarised in Table 5.3.1.

5.3.2 Engineering Geological Considerations

Tuff

In the fresh state, tuff is typically much stronger (extremely strong with UCS up to 400 MPa) and more abrasive than granite. This high strength can affect drillability and the performance of tunnel boring machines (TBM).

Tuffs also typically have closer joint spacing than granite but variations occur with grain size. Fine ash tuff tends to have closely spaced joints resulting in a blocky, angular rock mass. Coarse ash tuff tends to have wider spaced joints and can exhibit corestone development when weathered. Columnar jointing occurs in the fine ash tuffs of the High Island Formation.

Lava

The engineering geological characteristics of rhyolite lava (silica-rich) are similar to rhyolitic tuff. However, andesite lava of the Tuen Mun Formation has a mafic-rich mineralogy and is susceptible to deep weathering and usually forms a silt-rich soil. It also has relatively low shear strength when completely decomposed, reaching very low residual shear strength where previous movement has occurred (Section 5.3.6).

Tuff-related Rocks

Tuffaceous sedimentary rocks (e.g. Lai Chi Chong and Mang Kung Uk Formations) have similar engineering geological characteristics to sedimentary rocks (Section 5.6) which vary depending on the strength and composition of the clasts and the matrix, and on the spacing and continuity of the bedding. Minor sedimentary units also occur as interbeds within the main tuff sequences.

The main engineering geological characteristic of marble-bearing volcaniclastic rock (Tin Shui Wai Member, Tuen Mun Formation) is dissolution weathering of the marble clasts which is discussed in Section 5.5.

5.3.3 Tuff Material Characteristics

Rock Composition, Texture and Fabric

Tuffs are classified by a combination of grain size and composition of constituent fragments (Figure 5.3.2). The grain sizes are:

- fine ash (<0.06 mm)

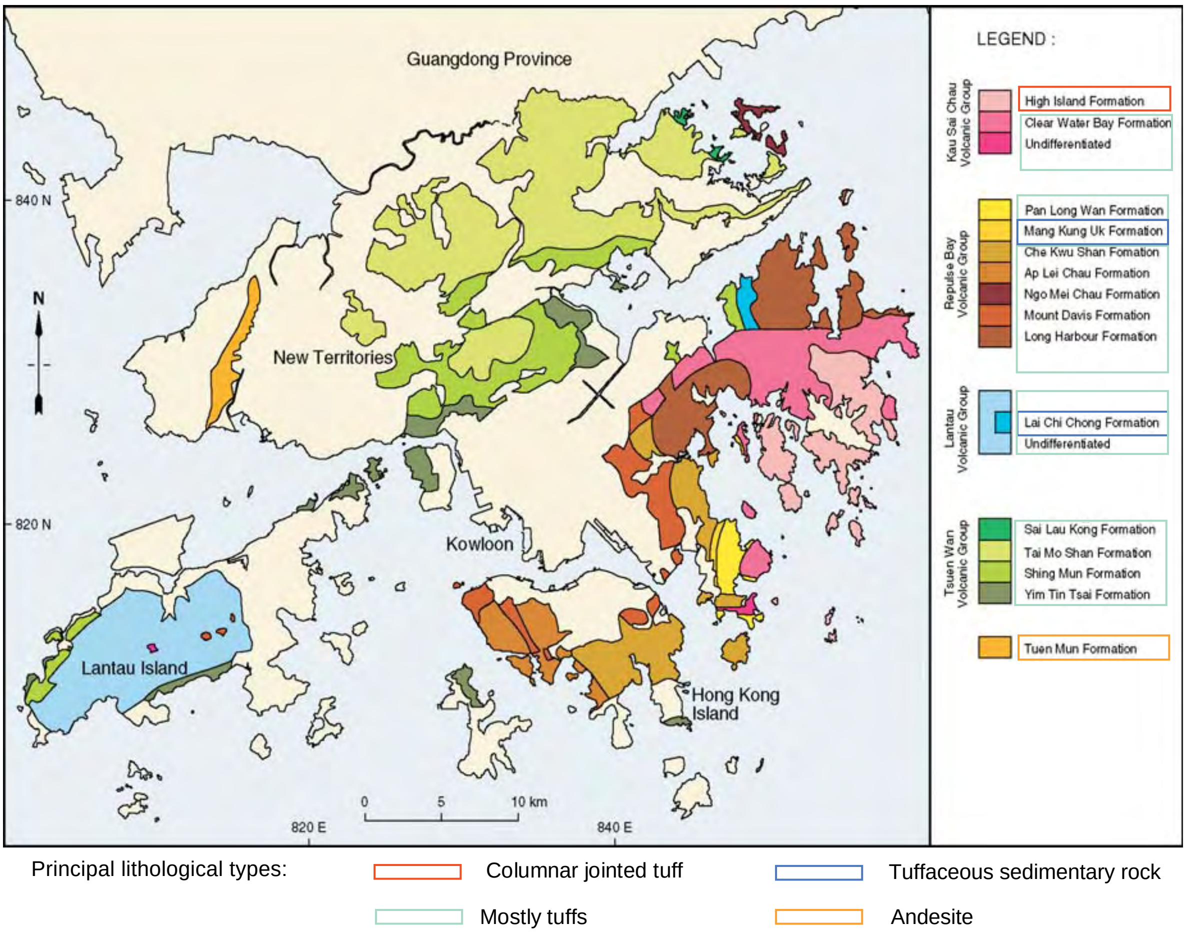


Figure 5.3.1 – Distribution of the volcanic rocks in Hong Kong (after Sewell et al., 2000)

Lithology	Tuff			Volcaniclastic Rocks	Andesite Lava
Sub-Type	**Fine Ash Tuff**	**Coarse Ash Tuff**	**Eutaxite**		
General Characteristics	Tuff and rhyolite lava have a higher SiO_2 content (felsic minerals) and are more resistant to weathering than andesitic (intermediate) rocks. Limited information on rhyolite lava but likely to have similar properties to fine ash tuff and is strongly flow banded. Volcanic rocks are generally less deeply weathered than plutonic rocks and are typically much stronger and abrasive than plutonic rocks when fresh. Tuff and lava are extremely strong when fresh. Volcaniclastic rocks have a wide range in strength, but may be extremely strong where altered by contact metamorphism.				
Discontinuities	Typically closely jointed giving a blocky rock mass. May contain sedimentary units. High Island Formation has well developed columnar jointing.	Typically wider joint spacing than fine ash tuff. May contain sedimentary units.	Welded and flattened fragments (fiamme) indicate bedding.	Commonly bedded.	Andesite contains little or no primary internal structure.
Weathering	Weathered less deeply than coarse ash tuff (typically <15m), rarely develops corestones. Typically forms a clay/silt saprolite.	Moderately thick weathered profiles (up to 35 m) and corestones present. Typically forms a silt or silty sand saprolite.	Weathered less deeply than coarse ash tuff, rarely develops corestones.	Generally more deeply weathered than tuff. If lithic fragments are present they may weather differentially especially if carbonate-rich.	High mafic mineral content results in deep weathering (>20 m) and a clayey silt saprolite.
Grain Size	Very fine-grained / glassy matrix.	Mainly composed of crystals and scattered angular clasts of volcanic rock.	Very fine- grained glassy matrix.	Wide range of grain sizes from fine grained mudstones to coarse conglomerates.	Fine-grained matrix.
Aggregate and Roadstone Properties	Range (Acceptable Limit)				
ACV AIV LAAV 10% Fines (kN) (see Table 5.2.1 for definition)	10-18 (<30) 9-21 (<45) 13-22 (<40) 200-335 (>50)			Insufficient data.	
Suitability for Use in Construction	Most tuffs are suitable as aggregate for concrete and roadstone when in the fresh state, but see Section 5.3.5 regarding uniformity and alkali aggregate reactivity considerations. Suitable for rockfill and armour stone when fresh.			Generally not suitable as aggregate or rockfill due to variability.	Insufficient data on suitability as aggregate. Suitable as rockfill when fresh.
	Most volcanic rocks are suitable as general fill in the weathered state.				
Excavatability	Rock generally requires blasting. Saprolite can be excavated easily by machine. Large corestones may be encountered in coarse ash tuff and may require splitting.				

Table 5.3.1 – Summary of material characteristics and properties for volcanic rocks

- coarse ash (0.06-2 mm)
- lapilli (2-60 mm)
- blocks and bombs (>60 mm).

The constituent fragments generally comprise crystal fragments (crystal tuff), rock fragments (lithic tuff) or pumice/glass fragments (vitric tuff).

The tuffs are mostly rhyolitic in composition (i.e. similar in composition to granite) and therefore consist primarily of quartz, feldspar and subordinate mafic minerals (e.g. biotite), set in a microcrystalline (quartz and feldspar) to vitric (glassy) matrix. Tuffs tend to be more resistant than the plutonic rocks, forming much of the mountainous terrain in Hong Kong.

The most common fabric within the tuffs are welding fabrics, where original glassy shards were aligned, fused and re-crystallised. This results in a fabric referred to as eutaxitic foliation which reflects the original orientation of deposition. The changes in orientation of eutaxite at the Shum Wan Road landslide suggested that faulting might have controlled the deeper weathering at the landslide location (Kirk *et al.*, 1997). A clay-rich zone in tuff, parallel to the eutaxitic foliation was identified as a major influence of the Fei Tsui Road landslide (GEO, 1996a,b).

Relatively thin layers of tuffaceous sedimentary rocks are irregularly distributed within the tuffs. Volcanic rock formations with a significant sedimentary rock component are discussed separately in Section 5.3.7.

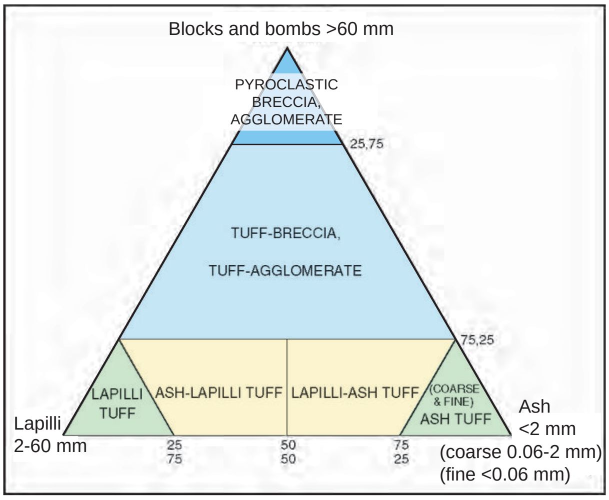


Figure 5.3.2 – Classification of pyroclastic rocks (after Schmid, 1981 and Fisher & Schminke, 1984)

Material Properties

As with other volcanic rocks, the main factor that causes variability in the material properties of tuffs is weathering, in particular chemical decomposition. The degree, depth and rate of weathering of tuff is strongly influenced by both the rock material properties, such as mineralogy, as well as mass properties discussed later in this section. Weathering is discussed in general in Section 4.4. However, the following reflects specific weathering characteristics related to the tuffs.

The decomposition of volcanic rocks generally results in a finer-grained soil than in the plutonic rocks due to the fine microcrystalline matrix which is most pronounced in the fine-grained tuffs (see soil properties). Microfracturing is generally less extensive than in the plutonic rocks. This may be due to the finer grain size and much shallower level of rock formation (resulting in less susceptibility to microcracking related to stress relief) or possibly due to the effects of stress relief being absorbed by the closer jointing (see Section 5.3.4). However, the rocks still exhibit some increase in microfracturing with increasing decomposition grade.

Rock Properties

The properties of tuffs vary considerably depending on the rock type and the degree of weathering and of disintegration (Table 5.3.1). Fresh and slightly decomposed rock, Grades I and II, are very strong to extremely strong. UCS values can be above 300 MPa and such rocks can be hard and abrasive. The finer-grained tuffs tend to be the strongest. High rock strength can be an important issue in some engineering applications, such as tunnel excavation by TBM, as cutter wear is an important economic and technical factor. For Grade III the UCS values drop off to below 50 MPa and for Grade III/IV rock the values are less than 25 MPa. Grade IV tuff is weak when intact but is very often disintegrated due to weathering. In such cases, the rock may be broken down by hand to gravel and smaller sizes and may then be described a soil.

Soil Properties

Saprolite derived from fine ash tuff typically forms a clay/silt when completely decomposed as a result of the breakdown of the constituent grains and the fine matrix containing microcrystalline feldspar and quartz. In coarse ash tuff, the matrix is coarser grained, and silt or sandy silt soil may result.

The fine portion of a soil has a significant effect on the engineering properties. Within the tuffs there can be considerable variability due to lithological variations. The fines content of completely decomposed fine ash tuff has a wide range from 30% to 90%. For coarse ash tuff the range is smaller, typically 50% to 80%, reflecting the nature of the constituent coarse grains.

Fine-grained tuff saprolite is usually in the intermediate plasticity range due to the high proportion of clay weathering products. Variations in plasticity within the tuffs generally relate to variations in chemical composition and degree of weathering.

Saprolites derived from volcanic rocks generally have a relatively high percentage of fines and low intact material shear strength parameters compared to saprolites derived from plutonic rocks. However, there can be considerable variability in density and strength, even within the same rock type and the same decomposition grade.

5.3.4 Tuff Mass Characteristics

The mass characteristics of tuffs depend largely on:

- nature, persistence and spacing of discontinuities, and
- degree of weathering.

Discontinuities

General

In general terms, the tuffs tend to have closer spaced discontinuities than the plutonic rocks, especially within fine-ash tuff where the joint spacing is relatively close with typically four major joint sets defining angular blocks (see Table 5.3.1).

Stress-relief or sheeting joints occur in tuffs but are generally less persistent than those in plutonic rock (see Section 5.2.4).

Columnar jointed tuff

Columnar cooling joints are characteristic of the High Island Formation which comprises massive, fine ash vitric tuff in the Sai Kung and Clearwater Bay areas. Columnar cooling joints typically form perpendicular to the plane of deposition so the High Island cooling joints are steeply dipping to sub-vertical and are observed to be up to 30 m in height (Sewell *et al.,* 2000). Little engineering data is available for these rocks as they are remote from developed areas. However, the engineering geology of the faulted and intruded rocks underlying the High Island Reservoir dam foundations and some of the associated construction difficulties are described in Watkins (1979) and Vail *et al.* (1976) respectively (see Section 6.5.3). Structurally controlled instability is also common in this rock type (Campbell *et al.*, 1999).

Rock Mass Weathering

The weathering profile in fine ash tuff is generally thinner (typically less than 15 m) in comparison to the plutonic rocks. However, in coarse ash tuffs weathering profiles of up to 35 m can develop (Irfan, 1998a). A transitional weathering profile with corestone development is only common in coarse ash tuff. Sharp soil to rock interfaces are more characteristic of fine ash tuff (Figure 4.4.5). Laterally persistent clay can accumulate along planar interfaces such as joints and sheared zones (see the Fei Tsui Road example in Section 6.4.4).

5.3.5 Tuff Material Uses

Most tuffs are mechanically superior to typical granite equivalents and can provide suitable concrete aggregates from both mechanical and physical property viewpoints (Burnett, 1989). From Table 5.3.1 it can be seen that the aggregates produced from fresh tuffs are typically within the acceptance limits for use in concrete and roadstone production. Kwan *et al.* (1995), indicate that aggregates derived from tuff can be more suitable for making high strength concrete than aggregates derived from plutonic rocks. Durability is generally very favourable and well within the soundness criteria (Irfan, 1998a). Tuff is also generally suitable for armour stone and rock fill.

A negative characteristic that can detract from the use of tuffs as aggregates is that bedded tuff rock masses can be heterogeneous and variations in lithology and physical properties can occur over short distances. As approved concrete and asphalt mix designs require uniformity of the aggregate, the geological variations within any prospective site need to be well understood to allow the suitable strata to be selectively extracted. For these reasons, relatively uniform and thickly-bedded coarse ash tuff is more favourable for quarrying.

A study of the alkali aggregate reactivity (AAR) potential of tuff aggregates from the Anderson Road Quarry indicated that they were “potentially reactive” (Leung *et al.*, 1995). The reactive component is generally microcrystalline and cryptocrystalline (glassy) quartz, which is found mainly in the fine ash and vitric tuffs. However, the AAR potential can be controlled with the addition of pulverised fuel ash (PFA) in the concrete mix. Chak & Chan (2005) give a review on prevention of alkali silica reaction which is the main form of AAR in Hong Kong.

The saprolitic soils resulting from the decomposition of tuffs are generally suitable as earth fill material.

5.3.6 Lava

Lavas typically have a fine-grained matrix whose individual crystals cannot be seen by the naked eye. These rocks are sometimes porphyritic, containing large individual crystals within the fine matrix. Although rhyolite lava is more common than other types, it is also more geographically remote and there is little engineering data on its properties. However, due to its rhyolitic composition, it is likely to have similar material properties to the tuffs.

In comparison, andesite lava within the Tuen Mun Formation presents significant engineering problems. The andesite lavas are intermediate in composition, i.e. they are quartz deficient and relatively rich in ferromagnesian minerals which makes them more prone to chemical decomposition.

Completely decomposed andesite is typically a firm to stiff, becoming very stiff with depth, greenish grey, slightly clayey silt. A summary of peak and residual shear strength values is given by Koor *et al.* (2000). The shear strength generally increases with depth, with typical peak values of $c' = 6$ kPa and $\phi' = 32°$ within the uppermost 10 m. Typical index properties for completely decomposed andesite are shown in Table 5.3.2.

Taylor & Hearn (2000) indicate that the uppermost 5 to 10 m of the andesite saprolite in Area 19, Tuen Mun, is intensely weathered with a marked increase in plasticity and fines content. This probably reflects almost complete decomposition of the feldspars and mafic minerals such as hornblende and pyroxene.

Liquid Limit (%)	Plasticity Index (%)	Moisture Content (%)	Clay Content (%)	Silt Content (%)
42-63	11-26	19-48	2-11	80-90

Table 5.3.2 - Index properties of completely decomposed andesite (Koor et al., 2000)

The relict joints within the completely decomposed andesite are commonly slickensided, typically with manganese oxide staining and thin films of clay. In Area 19 (see case history in Section 6.3), low-angle, large-scale shear surfaces which typically contain soft to firm grey remoulded clay are associated with large instabilities.

Shear box testing and back analyses of failures in Area 19 indicate that the residual friction angle of the shear planes generally varies from 9° to 17°, with $c' = 0$ (Koor *et al.*, 2000). Given such low residual strength, there is a possibility that very low friction clay minerals may be present in the shear plane infills.

Owing to its high silt content, the completely decomposed andesite is highly susceptible to erosion and softening by surface water. This can lead to the extensive development of pipes and deep gullies which typically exploit steeply dipping relict joints striking down slope.

A large-scale, very slow moving natural terrain landslide has been reported in andesite to the south of Leung King Estate (Parry & Campbell, 2003). This landslide involves approximately 40,000 m^3 of predominantly colluvial material which is gradually moving down slope on basal shear planes developed at or just below the colluvium/completely decomposed andesite interface. The landslide is a complex, probably very old feature with fresh, well developed lateral tension cracks. The overall angle of the surface of the landslide material is approximately 15°, and about 85 mm of cumulative downslope movement was recorded by an inclinometer over a period of 12 months (Parry & Campbell, 2003).

The previous instabilities affecting man-made slopes in Area 19, and the occurrence of large-scale natural terrain landslides associated with andesite, indicate that caution should be exercised when planning site formations in this material, particularly where high base groundwater levels and perched groundwater tables are suspected (see Section 6.3).

5.3.7 Other Volcanic Rocks

Tuffaceous Sedimentary Rocks

Sedimentary rocks have a variable range of grain size, depending on the depositional environment. The rocks range from fine-grained mudstones to coarse-grained conglomerates. The classification of tuffaceous sedimentary rocks is similar to that for sedimentary rocks (Section 5.6). Volcanic rock formations in Hong Kong which contain significant sedimentary units include the Mang Kung Uk Formation (Figure 5.3.3) and the Lai Chi Chong Formation. The remoteness of these formations (Figure 5.3.1) means that little geotechnical data are available, although detailed geological descriptions of the two formations are given in Sewell *et al.* (2000). Minor sedimentary units also occur within the more massive tuff units.

Due to the variable mode of formation and the heterogeneous, interbedded nature of these rocks, lateral and vertical changes in rock lithology may occur with implications for the engineering properties.

Bedding may affect the engineering properties of tuffaceous sedimentary rocks by forming planes of weakness and making them more susceptible to differential weathering. In closely-bedded sequences of varying lithology (and weathering properties), a heterogeneous rock mass may be formed consisting of alternating relatively unweathered and weathered beds (Figure 5.3.3).

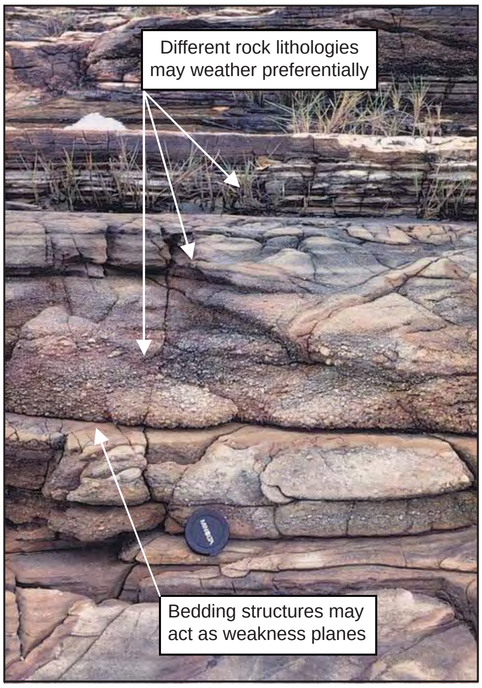


Figure 5.3.3 – Bedded tuffaceous sedimentary rocks with variable grain size (conglomerate, sandstone, siltstone and mudstone) in the Mang Kung Uk Formation (Sewell et al., 2000)

Marble-bearing Rock

Marble-bearing breccia in the Tin Shui Wai Member, Tuen Mun Formation occurs in discrete layers within the volcaniclastic succession. The marble clasts are relatively small and dissolution of the clasts is likely to be local and limited in scale (see Section 5.5).

5.4 DYKE ROCKS

5.4.1 Introduction

Dykes are minor intrusive igneous rocks that are typically sub-vertical and of limited thickness (i.e. a few centimetres to tens of metres wide). Sills (gently-inclined sheet-like intrusions) also occur but are generally minor. Since the engineering effects of sills are similar to dykes, where they are of similar thickness and composition, sills can be regarded as 'dyke rocks' for the purposes of this document. Dykes can have considerable lateral and vertical extent and may be composite in nature, varying in grain size and composition. Dykes occur throughout most of Hong Kong, either singly or in groups (dyke swarms).

A detailed description of dyke rocks is given in Sewell *et al.* (2000). However, in engineering geological terms, the dyke rocks of Hong Kong can be divided into silica-rich (rhyolitic) and silica-poor (mafic).

The key engineering geological issues with dyke rocks mainly relate to diferences in mass and material properties between the dyke and the surrounding host rock, the effects of the intrusion of the dyke on the host rock and the effect of differential weathering between the dyke and the host rock (country rock).

5.4.2 Engineering Geological Considerations

The contact margins between a dyke and the host rock may result in abrupt changes in material characteristics and discontinuity characteristics. Fractured zones can occur along the chilled contact margins with consequent poor rock mass properties and higher permeability (see Section 6.7.3). However, delineating contact margins over large areas from drillholes can be difficult as the contacts can vary from planar to highly irregular.

The weathering of dyke rocks is controlled mainly by their mineralogical composition and discontinuity frequency, spacing and persistence. Consequently, dykes may weather preferentially or be more resistant than the surrounding country rock. Mafic dykes readily weather to clay-rich soils (Au, 1986) whereas rhyolitic dykes are generally resistant to weathering.

Resistant rhyolitic dykes traversing hillsides may form positive topographic linear features resulting in areas of over-steepened terrain immediately downslope (see the Lai Cho Road case study in Section 5.4.6). Completely decomposed mafic dykes may act as aquitards (see the Tuen Mun Highway case study in Section 5.4.6). The 300 m^3 failure at the 14½ Milestone on Castle Peak Road in 1994 is another example of weathered mafic dykes affecting the hydrogeology (Franks, 1995; Chan *et al.*, 1996b).

Mafic dykes may preferentially weather for several tens of metres below the surrounding country rocks resulting in uneven rockhead levels. Given their commonly sub-vertical nature, weathered dykes may not be encountered during a ground investigation but may significantly affect subsequent works.

5.4.3 Origin and Occurrence of the Dyke Rocks

Rhyolite is granitic in composition with a grain size <0.06 mm. Rhylolitic dykes can be subdivided into feldsparphyric and quartzphyric, depending on the nature of contained phenocrysts. Feldsparphyric rhyolite dykes are the most common type and are mainly concentrated in a large dyke swarm on the northeast of Lantau Island, although they do occur elsewhere as single features. Quartzphyric dykes are located throughout Hong Kong, including part of the Lantau dyke swarm. Elsewhere, they form smaller swarms or single features.

Mafic dykes are basaltic in composition and are widespread throughout Hong Kong. They generally occur as narrow (<1 m thick) dykes (Sewell *et al.*, 2000), but may also be found occasionally up to 6 m wide or as small stocks (Sewell, 1992).

5.4.4 Material Characteristics

Feldsparphyric Rhyolite

Many of the smaller dykes (<5 m wide) are relatively uniform in grain size and texture. However, the larger dykes commonly grade internally from rhyolite on the margins to porphyritic fine-grained granite in the centre, with feldsparphyric rhyolite in between.

In its unweathered state, feldsparphyric rhyolite is a very strong to extremely strong rock due to its granitic mineralogy and fine grain size of the matrix. Feldsparphyric rhyolite typically decomposes to a silty soil, due to the fine-grained matrix, with some coarse quartz sand. These dyke rocks tend to decompose slightly faster than granite and slightly slower than volcanic rocks such as tuffs. This may result in linear surface expressions of subtle positive or negative topographic relief.

Quartzphyric Rhyolite

These rocks can occur as isolated dykes or as swarms and can be up to 60 m wide (Sewell *et al.*, 2000). In many cases the dykes exhibit flow banding structure and are generally northeast trending, along or parallel to major fault zones.

Unweathered quartzphyric rhyolite is a very competent rock due to its granitic mineralogy, but there is little quantitative data available. These dykes typically decompose to a sandy silt soil due to the disseminated quartz crystals, and tend to be relatively resistant rocks, forming positive topographic features.

Mafic Dykes

Mafic dykes are rich in dark magnesium and iron minerals. They vary in composition, with basaltic andesite, dacite, quartz-diorite and lamprophyre all being reported (Sewell *et al.*, 2000). For the purposes of this document these dykes are referred to as mafic dykes. Weathering of the mafic dyke rocks generally results in clay-rich soils (Figure 5.4.1).

5.4.5 Mass Characteristics

Rhyolitic Dykes

Fracturing along the contact margins of the feldsparphyric rhyolite dykes can be very pronounced, leading to blocky seams of rock with low RQD and relatively high permeability. For example, during ground investigations in northeast Lantau, highly fractured rock and very closely spaced joints were observed at sharp contacts between tuff country rock and feldsparphyric rhyolite dykes. These zones had low RQD values (35 - 50%) with fairly high permeability (about 10 Lugeon Units). During the construction of the Harbour Area Treatment Scheme (HATS) Stage 1 tunnels, rhyolite dyke contact margins observed in tunnel driving were characterised as having a "highly blocky" structure and a "complex network of voids" (CDM, 2004; see Section 6.7).

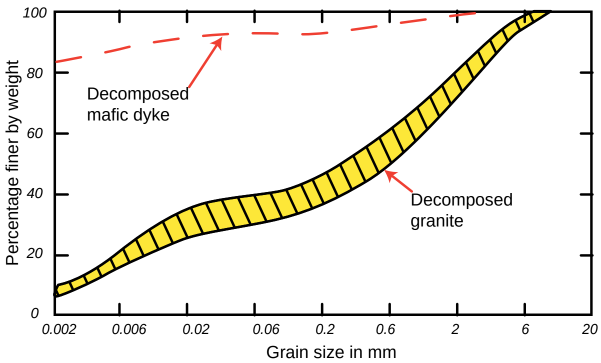


Figure 5.4.1 – Variation in grain size between decomposed mafic rock and decomposed granite (after Au, 1986)

Mafic Dykes

Mafic dykes encountered in the HATS Stage 1 tunnels CDM (2004) typically had fewer joint sets (two to three) than the rhyolitic dykes (three to four). The joints often contained veins or segregations of calcite. Observations made during construction of the HATS Stage 1 tunnels indicated that mafic dykes intruding into granite tended to have sharp contacts, whereas highly fractured margins were common in mafic dykes intruding into tuff as observed elsewhere (see Figure 5.4.2). Consequently, in the relatively unweathered state, mafic dykes may have variable rock mass quality related to the type of host rock.

On weathering, the small grain size and mafic composition typically results in clay-rich soils. In the HATS Stage 1 tunnels, completely decomposed mafic dykes, comprising firm to stiff clays, occurred at depths of 30 to 80 m below rockhead and were often found to be sheared. Sheared dyke margins were also noted at the Tsing Ma Bridge site (see the case study below).

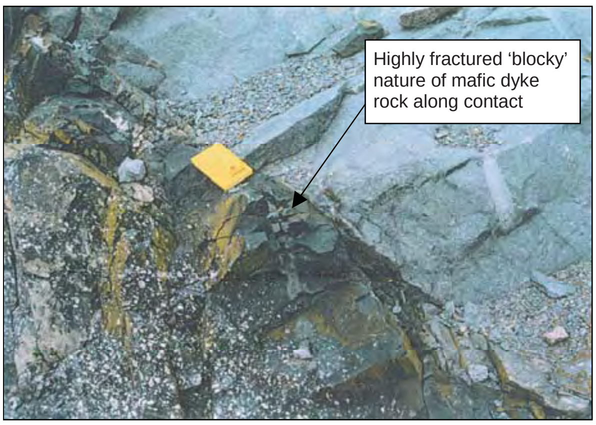


Figure 5.4.2 – Chilled contact margin of mafic dyke rock against coarse ash tuff in NE Lantau (Li et al., 2000)

5.4.6 Case Studies

Tsing Ma Suspension Bridge Anchorage on Tsing Yi Island (Langford, 1991)

A 200 mm thick clay-rich zone was found along the northern margin of a 5 m wide east-northeast trending mafic dyke at the Tsing Yi anchorage site of the Tsing Ma Bridge. This clay-rich zone was described as a soft to firm clay gouge and interpreted as a fault zone. The location and orientation of this feature, at a critical point in the proposed tunnel anchorage system (Figure 5.4.3), resulted in abandonment of this design (Yim, 1998).

Relict Landslides above Lai Cho Road, Kwai Chung (MGSL, 2002; Thorn et al., 2003)

Large relict landslide scars (~75,000 m^3 source volume) were identified in the granitic natural hillside above Lai Cho Road during an LPM site investigation (Figure 4.4.17). During field mapping it was observed that the main scarp of the relict landslide source areas coincided with a line of intermittently exposed aplite, fine-grained granite and feldsparphyric rhyolite dykes forming positive topographic linear features traversing obliquely across the hillside. It was inferred that these dykes may have influenced the extent of instability by influencing groundwater on the downhill side and by limiting uphill retrogression (Figure 5.4.4).

Tuen Mun Highway, 1982 (Hencher & Martin, 1984; Hencher, 2000).

A landslide occurred on the Tuen Mun Highway in 1982, following heavy rainfall. The failure occurred within Grade IV/V granite with persistent relict joints. Exposed within the source area were two mafic dykes

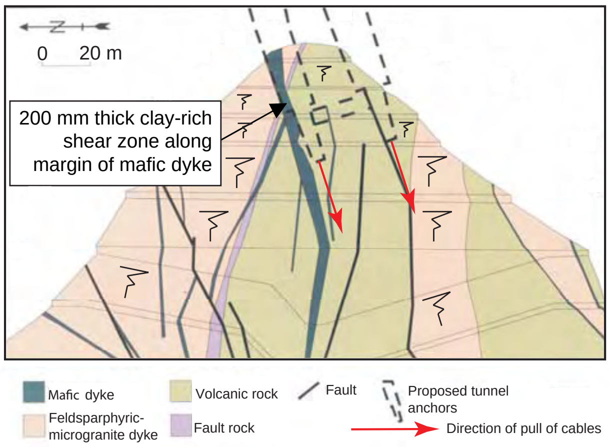


Figure 5.4.3 – Schematic geology of proposed anchorage site of the Tsing Ma suspension bridge on Tsing Yi Island (after Langford, 1991)

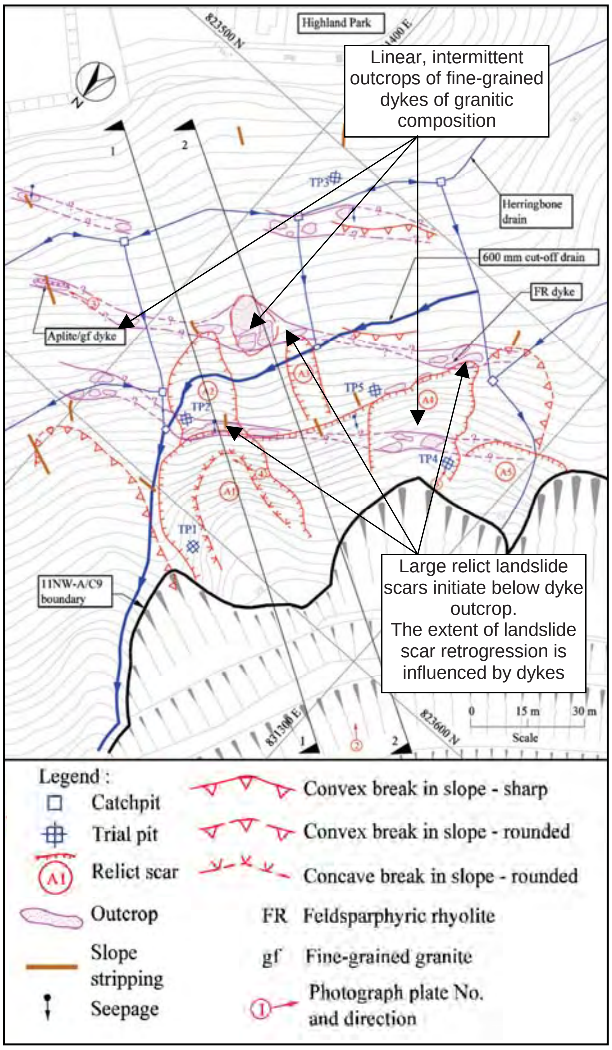


Figure 5.4.4 – Map of dyke outcrops on the natural hillside above Lai Cho Road (MGSL, 2002 and Thorn et al., 2003)

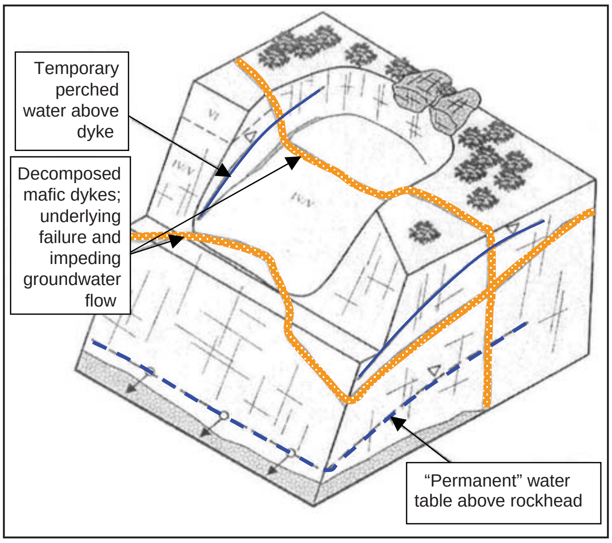


Figure 5.4.5 – Schematic geological model of the 1982 landslide on Tuen Mun Highway (Hencher, 2000)

about 2 m wide which had weathered to clay-rich soil. One dyke was steeply dipping into the slope and the other was a low angle feature dipping out of the slope. Some relict joints aligned at a similar angle to the gently dipping dyke were also present.The failure was mainly attributed to sliding along the relict joints and low angle dyke, promoted by the build-up of a high transient perched groundwater table which had developed above the relatively impermeable, low angle dyke (Figure 5.4.5).

5.5 MARBLE AND MARBLE-BEARING ROCKS

5.5.1 Introduction

This section considers the characteristics of marble and marble-bearing rocks, and their origin, properties and variations that may affect their engineering performance. Particular emphasis is given to the distinctive effects of weathering unique to these rocks, and the resulting engineering problems.

Marble results from the metamorphism of carbonate rocks such as limestone or dolomite. Natural outcrops of marble and marble-bearing rocks are extremely rare in Hong Kong. Consequently, information on the nature of these rocks is mostly limited to the interpretation of drillhole cores. Therefore, a relatively high degree of uncertainty is associated with knowledge of this rock type.

The key engineering geological consideration for marble is that it weathers by dissolution. Dissolution erodes the marble on surfaces in contact with moving water, often leading to highly irregular rockhead profiles and cavities within the rock, i.e. 'karst'. Factors which affect the extent and degree of dissolution include:

- purity of the marble,
- extent and thickness of the marble beds,
- frequency, orientation and number of discontinuity sets,
- rate and duration of flow of groundwater containing carbon dioxide over geological time, and
- size of the original marble clasts and permeability of the matrix where the marble is contained within other rock types.

These distinctive geological factors have a significant influence on the state of dissolution of marble and rocks containing marble clasts. Consequently, establishment of sound geological models and the characterisation of the state and pattern of dissolution within a site are key to understanding the potential geotechnical constraints.

5.5.2 Engineering Geological Considerations

The nature and depth of dissolution in marble rocks depends on the amount and type of impurities present in the material, as well as the vertical and lateral extent of the marble. Although in rare cases un-metamorphosed limestone is identified locally in drillhole cores, in Hong Kong marble is the dominant carbonate rock and its occurrence can be broadly divided into three main types:

(i) Marble

Relatively pure marble (>95% carbonate) goes into complete dissolution and unweathered marble may be overlain by a very thin residual soil representing the insoluble residue of the original rock. Given its purity, dissolution can be very extensive in this rock type. Pure marble occurs around Yuen Long (Ma Tin Member of the Yuen Long Formation) and Ma On Shan (Ma On Shan Formation).

(ii) Impure and Interbedded Marble

Where the marble is impure (50-95% carbonate), or where the marble is interbedded with other rock types, the amount of dissolution is restricted. Impure marble occurs around Yuen Long (Long Ping Member of the Yuen Long Formation) where it is about 300m thick (Lai, 2004). Because of the higher amount of impurities, impure marble may develop a residual soil more than 30 m thick (GCO, 1990a).

(iii) Marble Clasts in Other Rocks

Where the marble forms individual clasts within other rock types the amount of dissolution is related to the size and frequency of carbonate clasts, and the permeability of the non-carbonate matrix.

The marble was exposed during low sea-level stands in the Quaternary Period (Fyfe *et al.*, 2000). As a result, karst development included the formation of a highly irregular rockhead with pinnacles, overhangs and depressions, cavities (linear and spheroidal), cavity infill deposits, and collapsed cavity features (dolines). Following the rise in sea-level in the Holocene, the karst surfaces were covered by superficial sediments, and in the case of Man On Shan, submerged. Hence karst in Hong Kong is palaeokarst in that its formation is restricted in the present day environment. The karst can be classified as mature to complex and only locally extreme (Waltham & Fookes, 2003).

The main zone of karst features (caverns, cavities, etc.) may be up to 30 m thick in pure marble. Local solution features may continue considerably deeper, especially adjacent to faults, and boundaries with other lithologies including igneous intrusions. Voids are commonly infilled with unconsolidated deposits. These infills are usually poorly cemented silts and sands or soft clays, commonly with an organic content. Recovery of cavity-fill material from drillholes is usually poor, with the soft sediments being washed away by the drill flush, particularly where the cavity is narrow.

As dissolution is the main weathering process, material weathering classifications based on decomposition should not be applied to marble (GEO, 1988a). A summary of the characteristics of marble rocks found in Hong Kong is shown in Table 5.5.1.

5.5.3 Material Characteristics

Yuen Long Formation

The Yuen Long Formation was first encountered in drillholes during the development of the Yuen Long new town (Langford *et al.*, 1989; Frost, 1992). It is located within an area that stretches from south of Yuen Long to Mai Po (Figure 5.5.1). In the Yuen Long area, it is more than 600 m thick and is divided into two members (Frost 1992):

- the lower, older, Long Ping Member is an impure marble, often interbedded with non-carbonate lithologies.
- the upper, younger, Ma Tin Member is generally a massive, pure white marble.

The main extent of these two units in the northwest New Territories is shown in Figure 5.5.1, although some marble of the Ma Tin Member has also been located at Lok Ma Chau (Campbell & Sewell, 2004). Marble also subcrops below the Brothers Islands where it probably reaches a similar thickness to that found in Yuen Long (Langford *et al.*, 1995).

Both types of marble in the Ma Tin and Long Ping Members are strong to very strong rocks in the fresh state, with UCS values of about 50 MPa to 140 MPa. Elastic modulus values range from about 45 GPa to

	Marble			Marble-bearing Rocks	
	Yuen Long Formation		Ma On Shan Formation	Tolo Harbour Formation (Designated Area of Northshore Lantau only)	Tuen Mun Formation
	Long Ping Member	Ma Tin Member			Tin Shui Wai Member
Typical Description (fresh)	**Grey to Dark Grey Marble.** Locally impure. Interbedded with non-carbonate rocks.	**White Marble.** Generally pure.	**White to Grey Marble.** Dolomite to calcite marble with thin (<10 mm) interbeds of dark green metasiltstone.	**Marble and limestone xenoliths.** Cream, blue grey or dark grey marble.	**Marble Clasts.** Within tuffaceous volcaniclastic rock.
Distribution/Origin/ Depositional setting/ Thickness	Yuen Long. Predominantly fault-bounded. Derived from impure limestone and interbedded sequences of thin limestones and calcareous mudstones and siltstones (coastal inshore to tidal/ swamp depositional setting (>300 m thick).	Yuen Long, Tin Shui Wai and Fairview Park. Predominantly fault-bounded. Derived mostly from pure limestone and dolomitic limestone deposited in a shallow shelf sea environment (>250 m thick).	Ma On Shan reclamation. Fault-bounded blocks; probably extending out into Tolo channel. Originally pure to impure limestone deposited in a shallow shelf sea environment (>200 m thick).	In the vicinity of Tung Chung, probably extending offshore along N. Lantau. It occurs as large (up to 350 m) xenoliths (blocks) assimilated from marble country rock by the later intruded Lantau granite plutons.	Tuen Mun Valley to Deep Bay. Occurs as angular to subrounded clasts within a tuffaceous, volcaniclastic matrix.
Mineral content Calcium Carbonate (calcite). Calcium Magnesium carbonate (dolomite).	Calcite with dolomite in dolomitic marble plus varying silicious and clay mineral impurities (typically 8 -33%).	Calcite with lesser dolomite in dolomitic marble.	Calcite and dolomite with minor silicious and clay mineral impurities.	Predominantly re-crystallised calcite (limestone). Little dolomite. Minor silicious and clay mineral impurities.	Depends on parent carbonate rock composition but typically calcite and dolomite.
Grain size/Fabric/Structure	Fine-grained. Moderately to widely spaced joints.	Fine- to medium-grained. Crystalline. Very widely spaced joints.	Fine-grained. Very widely-jointed. Dips steeply (70-80°) to SE.	Fine to coarse grained. Crystalline. Massive, banded or calcite veined fabric.	Very fine-grained clasts. Sedimentary breccia/conglomerate.
Karst Characteristics Supra-karst deposits ('weathered' soil profile)	Up to 30 m thick depending on % impurities.	Little or none. In pure carbonate rocks intermediate 'weathering' grades and karst superficial deposits do not normally occur.	Thin (generally less than 5 m).	May develop on large impure marble blocks.	N/A
Karst surface	Irregular karst surface (on smaller scale with increasing supra-karst development). Developed over laterally extensive areas.	Highly irregular karst surface with pinnacles, depressions, overhangs and gullies.	Locally highly irregular karst surface with pinnacles, depressions and gullies.	Restricted to local blocks of carbonate rock (some very large – i.e. 350 m).	Restricted to local blocks of carbonate rock (usually small – i.e. 20 – 50 mm).
Epikarst development (dissolution features e.g. cavities)	Cavities generally less than 1m high.	Developed over laterally extensive areas. Up to 30 m thick (less with increasing impurities). Pure marble most susceptible to cavity formation. Linear and spheroidal cavities commonly 0.1-2 m in length mostly occur near karstic surface; less commonly up to 25 m in length (Ma On Shan, maximum cavity size up to 10 m). Cavities usually infilled.		Localized collapse and cavity fill deposits within isolated carbonate blocks (extent depends on size of individual blocks).	Dissolution of the clasts is likely to be local and limited in scale.
Nature/consistency of cavity infill	Silt/clay. Typically soft deposits.	Silt/clay. Typically soft deposits.	Silt, clay and sand with clasts of marble and other lithologies. Wide range of consistency.	In large marble blocks the cavity fill deposits can be very variable including sand, gravel, laminated silts and collapse fragments.	Normally not infilled.
Engineering Implications (see also (Table 5.5.2)	Needs to be differentiated from thick sequences of pure marble, due to smaller potential cavity size.	Highly irregular rockhead profile which is difficult to interpolate from limited drillholes. Possibility of gross failure of foundations if located above cavity. Thick epikarst development. Some cavity solution features may go much deeper locally, especially adjacent to faults, minor igneous intrusions or lithological boundaries (e.g. in Ma On Shan cavities found at -100 mPD and solution features in joints found at -140 mPD).			Small and isolated karst features. Possibility of rock mass deformation issues rather than gross failure.
Material Strength Properties*	Typically moderately strong to strong. UCS: 50 - 140 MPa * (120 -190 - silicified)**	Typically moderately strong to strong. UCS: 90 -120 MPa*	Similar to Yuen Long Marble.	Moderately strong to strong. UCS: 65-140 MPa *	No data.

* Based on testing carried out by GCO (1990)
** Silicified by nearby granitic intrusion

Table 5.5.1 – Summary of material and mass characteristics of marble and marble-bearing rocks

95 GPa (GCO, 1990a). The range in strength is due to variations in texture, grain size and proportion of impurities present in the rock. In the more variable Long Ping Member, UCS values are generally between about 40 MPa and 65 MPa. In contrast, where impure marble has been silicified by contact metamorphism from granite intrusions, UCS values can be up to 190 MPa (GCO, 1990a).

Ma On Shan Formation
Marble subcrop is found below the reclamation at Ma On Shan (Figure 5.5.2), and in offshore drillholes in Tolo Harbour (Sewell, 1996). Similar to the Ma Tin Member this is a relatively pure marble although it does contain thin (<10 mm) interbeds of meta-siltstone (Sewell, 1996). It is strongly foliated with a steep dip angle (70 – 80°) to the southeast and has a minimum thickness of 200 m.

The Ma On Shan Formation has a faulted contact with the adjacent granite. The fault zone is 10 to 100 m wide and trends northeast. The fault zone comprises highly sheared rock, consisting of brecciated marble and siltstone which have been mineralised and hydrothermally altered to skarn in parts (Sewell, 1996). A cross-section through the contact zone is shown in Figure 5.5.3.

The marble of the Ma On Shan Formation has similar strength properties to the Yuen Long Formation, being strong to very strong in the fresh state. Given the lack of impurities, the residual soil is usually less than 5 m thick. The rock mass characteristics are similar to the marble of the Ma Tin Member of the Yuen Long Formation.

Tolo Harbour Formation
Marble has been found at depth below recent developments in Tung Chung, and adjacent areas along the Northshore of Lantau Island (Figure 5.5.4).

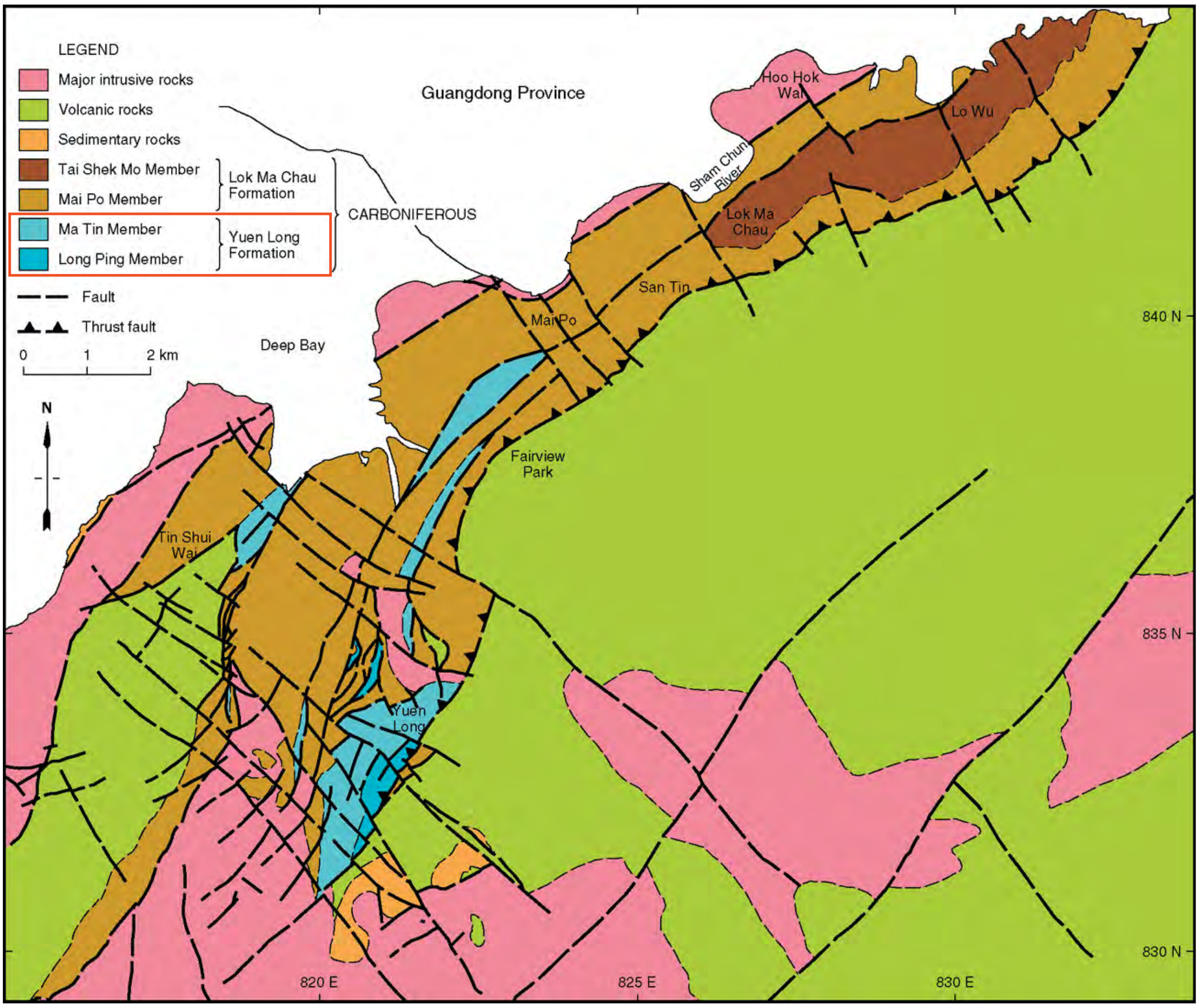


Figure 5.5.1 – Distribution of the Yuen Long Formation in the northwestern New Territories (after Sewell et al., 2000)

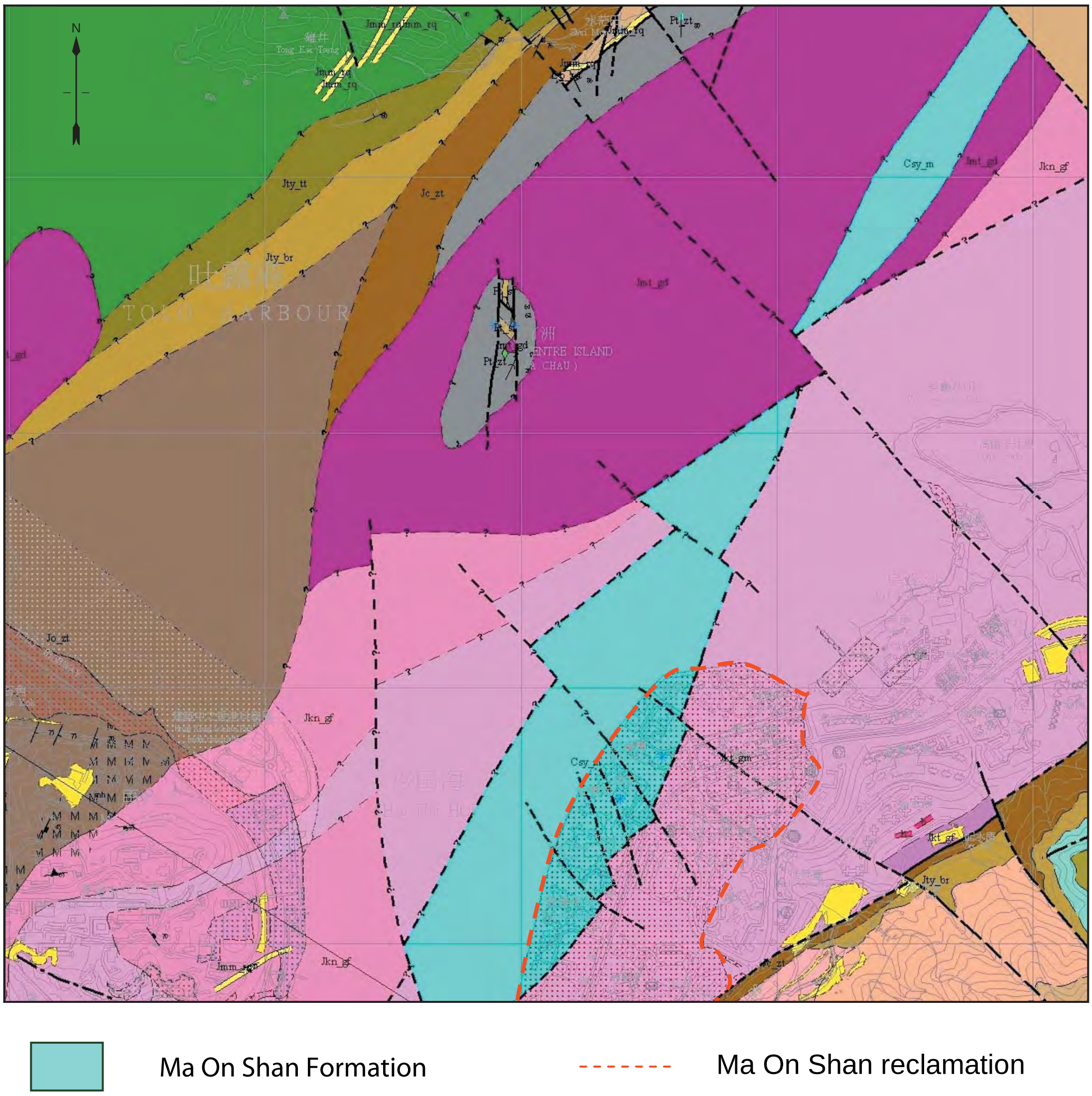


Figure 5.5.2 – Marble extent below the Ma On Shan area (updated HKGS 1:20,000 scale geological Map Sheet 7, 2005)

It does not form a laterally extensive subcrop but occurs as isolated xenoliths in younger granite plutons. Some of these xenolithic blocks can be very large, reaching up to 300 m across (Sewell & Kirk, 2002). Fossils show the rocks to be of Permian age, hence they are considered to be equivalent to the Tolo Harbour Formation. Information on material characteristics and properties of these carbonate rocks is limited.

Tuen Mun Formation

Although the Tuen Mun Formation comprises mainly volcanic rocks (Section 5.3), the Tin Shui Wai Member contains marble, siltstone and quartzite clasts. Darigo (1990) suggested different possible modes of formation for the Tin Shui Wai Member. These include emplacement or extrusion of magma along dykes and sills with incorporation of the marble and other country rocks through underground brecciation and mixing, deposition as lahar-type flows, and reworking of the volcanically derived material by sedimentary processes. Lai (2004) suggested a purely volcanic origin. While the mode of formation of the marble clast-bearing zones may have some bearing on the ability to define the extent of the zones within a site, the mode of formation carries little implication for the susceptibility of the rock to dissolution.

5.5.4 Mass Characteristics

Pure Marble of the Ma Tin Member of the Yuen Long Formation and the Ma On Shan Formation

The bedding has been largely destroyed by metamorphism, and the rock is massive in nature

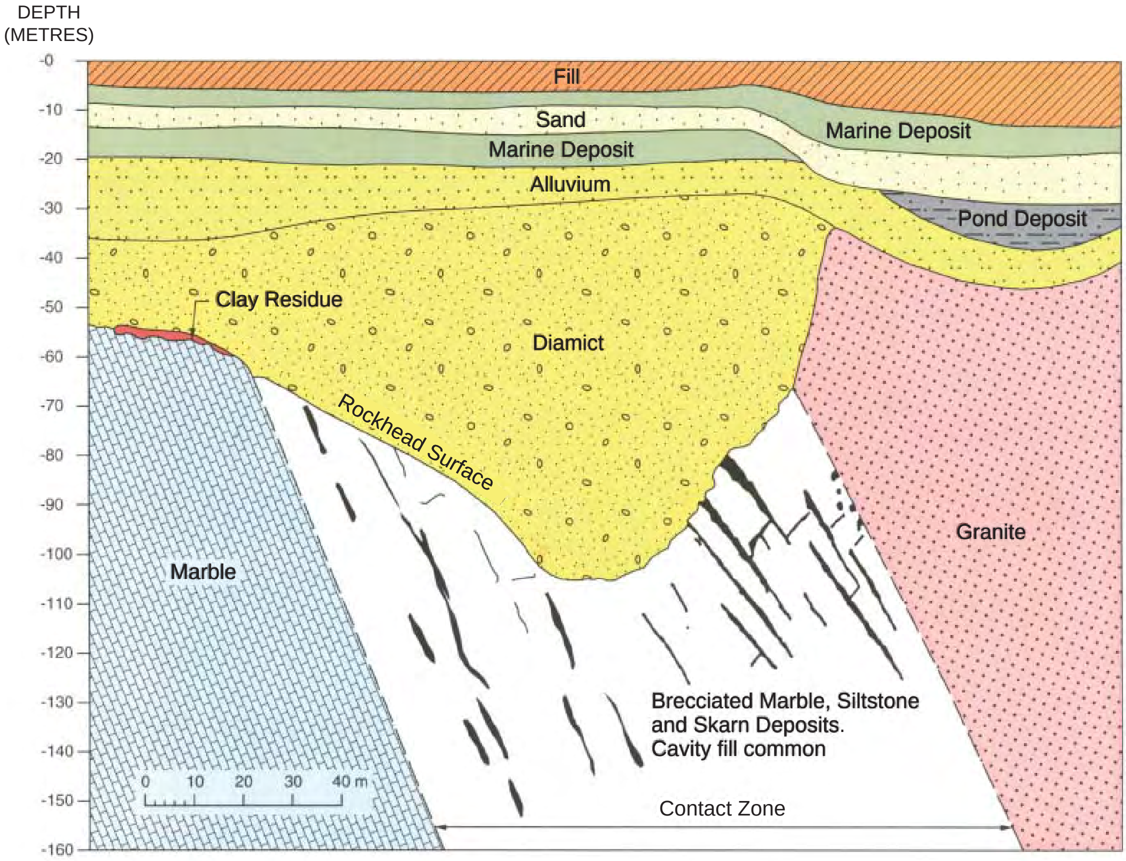


Figure 5.5.3 – Cross-section through the contact zone between marble and granite beneath the Ma On Shan reclamation (after Sewell, 1996)

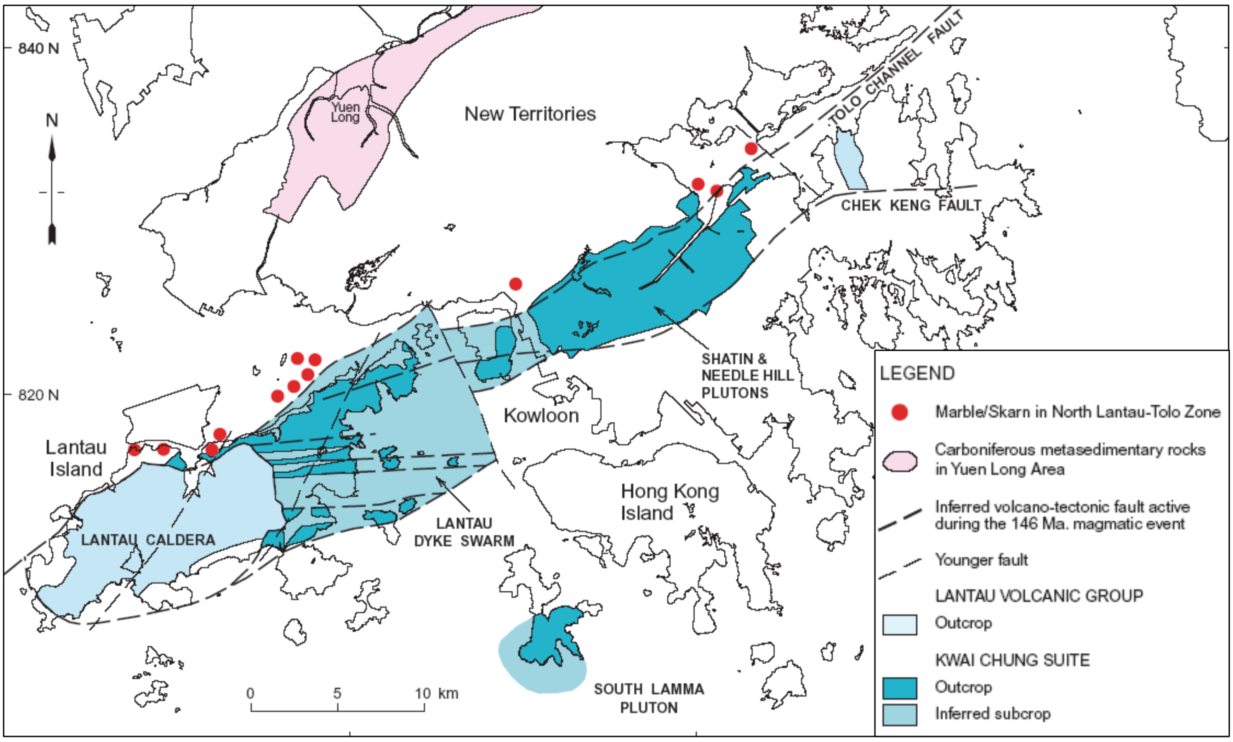


Figure 5.5.4 – Volcano-tectonic map of Hong Kong showing Tung Chung and Ma On Shan marble sub-crops and faults that were active during emplacement of the Lantau Volcanic Group and Kwai Chung Suite granitoid dyke swarms and plutons (Sewell & Kirk, 2002)

with very widely spaced joints. The rock is susceptible to solution weathering and consequently the rockhead is typically irregular with karst surface features such as pinnacles, overhangs, gullies and dolines (Figure 5.5.5). The joints allow the formation of extensive linear cavities and spheroidal cavities where joints intersect or where adjacent to lithological contacts. Voids are rare as the cavities are commonly infilled with clay/silt. The zone of significant dissolution features is generally up to 30 m below rockhead (Frost, 1992) with cavities typically ranging from 1 m to 2 m high but reaching up to about 25 m (Lai, 2004).

The fault zone at Ma On Shan has allowed deeper and more extensive karst development with large (up to 10 m in height) cavities down to great (-100 mPD) depths (Kwong *et al.*, 2000). The depth of weathering and karst solution in Ma On Shan are shown in Figure 5.5.6).

Long Ping Member of the Yuen Long Formation

The Long Ping Member is considered to have originated as an interbedded sequence of thin limestones, calcareous, mudstones and siltstones prior to being metamorphosed (Frost, 1992). Due to the interbedded characteristic and the relatively high content of impurities most cavities encountered are less than 1 m high with the maximum height reported being 4.5 m (Lai, 2004).

Tolo Harbour and Tuen Mun Formations

In terms of rock mass characteristics the marble within these formations differs fundamentally from other marble strata.

At Tung Chung, the marble of the Tolo Harbour Formation forms discrete isolated blocks within the surrounding granite. For engineering purposes the surrounding rock mass should be considered together with the effects of dissolution weathering.

Marble-bearing breccia in the Tuen Mun Formation occurs in discrete layers within a meta-siltstone to metatuff succession. The marble clasts found in the Tuen Mun Formation have been found to be relatively small, and dissolution of the clasts is likely to be local and limited in scale. As a result, cavities are limited in size when the matrix is in a relatively fresh state. Lai (2004) described them as typically less than 0.2 m in size in which case the cavities are likely to have more implications for rock mass deformation than karst-related bearing failure when loaded by piles. Darigo (1990) described a range of heights of cavity of between 0.6 m and 0.8 m in the northern subcrop area and discussed the implications of dissolution of the marble clasts on the behaviour of the weathered rock.

The development of solution dolines and collapse dolines in the marble xenoliths at Tung Chung (Figure 5.5.7) have posed severe geotechnical

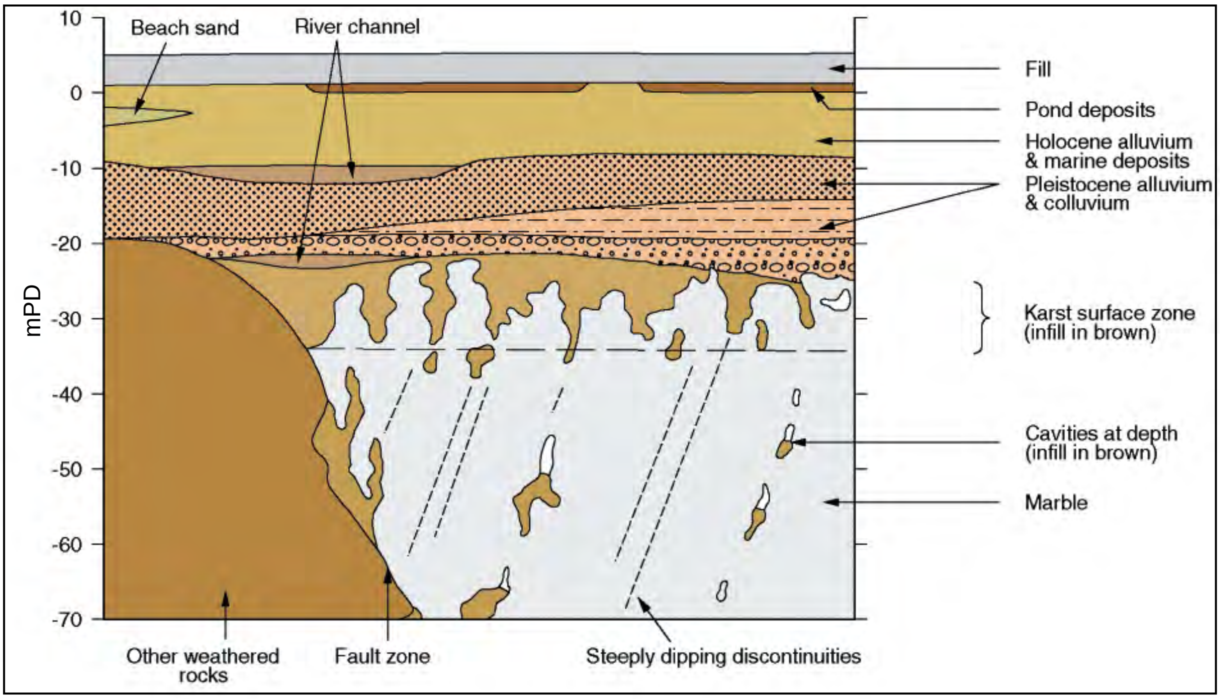


Figure 5.5.5 – Schematic section through buried karst beneath the Yuen Long floodplain (Fyfe et al., 2000)

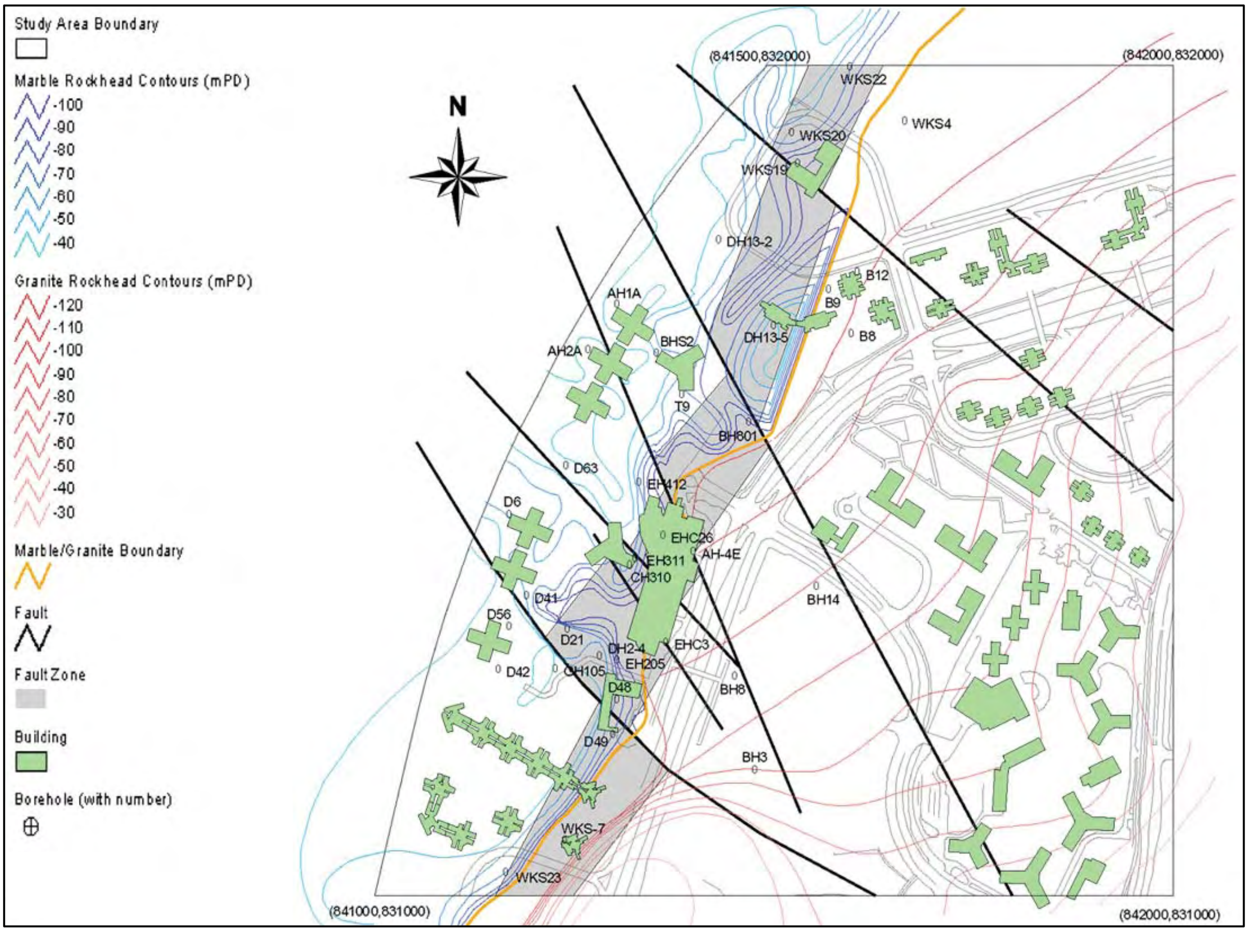


Figure 5.5.6 – Complex fault zone between marble and granite at Ma On Shan showing deepest weathering and karst solution within the fault zone (Sewell et al., 2000)

constraints (Sewell & Kirk, 2002). In one case a proposed residential Tower Block was abandoned where a complex succession of in fill deposits and collapse fragments was found to depths greater than 150 m. Fletcher *et al.* (2000) describe the complex ground conditions at this site and Wightman *et al.* (2001) describe the advanced ground investigation techniques that were employed (see case study in Section 6.5). Geophysical techniques such as gravity surveying subsequently proved useful in identifying the general extent of zones of deep weathering along the northern shore of Lantau Island (Kirk *et al.*, 2000) and where rockhead gradients were particularly steep (Sewell & Kirk, 2002; Figure 4.2.4).

Groundwater and Weathering

Groundwater flow can be significantly affected by the presence of dykes, faults and lithological contacts. Since dissolution is controlled by groundwater flow, such features can lead to considerable variations in the size and location of cavities. Preferentially weathered mafic dykes may act as aquicludes and thus

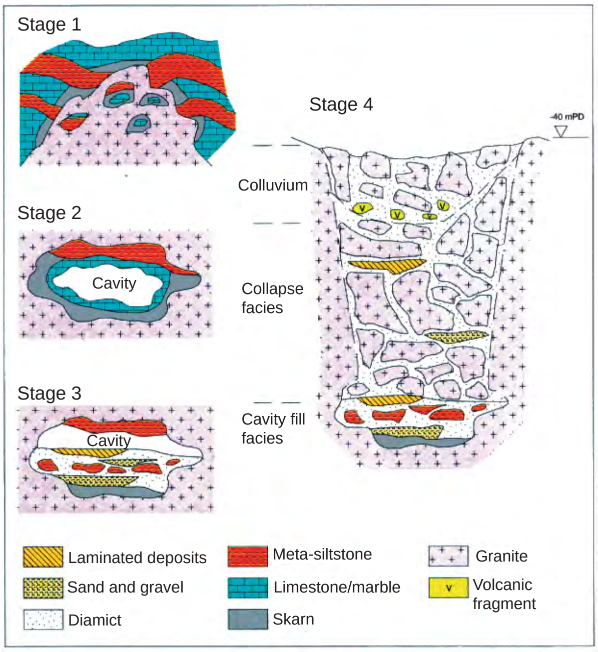


Figure 5.5.7 – Schematic representation of the development of karst deposits beneath Tower 5, Site 3, Tung Chung East Reclamation (Fletcher et al., 2000)

inhibit dissolution of the marble on one side of the dyke. Faults may allow dissolution to extend locally to significant depths (e.g. below -140 mPD). Marble adjacent to lithological contacts, and in particular faults, may also result in selective dissolution, e.g. broad, shallow, sinuous rockhead depressions along the thrusts at Lok Ma Chau, and deeper , linear rockhead depressions along more steeply inclined reverse faults at Yuen Long and strike slip faults at Ma On Shan (Campbell & Sewell, 2004).

Dissolution weathering is not significantly active today. However, man-made changes to the regional hydrogeological setting (i.e. drainage, dewatering, etc.), together with polluted or acidic groundwater , might cause an increase in dissolution activity.

5.5.5 Engineering Issues

The potential geotechnical constraints caused by karst features in the Yuen Long and Ma On Shan areas resulted in their designation as Scheduled Areas 2 and 4 respectively under the Buildings Ordinance with strict requirements for , amongst other things, ground investigation and design submissions as outlined in BD (1993). The areas of complex ground conditions at Tung Chung and Northshore Lantau related to faulting and weathered marble xenoliths have been included within a Designated Area. Guidelines on the approach required for ground investigation in this area are outlined in GEO (2004k) and BD (2004b).

The main geotechnical issues associated with karst features are shown in Table 5.5.2.

Geophysical techniques, which rely on the contrast in physical properties between the marble and the infill material, are a useful supplement to conventional site investigation techniques and have been used with some success in identifying zones of deep

Foundation Type	Main Geotechnical Issues
Bored Cast Insitu Piles	Uncertain or discontinuous support due to presence of compressible layers or cavities. Possibility of gross failure when loaded if pile is located on top of large cavity. Sudden loss of bentonite support (during boring) and dewatering.
Driven Piles	Pile buckling, deflection and tip damage and uncertain support, particularly on steeply sloping karst surface.
Shallow Foundations	Collapse or subsidence of ground surface due to upward migration of voids.

Table 5.5.2 – Geotechnical issues associated with foundations on karst

weathering, but success in identifying cavities is limited.

Variations in the type, extent and complexity of karst development reflect the interplay of several independent geological processes. The key engineering implications are considered in the 'method of karst morphology' as outlined in Chan (1994) and Chan & Pun (1994). Engineering geological input can assist in interpreting the ground conditions in three dimensions, based on the results of drillhole coring and any preliminary geophysical investigations that may have been carried out.

The initial geological model for a site can help to determine the potential feasibility of any geo-engineering works and help to optimise the ground investigations. Further development of the model during the investigation stage can help to determine the need for additional investigations. The 'method of karst morphology' provides a framework for the geological and engineering characterisation of sites in areas where pure marble is involved. The method may also be applicable to some areas where impure marble is found.

Engineering geological input is also useful where rocks occur that contain marble clasts or where the rock matrix contains subordinate quantities of calcium carbonate. The miss-identification of deformed marble clasts as marble beds or layers can have considerable implications as this would entail drilling 20 m into 'sound marble', as noted in BD (1993). GEO (2005c) provides clarification on this issue. The determination of pure or impure marble requires careful geological identfication and cannot be done on the basis of simple reaction to dilute hydrochloric acid. Such a test will also show a reaction in calcareous mudstone, for example in the Lok Ma Chau Formation, which is not subject to karst development. In complex ground conditions, inaccurate or inadequate descriptions and poor quality sampling can result in the miss-identfication of material and an incorrect geological model being adopted (Kirk, 2000). Similarly , poor recovery of silt-filled cavities in marble may be miss-interpreted as a weathered interbedded meta-siltsone and marble or tuff-breccia with marble clasts (Frost, 1992).

Rock Mass Classification of Marble

Most rock mass classifications in common use were developed for tunnels or under ground excavations (see Section 3.5.4) and do not include degree of dissolution as a key parameter. A marble rock mass classification system, based on the Marble Quality Designation (MQD), has been proposed by Chan (1994) and Chan & Pun (1994) to facilitate the zoning of pure marble rock masses for interpretation of the dissolution process and assessment of suitability for foundations. The MQD uses two main input parameters derived from drillhole records:

- RQD: Fracture state is related to dissolution as the fractures would have provided paths for water flow in the past. Fractures may also form preferentially near major cavities due to changes in stress related to the formation of the cavity.
- Marble core recovery ratio: The percentage of voids or dissolved marble with or without infilling indicates the degree of dissolution.

The marble rock mass can be subdivided into five classes (Class I to V) in accordance with the range of MQD values given in Table 5.5.3. Class I and II marble are considered at worst to be marginally affected by dissolution and are therefore likely to form sound founding strata. Classes IV and V marble reflect serious dissolution and are beyond doubt unsuitable as founding strata. Examples of the use of this system in the classification and interpretation of karst morphology are given in Chan (1994, 1996) and Chan & Pun (1994).

Marble Class	MQD Value (%)	General Description of Marble Rock Mass
I	75.1– 100	Very good quality marble essentially unaffected by dissolution and with favourable mass properties with few fractures
II	50.1– 75	Good quality marble slightly affected by dissolution or a slightly fractured rock essentially unaffected by dissolution
III	25.1– 50	Fair quality marble fractured or moderately affected by dissolution
IV	10.1– 25	Poor quality marble heavily fractured or seriously affected by dissolution
V	≤ 10	Very poor quality marble similar to Class IV but cavities can be large

Table 5.5.3 – Classification of marble rock mass based on MQD Values

5.6 SEDIMENTARY ROCKS

5.6.1 Introduction

Sedimentary rocks can be broadly divided into two groups:

- Clastic rocks – e.g. sandstones, formed from the accumulation of mineral or rock fragments (clasts) derived from weathering and erosion of pre-existing rocks, and subsequently transported and deposited by agents such as water or air.
- Non-clastic rocks – formed from biological and chemical precipitation, e.g. limestone.

This section considers clastic sedimentary rocks, which are primarily classified according to their grain size and comprise:

- conglomerates: gravel to boulder size (>2 mm)
- sandstones: sand size (0.06-2 mm)
- mudstones (siltstone and claystone): silt/clay size (<0.06 mm).

Non-clastic sedimentary rocks (marble) are discussed in Section 5.5.

Clastic sedimentary rocks from three separate time periods are present in Hong Kong (Sewell *et al.*, 2000). From youngest to oldest, these are:

Cenozoic
- Ping Chau Formation (youngest)

Mesozoic (post-volcanic/plutonic events)
- Kat O Formation
- Port Island Formation
- Pat Sin Leng Formation

Mesozoic (pre-volcanic/plutonic events)
- Tai O Formation
- Tolo Channel Formation

Palaeozoic
- Tolo Harbour Formation
- Lok Ma Chau Formation
- Bluff Head Formation (oldest)

The distribution of clastic sedimentary rocks in Hong Kong is shown in Figure 5.6.1.

The key engineering geological factors that influence the properties of the clastic sedimentary rocks include:

- Variations in rock material strength and durability, which are largely dependent on:
  - composition and properties of the constituent grains.
  - degree of diagenesis (e.g. compaction, cementation, leaching).
- Sedimentary structures (e.g. bedding) may affect rock mass properties by:
  - introducing planes of weakness.
  - acting as hydrogeological conduits or barriers.

- Potential vertical and lateral variability in material and mass characteristics over short distances.

Sedimentary rocks which occur within sequences of volcanic rocks (volcaniclastic rocks) are dealt with separately in Section 5.3. Marble, derived from metamorphosed limestone, and clastic sedimentary rocks that have been partly or completely metamorphosed, are covered separately in Sections 5.5 and 5.7 respectively because of their different engineering geological characteristics.

5.6.2 Engineering Geological Considerations

Bedding planes are primary sedimentary structures reflecting changes in the depositional environment. Therefore adjacent beds may comprise materials of significantly different composition, grain size and cementing properties, which in turn leads to heterogeneity in the engineering geological properties. These beds can vary greatly in thickness both laterally and vertically.

The wide range of geological age of the sedimentary rocks, from the Palaeozoic to the Cenozoic also influences the engineering geological properties as the older rocks have a higher likelihood of being affected by regional tectonic activities. Consequently bedding structures may be folded or tilted (see Sections 4.2 and 4.3). Where such structures are adversely orientated with respect to an engineering structure, stability and/or groundwater issues may arise. Furthermore, many sedimentary rocks deposited before the Mesozoic volcanic-plutonic activity have been partly or completely metamorphosed, and this has resulted in changes to their engineering geological properties (see Section 5.7).

Given the inherent variability of the environments of deposition, extrapolation of individual units between outcrops or drillholes requires knowledge of sedimentary environments to reflect adequately the possible range of engineering geological properties.

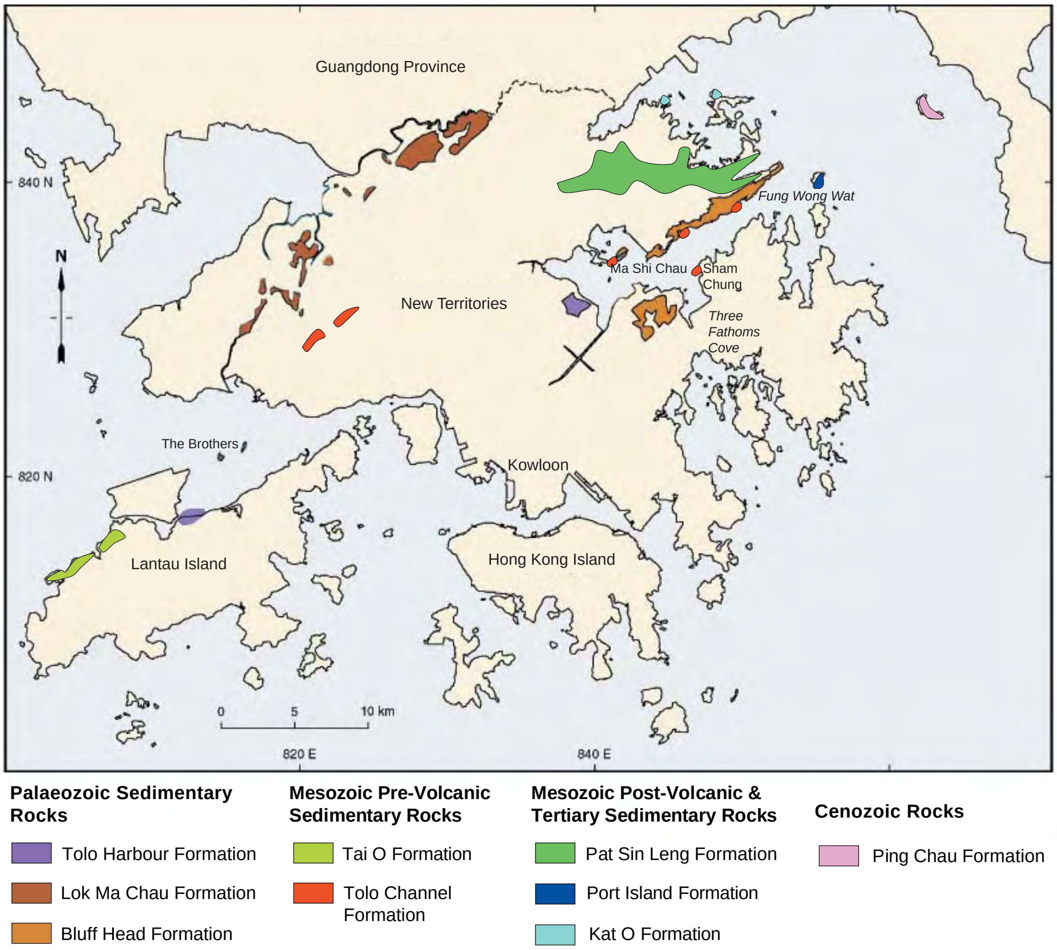


Figure 5.6.1 – Distribution of the clastic sedimentary and meta-sedimentary rocks in Hong Kong (after Sewell et al., 2000)

5.6.3 Material and Mass Characteristics

Due to the limited extent of the clastic sedimentary rocks in Hong Kong and their remote locations (Figure 5.6.1), little quantitative information exists on their weathering and engineering characteristics. Thus, the material and mass characteristics are only outlined in general terms.

One of the most important influences on strength and durability of clastic sedimentary rocks is quartz content, in both the clasts and matrix. Sandstones generally consist mostly of quartz, although calcareous sandstones (<50% calcium carbonate) also occur. Mudstones also contain fine quartz, but usually have a higher proportion of other less resistant minerals than sandstone. Another significant factor on the strength and durability of sedimentary rocks is the degree of diagenesis (i.e. lithification) to which the grains have been subjected. Most sedimentary rocks are typically well-lithified, having either inter-grown crystals, or grains that are joined together with cementing agents. Over time, weathering may decompose the cement and weaken the rock. Quartz cements are resistant to decomposition, whereas calcite cements weather relatively readily by dissolution.

The strength and durability of clastic sedimentary rocks can vary over a wide range depending on the above factors. Generally, well-indurated quartz sandstones are the strongest and most durable. In contrast, sedimentary rocks with calcite cement and poorly-indurated mudstones are the weakest and least durable. Overall, the clastic sedimentary rocks are typically more susceptible to weathering than the volcanic or plutonic rocks, due to their lack of an interlocking crystalline texture. Weathering is commonly heterogeneous resulting in stronger, more resistant strata interbedded with weaker, less resistant strata (e.g. sandstone interbedded with mudstone).

In hydrogeological terms, well-sorted, coarse-grained, but poorly-cemented, sedimentary rocks generally have a high primary permeability.

The classification used in Hong Kong for the material description of chemically decomposed rocks of plutonic and volcanic origin (GCO, 1988a) is not readily applicable to the sedimentary rocks as their original strength in the fresh state is not comparable to fresh granite/volcanic rocks. Alternative classification systems may be considered, e.g. BS 5930 (BSI, 1999).

Mass characteristics of clastic sedimentary rocks are heavily influenced by the spacing and continuity of bedding. The bedding characteristics may result in a high secondary permeability if the bedding planes are relatively open and continuous. Preferential groundwater flow paths and perched groundwater can develop at interfaces of significant contrasts in permeability (e.g. between conglomerate and mudstone). Bedding planes may facilitate the transport and deposition of infill within the apertures (see the Wu Kau Tang case study below). This may be exacerbated if the bedding planes are sub-parallel to the ground surface and are dilated due to stress relief at shallow depth.

The potential variability in material properties of the sedimentary rocks means that they are generally not suitable for aggregate.

Case Study - Wu Kau Tang Landslide

Landslides occurred in sedimentary rocks of the Pat Sin Leng Formation in highway cut slopes at Wu Kau Tang near Bride's Pool in the New Territories during 1986 and 1987 (Irfan & Cipullo, 1988). The rocks comprise conglomerates, sandstones, and siltstones (Sewell *et al.*, 2000), which near the site dip at about 20° to 35° to the North or North-northeast (Figure 5.6.2). The API indicated that the slopes in the area had a history of instability dating back to pre-1964 (earliest aerial photographs) in both the cut slope and natural terrain.

The ground investigation showed the cut slope is formed of sandstone, which was conglomeratic near the toe of the slope, becoming finer grained upslope. Occasional thin interbeds of mudstone are present, with the general dip being about 21° towards the road. A thin (<1.5m) layer of colluvium was present. Tension cracks were noted above both the cut slope and in the natural terrain. Two types of failure mechanism were noted, namely, the formation of a shear surface at the boundary of the colluvium and weathered sandstone, and slipping along polished, clay-infilled bedding planes. In the bedding plane-controlled failure a steeply dipping joint set, striking sub-parallel to the slope, acted as the rear release surface to the failure and may have allowed the build-up of cleft water pressure (Figure 5.6.3).

Figure 5.6.2 – Orientation of bedding planes in sandstone of the Pat Sin Leng Formation, at Wu Kau Tang Road (Irfan & Cipullo, 1988)

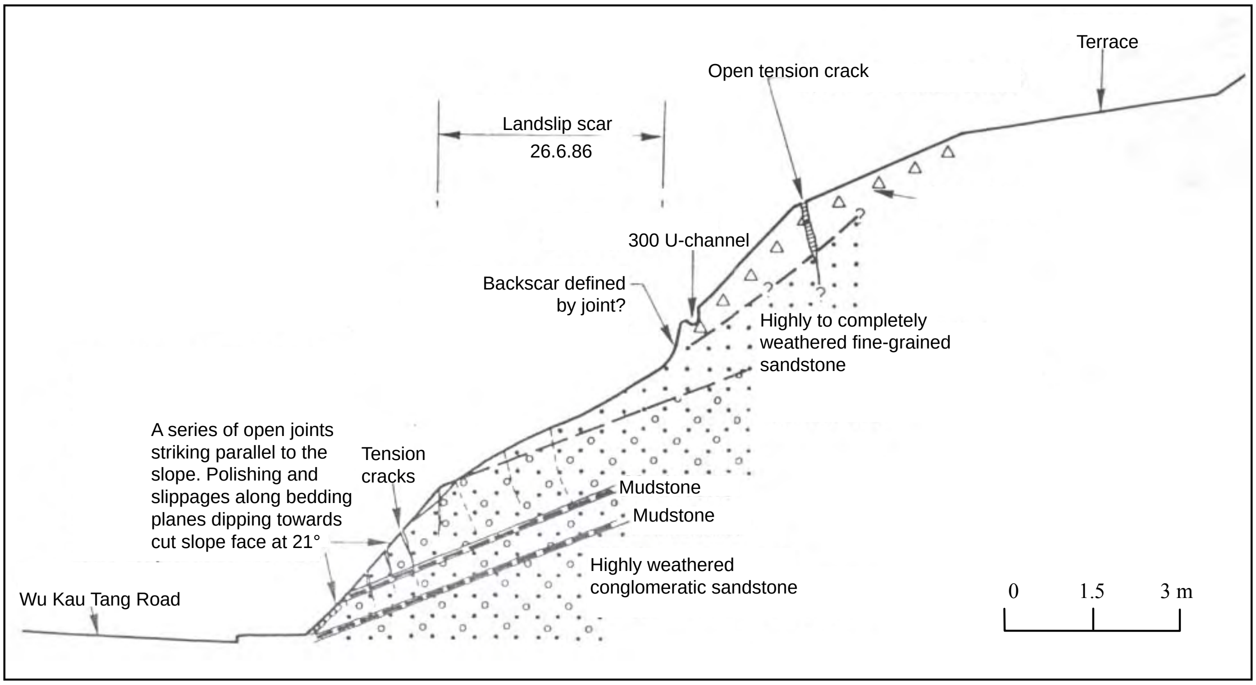


Figure 5.6.3 – Section through the 1986-87 landslide at Wu Kau Tang Road (Irfan & Cipullo, 1988)

5.7 METAMORPHIC ROCKS

5.7.1 Introduction

Metamorphic rocks can be broadly divided into two types: foliated and non-foliated. The degree of foliation can vary from strong to weak. Non-foliated metamorphic rock types include marble (see Section 5.5), hornfels and skarn (see Section 4.3).

5.7.2 Engineering Geological Considerations

The clastic sedimentary rocks and adjacent tuff of the northern New Territories have been affected by dynamic metamorphism to varying degrees depending on their location and original composition (Figure 4.3.1). The metamorphism also resulted in the alteration of limestone and dolomite into marble (see Section 5.5).

A key engineering geological effect of dynamic metamorphism is the development of foliation. Typically, foliation in the metamorphic rocks of the New Territories is inclined at low angles to the north or northwest (Sewell *et al.*, 2000). Where this foliation coincides with an unfavourable slope aspect and angle, instability can result (see the Lin Ma Hang Road case study below).

The rocks of the northwest and northern New Territories commonly exhibit extensive folding and faulting. Folds, with wavelengths of metres to tens of metres, occur in the Lok Ma Chau Formation (Figure 5.7.1). Larger scales of folding, with wavelengths of tens to hundreds of metres and more, and fold axial traces that extend for distances of hundreds of metres up to kilometres, have also been mapped.

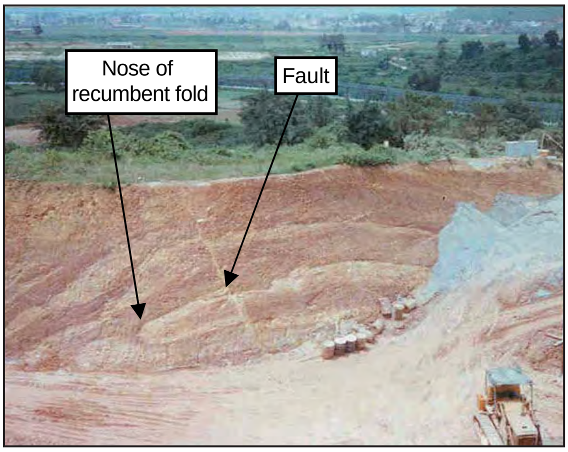


Figure 5.7.1 – View of slope at Lin Ma Hang Road showing complex structural geology of the meta-sedimentary rocks in the Lok Ma Chau Formation, including recumbent folding and a minor fault across the slope

The variable effects of metamorphism on interbedded sedimentary rocks, superimposed with the differing effects of weathering, can give rise to complicated weathering profiles and varying geotechnical parameters. Greenway *et al.* (1988), when reviewing existing cut slopes, note that steep (over 60 °) stable slopes of moderate height (20 m) could be achieved in massive or very thickly-bedded sandstones where foliation dipped into the slope. Where a succession of interbedded phyllite and meta-sandstone occurred, the stable slope gradients were shallower, even when foliation dipped into the slope. Where slopes were predominantly of phyllite and orientated in general alignment to the strike of the slope, the gradient was much shallower, being close to the dip of the foliation.

Metamorphic rocks exhibit a range of strength. Point load tests (Is_{50}) carried out for a tunnel project in the Lok Ma Chau Formation, indicated that the rocks were moderately strong to extremely strong (McFeat-Smith *et al.*, 1985). Point load tests carried out normal to the foliation on moderately decomposed meta-siltstone and meta-sandstone (GCO, 1990a), gave a maximum Is_{50} point load strength of 3.5 MPa, i.e. strong.

The classification used in Hong Kong to describe chemically decomposed rock materials of plutonic and volcanic origin (GCO, 1988a) is not readily applicable to most metamorphic rocks, due to their fissility (e.g. phyllite) or high degree of alteration (e.g. skarn). Alternative classification systems may be considered, e.g. BS 5930 (BSI, 1999).

5.7.3 Case Studies

Lin Ma Hang Road Landslide

The slope engineering aspects of this landslide with respect to adverse discontinuities are discussed in Section 6.4.4. The following discussion focuses on the metamorphic effects.

A landslide occurred in and above a cut slope within the Mai Po Member of the Lok Ma Chau Formation on Lin Ma Hang Road in the northwest New Territories. The landslide involved the displacement of about 2,100 m^3 of material by about 1.5 m.

The meta-sedimentary rocks at the landslide site comprise phyllite, meta-siltstone and meta-sandstone. Foliation is present in all the rock types but is most developed in the phyllite which is also the most common rock type. The foliation planes exposed in trial pits were often open (probably due to stress relief) and in filled with soft to firm brown clayey silt. Foliation planes were orientated at about 34°/300°, roughly sub-parallel to the slope aspect. The landslide is located in an area where previous movement was inferred from API (see Section 6.4.4). The rock mass is also more deeply weathered at this locality.

Shear box tests were carried out along and perpendicular to the foliation. Along the foliation the shear strength was relatively low, with residual parameters of $c' = 0$ kPa, $\phi' = 16°$, in contrast to the peak values measured perpendicular to the plane of foliation where $c' = 0$ kPa, $\phi' = 69°$. These results may represent extreme upper bound and lower bound values but they illustrate the effect that orientation of these weak planar interfaces can have on stability. The foliation surface was observed to be 'shiny' due to growth of platy minerals such as mica which reduce the shear strength along these planes as they weather to clay.

The various meta-sedimentary rocks at the site were found to weather differentially. Also, there was evidence to suggest that some of the rocks have a relatively higher susceptibility to weathering in engineering time scales when exposed by man-made cuttings, or along existing failure surfaces where the agents of weathering can penetrate. Such an accelerated weathering process may have been a contributory factor to the landslide, in addition to the effects of stress relief, over-steepening of the ground profile and elevated groundwater pressures during heavy rain.

Table Hill Reservoir

Greenway *et al.* (1988) reported that the effect of foliation on the slope design for an access road and reservoir resulted in the realignment of the road in order to reduce the height of the cut slopes. Realignment of the reservoir, which was orientated unfavourably with respect to slope stability, was not possible as its location had been fixed to minimise visual impact.

Stability conditions at the site were further complicated by intense folding and faulting and the interbedded nature of the meta-sandstone and phyllite, the latter preferentially weathered to a clayey silt. During construction, the dip of the foliation surfaces were found to be variable, ranging from 27° to 55°. Most of the foliation planes, particularly in the more intense weathered zones, contained clay and silt infilling resulting from weathering of the mica-rich layers. The possibility of strain softening along the foliation planes due to further weathering with time was also considered. It was noted that the weakly cemented, highly weathered phyllitic siltstones became friable, easily erodible soils when exposed to the atmosphere and rainwater.

One favourable aspect of the foliation with respect to stability was that the surfaces were not very persistent and were wavy. Hence failures on temporary slopes were localised and sliding type failures on foliation planes gentler than 50° to 55° were rare.

5.8 SUPERFICIAL DEPOSITS

5.8.1 Introduction

This section summarizes the engineering geological characteristics and geotechnical properties of superficial deposits, both onshore and offshore.

In engineering terms, superficial deposits are soils. The strength of superficial deposits varies from very soft to very stiff, and their consistency from very loose to very dense. Weakly cemented deposits also occur. The geotechnical properties of superficial deposits, e.g. shear strength, permeability, compressibility and susceptibility to internal erosion, depend on factors such as particle size distribution, clast composition and stress history, all of which can vary significantly both laterally and vertically within a particular superficial unit. The way in which these factors vary is related to the processes operating during their deposition and their subsequent geological history. Understanding these processes is a pre-requisite to formulating the geological and ground models that provide the basis for predicting engineering behaviour.

Fyfe *et al.* (2000) provide detailed geological coverage, particularly concerning the Quaternary palaeo-environments. Caution should be exercised when using the published geological maps and memoirs for engineering purposes because they have been compiled on the basis of stratigraphy rather than properties of materials. Furthermore, superficial deposits are only shown on the 1:20,000-scale geological maps if they were estimated to be greater than two metres thick.

Superficial deposits in Hong Kong can be broadly grouped into terrestrial and marine based on their depositional environments. However, the engineering geological characteristics and geotechnical properties are more conveniently discussed under three categories, namely:

- terrestrial deposits,
- Pleistocene marine deposits, and
- Holocene marine deposits.

Figure 5.8.1 presents a schematic representation of how the superficial units shown on geological maps and as described by Fyfe *et al.* (2000) fit into the framework of these three broad categories.

In general, the energy of the environment of deposition and erosion, and the degree of lateral variability in sediments, are primarily related to slope gradient, relative location of sea-level and degree of channelisation of surface, tidal and sub-tidal flows. Variations in rainfall and wave action also influence the degree of erosion and deposition. At any one location, all these factors have changed with time, which can lead to a high lateral and vertical variability of the soils. At many sites, the main geotechnical issue is often the prediction of spatial variability based on relatively few ground investigation data and engineering geological interpretation, rather than determination of the geotechnical properties of a few soil samples based on laboratory testing.

5.8.2 Quaternary Palaeo-environments

The superficial deposits essentially date from the Quaternary Period. The successive rises and falls in global sea-level during the Quaternary glacial cycles (see Section 4.5.6) had little effect on Hong Kong until the Holocene (approx 11,500 BP). Before that time, i.e. during the Pleistocene, the sea only transgressed over localized areas along the lowest-lying drainage routes, and most of the terrain was sub-aerial (see Figure 5.8.1). By about 6,000 BP the present-day marine area was fully established and the submerged valleys and their terrestrial sediments were then progressively covered with Holocene marine deposits.

During the Pleistocene, terrestrial deposits were laid down more or less continuously from the colluvial hillsides to the alluvial plains which now lie below present-day sea-level. Consequently, the location of the present-day shoreline has no relevance to the Pleistocene sedimentary processes and deposits. At elevations above present-day sea-level, erosion and accumulation of terrestrial sediment have been more or less continuous throughout the Pleistocene to the present day.

5.8.3 Terrestrial Deposits

Overview

Terrestrial deposits, shown in Figure 5.8.1, are mostly brownish in colour, indicative of oxidation in a generally well-drained subaerial or fluvial depositional environment. Lacustrine deposits, organic-rich deposits and strata that have been waterlogged for long periods in oxygen-poor conditions are usually greyish in colour. Where oxygen levels in the soil have fluctuated over long periods of time, brown and grey mottling may occur.

The variety of grain size, sedimentary structures and morphology of the sedimentary units reflects the very wide range of energy and modes of sediment transport and deposition. Patterns of sediment erosion, transport and deposition are dominated by gravity in the mass wasting products on the hillslopes, by fast flowing, intermittent streams further down slope, and by meandering fluvial networks and occasional shallow lakes in the flatter-lying more distant terrain. Re-erosion of sedimentary deposits adds to the complexity of the resulting accumulations. The sediments are described below under broad headings reflecting their principal characteristics.

The Pleistocene Chek Lap Kok Formation forms the bulk of the terrestrial deposits, as shown in Figure 5.8.1. However, also included is the Fanling Formation, the present-day continuation of the Chek Lap Kok deposits onshore, and the Tung Chung Formation, a local expression of very early deposits with many related characteristics. Processes of hillslope erosion and deposition are discussed in Section 4.5.

The 1:20,000-scale geological maps (1985–1995) identify both talus and debris flow deposits. However, a more general term such as ‘colluvium’ is preferred for non-talus deposits because the term ‘debris flow deposit’ implies a specific origin, and most colluvium will have a variety of origins, from soil creep through to landsliding.

Colluvium

The engineering geological characteristics of colluvium depend on numerous factors. Primary controls include:

- the rock type, structure and degree of weathering of source material,
- the processes of transportation and deposition,
- the age of the deposit and weathering history, and
- subsequent modifying factors such as groundwater conditions.

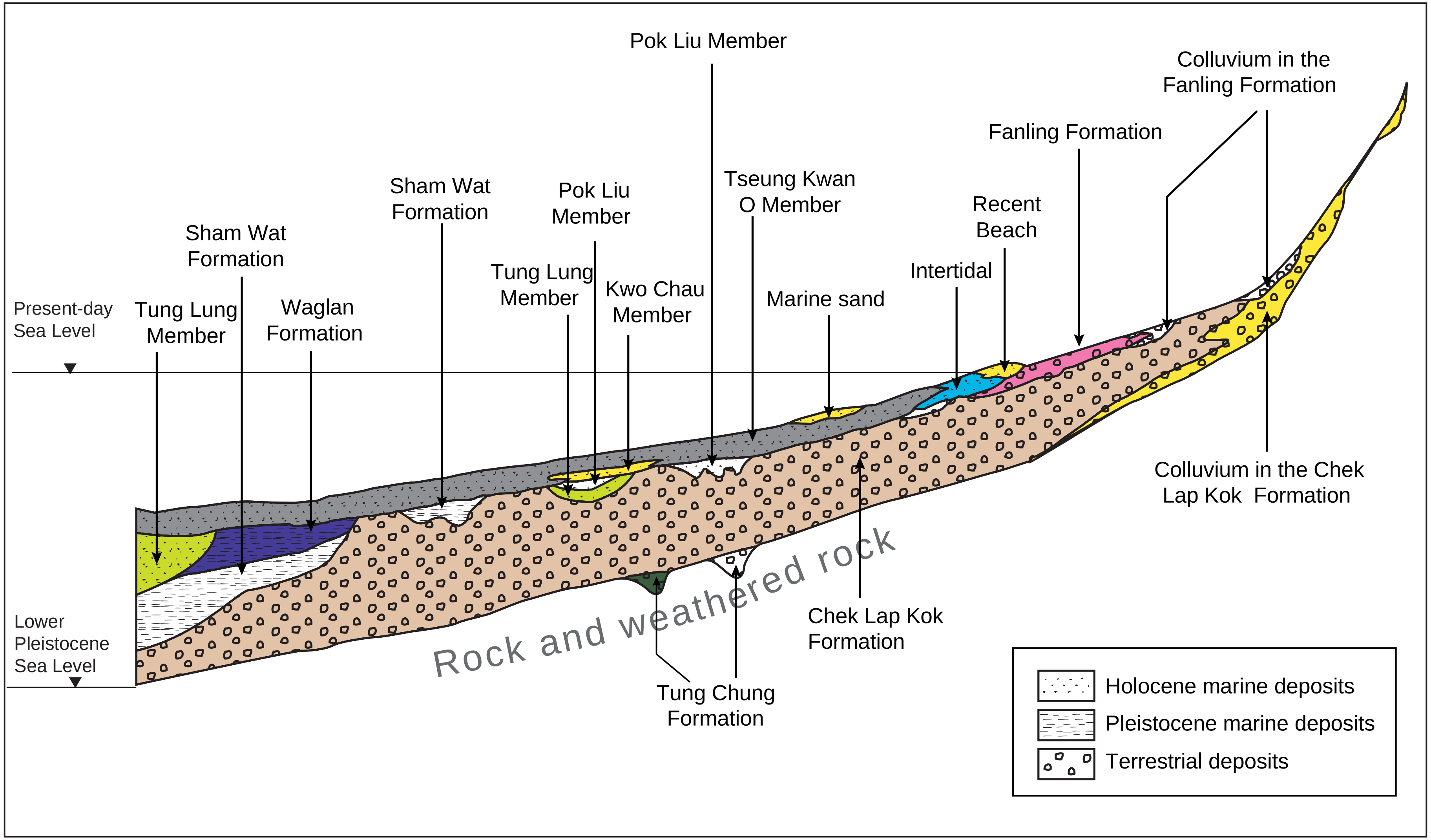


Figure 5.8.1 – Schematic representation of the stratigraphy of Hong Kong superficial deposits (after Fyfe et al., 2000)

Colluvium is commonly a heterogeneous material consisting of rock fragments (ranging in size from several millimetres to a metre or so) supported within a sandy, clayey, silty matrix. Some generalisations are possible :

- the larger boulders found in colluvium are generally derived from coarse grained bedrock because of its widely-spaced joints,
- granitic colluvium is generally thicker than volcanic colluvium,
- older colluvium tends to have been eroded from drainage lines and is therefore now more likely to be found on interfluves, and
- older colluvium tends to be stiffer or denser and displays more weathering of the clasts.

On Hong Kong Island, the Mid-Levels Study (GEO, 1982) identified three classes of colluvium based on morphology and soil characteristics. It was found that in general, soil density and strength increased with age, as did the degree of weathering of the boulders. The three classes of colluvium may represent separate climatic periods of deposition. Lai & Taylor (1984) also differentiated colluvium of different ages and extended the classification to include the source materials (i.e. bedrock lithology).

Based on the foregoing studies it is considered likely that colluvium throughout Hong Kong was formed during multiple events spanning from the mid-Pleistocene to the present, albeit perhaps, with increased intensity of mass wasting during wetter interglacial periods. Lai & Taylor (1984) report and illustrate a rare confirmation of a Holocene age for a colluvial deposit that was found overlying soft Holocene marine deposits. Sewell & Campbell (2005) report on a trial programme of age dating of relict landslide scars, which indicates ages of up to 50,000 years. There was also some evidence to suggest that some landscape surfaces were up to about 300,000 years in age.

Absolute age is important for certain purposes, such as calculating risk from natural terrain landslides (see Section 6.2). Relative age is also useful, e.g. subdividing colluvial deposits into units related to their relative age as inferred from detailed API or the degree of weathering of the clasts.

Depending on the degree of saturation, iron cementation of the matrix can significantly add to the material strength (Ruxton, 1986). Occasionally, kaolin veins and to a lesser extent, kaolin-rich zones occur within Pleistocene colluvial deposits. They typically do so within the matrix, especially towards the base of these deposits, and around some clasts (Ruxton, 1986). It is considered that these occurrences are weathering related (Campbell & Parry, 2002).

The engineering properties of colluvium are likely to be influenced by the proportion and strength of cobbles and boulders, particularly if that proportion exceeds about 30% by volume (Irfan & Tang, 1993). However, the cobbles and boulders range in size and display varying states of weathering. This makes it difficult to estimate their contribution to overall mass strength. The variable composition of the matrix is reflected in the relatively wide range of the strength parameters given in Geoguide 1 (GEO, 1993). It is difficult to ascertain mass properties from small drillhole samples and colluvium is best examined in trial pits or exposures.

Relatively recent colluvium can contain moderately- to slightly-decomposed boulders which may be several metres across. Where these are encountered in drillholes, difficulties may arise in the interpretation of rockhead level (e.g. where the drillhole terminates within the boulder) or bedrock lithology (e.g. where the lithology of the boulder and the underlying bedrock are different and where the boulder and bedrock are separated by only a thin layer of saprolite). In such cases, engineering geological knowledge of the site setting and critical interpretation of other available data can facilitate the development of a representative geological model.

The hillside setting of colluvial deposits corresponds to high surface runoff and potentially to major groundwater throughflow at depth. Depending on the relative proportion of the two, surface and sub-surface erosion can occur. The permeability contrast around unweathered boulders promotes the development of soil pipes (see Section 4.4.6). Preferential drainage paths can also develop along the base of colluvium, and streams on areas of shallow bedrock commonly disappear on entering areas of colluvium. As a result, the build-up of transient perched water pressure in colluvium during intense rainfall is common (GCO, 1982).

Talus

Talus comprises angular rock fragments derived from steep outcrops and cliffs. Although deposits visible on hillsides are probably of relatively recent age, it is possible that older deposits exist at depth within the blanket of colluvial deposits. The size of individual fragments, and therefore potentially the size of voids, varies considerably and is mainly dependent on the joint spacing of the parent bedrock. The voids within these deposits are commonly unfilled.

The extent of talus in Hong Kong is much less than that of colluvium, but where present, talus can have significant engineering consequences. Large blocks of rock can make the design and construction of foundations difficult, and buried talus can provide preferential groundwater flow paths. Problems associated with buried talus are analogous to the problems encountered with bouldery colluvium beneath a fill slope at Tai Po Road, as described in Section 5.9.

Alluvium

Colluvium interdigitates with and grades into alluvium where hillslopes intersect valleys. Interstratification of these deposits may be evident in vertical section or drillholes, though they are unlikely to persist laterally.

Where fluvial conditions exist, these are associated with finer grained alluvial floodplain deposits and networks of more granular channel sediments, with successive layers built up as streams meandered and coalesced. Alluvial deposits are commonly partly sorted, channelised and cross-bedded with successive cross cutting. Where the alluvial deposits are extensive, such as the Chek Lap Kok Formation, this high degree of small-scale sedimentary variability, when considered over a large area, can be sufficiently regular that a single set of soil parameters can be adopted for design of larger-scale engineering works. That is, despite the small-scale complexity of the geological model, the engineering behaviour of the ground can be greatly simplified in the ground and design models (see Section 6.8).

Issues of engineering concern include the extent and rate of consolidation settlement and the potential for differential settlement. The permeability of the alluvium *in situ* is critically important. It is usually the case that laboratory oedometer tests underestimate the rate of settlement, partly because they are unable to take account of the presence of fine laminae and beds of fine sand. Alluvial clay can also cause stability problems in embankments, seawalls, trenches and deep excavations.

Beggs & Tonks (1985) concluded that the alluvial sediments in the northwest New Territories could be divided into two categories with significantly different engineering properties. The younger Holocene deposits (part of the Fanling Formation) are generally soft and normally consolidated, whereas the older Pleistocene deposits are stiff and over-consolidated. Table 5.8.1 provides some indication of the type of variation between the engineering properties of older and younger alluvium. However, the engineering characteristics of the alluvial deposits are very site specific.

An important feature of the alluvium now offshore is its upper surface, which was subaerially exposed immediately prior to the Holocene marine transgression. Two attributes of this surface are relevant to the design and construction of engineering works. Firstly the surface is channelled, resulting in an uneven base to the overlying soft Holocene marine mud. Secondly, the surface commonly has a subaerially weathered crust that facilitates the placement of fill material for reclamations, etc., without the risk of penetration into the underlying weaker material (Endicott, 1992). Section 6.8 provides details of the investigation and construction of the airport reclamation at Chek Lap Kok Island, including an account of the distribution and nature of this weathered crust, overconsolidation ratio of the alluvium, etc. Lo & Premchitt (1998, 1999) give a detailed account of the consolidation properties of the Chek Lap Kok Formation. Included within the alluvium now located offshore are sediments that were laid down in salt water or brackish water (Yim, 1992).

Section 6.10 discusses the large deposits of sands in old alluvial channels that have been dredged for use as fill material.

Lacustrine Deposits

Localised lacustrine deposits occur in some areas of the flat-lying terrain in the northern New Territories. These were formed by ponding of late Pleistocene rivers and streams, resulting in the accumulation of fine-grained sediments. The yellow-brown sediments commonly contain layers of grey silty clay rich in organic matter. These sediments tend to be soft, plastic, normally consolidated and relatively compressible.

Engineering Properties	"Old Alluvial" Clay	"New Alluvial" Clay	Alluvial Silts	Alluvial Sand
PI (%)	20 - 55	20 - 35	15 - 33	10 - 22
LL (%)	40 - 80	38 - 60	32 - 60	22 - 38
%Clay	20 - 50	20 - 45	5 - 20	0 - 20
%Silt	40 - 70	30 - 50	30 - 70	10 - 30
%Sand	0 - 50	10 - 50	15 - 55	50 - 95
c' (kPa)	5	0	5	0
Φ(°)	28	24	30	36
Su (kPa)	30	30	-	-
c_v (m^2/year) Range	1.0 -10.0	1.0 - 20.0	8.0 - 40.0	-
Average	6.0	10.0	20.0	-
Cc (1+e_0) Range	0.05 - 0.15	0.08 - 0.24	0.05 - 0.28	-
Average	0.09	0.14	0.10	-
m_v($x10^{-4}$) m^2/kN Range	0.9 - 4.1	1.6 - 6.0	0.9 - 7.5	-
Average	1.8	3.5	2.5	-
C_{α}	0.002	0.008	0.014	-

Table 5.8.1 – Summary of alluvial material properties in Yuen Long area (after Beggs & Tonks, 1985)

Depression Infill

The Tung Chung Formation (see Figure 5.8.1) comprises bouldery silts and sands that are only present in deep localised bedrock depressions, probably karst-related, in the Tung Chung-Brothers Islands area north of Lantau (see also the discussion and example in Section 6.5).

In many respects, these deposits are similar to the colluvium found elsewhere in Hong Kong. The type section at Tung Chung, for instance, consists of completely decomposed, rounded to sub-rounded boulders supported in a matrix of poorly sorted brown sandy silt (Fyfe *et al.*, 2000). These materials were originally mistaken for colluvial deposits. However, detailed ground investigations and logging have indicated a depositional environment of deep, steep-sided doline depressions where the characteristics and distribution of the infilling sediment differ from that normally found in colluvial deposits. Delineation of these deposits has required extensive and specialised methods of ground investigation (Wightman *et al.*, 2001) and specialised engineering geological input (Kirk, 2000).

5.8.4 Pleistocene Marine Deposits

The Pleistocene rises in global sea-level associated with Quaternary interglacial periods only affected the lowest lying areas within the main drainage basins. Two events of sea-level rise, around 120,000 BP and 80,000 BP were responsible for the deposition of the Sham Wat and Waglan Formations respectively (Fyfe *et al.*, 2000). These predominantly silt and clay marine sediments are confined to localised channels in the eroded surface of the Chek Lap Kok Formation. The importance of these deposits is that they are deeper and firmer than Holocene marine clay and the organic content of the intertidal sediments, particularly the Sham Wat Formation, can give rise to gas blanking of seismic records (see Section 6.8).

5.8.5 Holocene Marine Deposits

Overview

The various deposits laid down as a result of the Holocene marine transgression are shown schematically in Figure 5.8.1. As the sea-level rose, it first flooded the drainage lines, which became inter-tidal creeks with deposition of basal sandy sediment, followed by organic-rich laminated silts (the Tung Lung and Pok Liu Members respectively, possibly late-Pleistocene in age). About 10,000 BP the rising sea flooded over the entire late Pleistocene alluvial plain. Tidal currents winnowed the surface of the old alluvium leaving transgressive granular deposits (the Kwo Chau Member). The sea continued to rise, and as seabed currents diminished, soft grey mud was deposited which blankets most of the present offshore area, commonly to a depth of about ten metres (the Tseung Kwan O Member).

Material Properties	Marine clay	Marine silt
PI (%)	20 - 40	10 - 25
LL (%)	35 - 65	25 - 40
%Clay	20 - 50	5 - 20
%Silt	45 - 75	30 - 60
%Sand	5 - 35	20 - 50
c' (kPa)	6	0
Φ(°)	20	28
Su (kPa)	5 - 30	5 - 30
c_v (m^2/year)		
Range	0.6 - 0.4	3.0 - 13.0
Average	1.3	4.5
Cc (1+e_0)		
Range	0.11 - 0.26	0.11 - 0.25
Average	0.23	0.20
m_v ($\times10^{-4}$) m^2/kN		
Range	2.0 - 8.0	2.0 - 12.0
Average	5.6	5.5
C_α	0.008	0.004

Table 5.8.2 – Comparison of engineering properties of clayey and silty Holocene marine deposits in the Yuen Long basin area (after Beggs & Tonks, 1985)

As the sea rose further, it reached the late Pleistocene break of slope where the alluvial plain gave way to the colluvial slopes. From this point on the rising sea began eroding into the steeper hillslopes, washing fines offshore and leaving coarser granular material as beach deposits. By about 6,000 BP the sea stopped rising, then dropped again by about two metres to leave today's shoreline. Intertidal deposits were laid down in the small estuaries around water courses. In the flat-lying terrain of the northwest New Territories the slight drop in sea-level left areas of the soft marine deposits inshore where they are now covered by a thin layer of recent alluvial sediment. Offshore, strong tidal flows in constricted areas locally resulted in sand deposits, including comminuted shell sands (see Figure 5.8.1).

Soft Marine Deposits

The soft marine mud of the Tseung Kwan O Member has fairly similar geotechnical properties throughout Hong Kong and previous documented work can be useful in providing indicative design parameters (e.g. Yeung & So, 2001). However, there are variations in particle size distribution, especially closer to the shoreline, and these can affect material properties. Table 5.8.2 shows the range in parameters between clayey and silty marine mud from the Yuen Long basin. At a more detailed level, studies of the chemistry and microfabric, such as those of Tovey (1986), provide an insight into the causes of variations in geotechnical properties.

The main engineering concern with the soft marine mud is its compressibility and resulting settlement under imposed loads. Both the rate and the amount of settlement are important. Standard methods of field and laboratory testing can provide reliable predictions of settlement; however, two additional engineering geological factors need to be considered:

- The rate of settlement can be significantly affected by thin laminae of coarse silt or fine sand, which are common, and which can act as drainage paths for release of excess porewater pressure (see Section 6.8).
- The total amount of settlement depends on the thickness of the mud, which is greater in areas of irregular pre-Holocene drainage channels. Offshore, delineation of the pre-Holocene drainage channels using seismic surveys can be problematic because of acoustic blanking due to gas bubbles in the sediments (Figure 5.8.2). This is especially associated with the Pok Liu Member (see the discussion in Section 6.8).

Recent anthropogenic mud containing sewage also gives rise to gas blanking within sheltered areas of the harbour where the lack of seabed currents or tidal flows allow the mud to accumulate.

In the last century, large areas of soft Holocene mud were excavated in the northwest New Territories to form ponds for rearing fish and ducks. The excavated soil was used to form bunds separating the

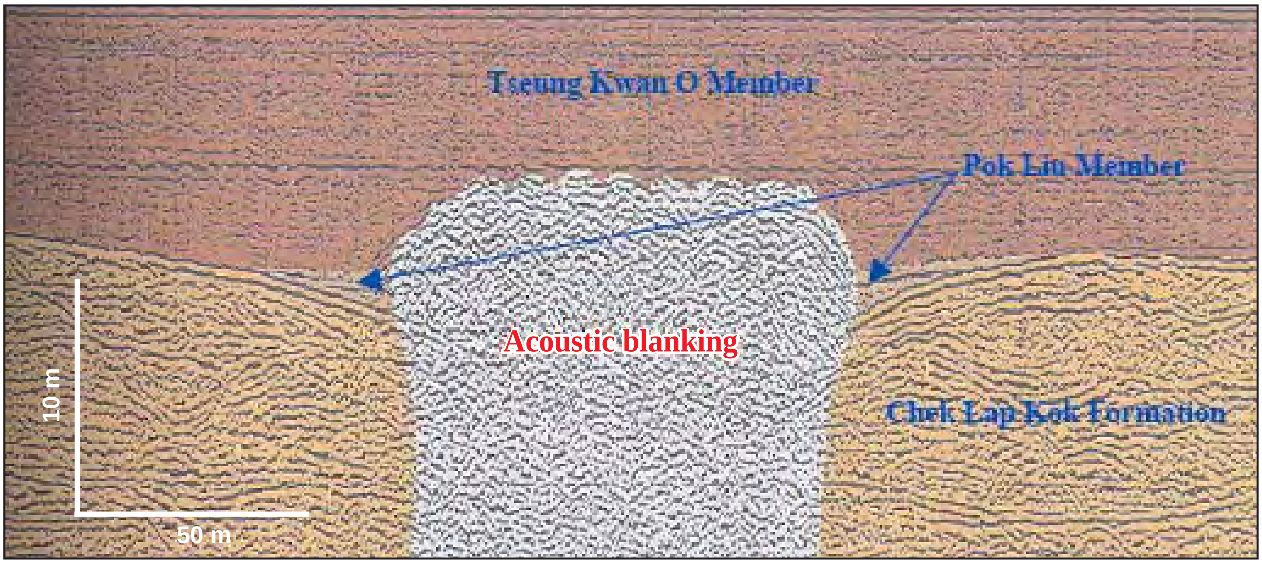


Figure 5.8.2 – Seismic boomer profile showing acoustic blanking caused by gas associated with an intertidal channel (after Fyfe et al., 2000)

ponds. Portions of these pond areas are now being redeveloped for low-rise housing and infrastructure. The geotechnical characteristics of this ground need special investigation as the natural soil structure and properties have changed significantly and layers of extremely soft, organic-rich sediment have accumulated in ponds.

5.9 MADE GROUND

5.9.1 Introduction

Although made ground exists in many different engineering contexts, almost all of it is encountered in two types of situation:

- reclamations and fill platforms on low-lying, coastal or near-shore terrain
- development platforms and roads on sloping terrain

In both situations, the performance of the made ground is very dependent on the nature of the fill and on the method of construction. Although made ground was not formed by natural processes and therefore does not possess the same patterns of material and mass characteristics found in natural ground, many engineering geological principles are relevant to the assessment of this material. Similarly, whilst the use of a geological model approach is more limited than in cases of natural ground, it is still applicable, notably in respect of engineering geological aspects of the natural topography and ground conditions prior to filling.

Although reclamation is by far the most common example of made ground (more than 60 km^2 of the land in Hong Kong is reclamation), the unpredictability of made ground on hilly terrain is much more of a concern because of the potential consequences of failures. Coastal reclamation is generally in relatively stable environments, although some failures have occurred (Blower *et al.*, 1993) and extensive reclamation may affect the regional groundwater regime (Jiao, 2000b). In contrast, fill slopes on hillsides are typically in meta-stable environments, and the water table and groundwater flow can vary rapidly and dramatically, which may give rise to potential instability if groundwater control measures are inadequate.

After the catastrophic fill slope failures at Sau Mau Ping in 1972 and 1976 (see Section 5.9.4), design and construction of fill slopes began to improve significantly. Discussion in this section is essentially directed at the older areas of made ground on sloping terrain where stability and other problems are more of a concern. Similarly, it is the made ground of older reclamations and inland site formations on low-lying terrain that are dealt with in this section.

Issues associated with reclamations are addressed separately in Section 6.8, while Section 6.3 covers some issues related to site formations. The problems associated with made ground that has become contaminated by industrial and other activities are covered in Section 6.9.

5.9.2 Historical Reclamations and Fill Platforms on Low-lying Terrain

General

Reclamation has been carried out widely in Hong Kong. The oldest reclamations are located along the original coastline of north Hong Kong Island and around Kowloon, with later reclamations located progressively further from the central harbour. The general areas of reclamation are well known (e.g. Fyfe *et al.*, 2000).

Nature of the Made Ground

Where older reclamations were extended, the later works may incorporate earlier structures such as concrete or masonry sea walls. Domestic and industrial waste may also occur in some of the older reclamations, and the engineering behaviour of this waste material is likely not only to be highly variable, but also to change over time. However, the presumption that the greater the age of a reclamation the greater the variability of the fill material might not always be justified. One reason being that when the earliest reclamations were formed, natural material from borrow areas was more readily available than construction and demolition material now commonly used.

Construction Techniques

The lower layers of reclamations are placed below water and if the fill is variably graded there is the potential for segregation of material. While the portion of made ground above the water table may be compacted, or simply becomes compacted through usage, below the water table this is unlikely. Many earlier reclamations were formed on top of reasonably competent nearshore material, e.g. beaches. However, later reclamations, further from the original shoreline, were formed on the soft Holocene marine deposits. The process of end tipping of fill into deep water in the past resulted in many instances of displacement of soft marine mud forming 'mud waves' (Endicott, 2001). Subsequent excavations into such reclamations have occasionally shown that the lower layers of fill material have sunk into and displaced the mud, thus posing additional difficulties with certain types of basement works and tunnels.

Potential Problems

The grading of filling material is an important characteristic of made ground because the presence of voids can provide the opportunity for long-term downward migration of finer material, with the resulting upward migration of voids, possibly even reaching the ground surface. This can be particularly important where layers of unblinded and irregularly sized rockfill have been incorporated into the formation of made ground.

The nature of the made ground in reclamations can cause problems during new construction works, e.g. the presence of large boulders, dumped tyres and pieces of reinforced concrete, could pose difficulties during piling operations. Furthermore, gases derived from decomposed organic materials within the fill, can be problematic in tunnels and excavations.

Excavations for basements and tunnels require dewatering, and special precautions are needed to minimise settlement in adjacent areas. Settlement occurs by a combination of self-weight compaction and washing out of fines by groundwater flowing towards the excavation. Consequently, ground investigation outside the site to characterise the made ground and to assess the potential for settlement may be required.

5.9.3 Development Platforms and Roads on Sloping Terrain

General

An understanding of geomorphological processes, associated regolith types and hydrogeology is important when constructing or investigating made ground on hillslopes. Reference should be made to Section 4.5, which discusses these processes.

Nature of the Made Ground and Associated Hydrogeology

Large areas of sloping terrain in Hong Kong have been modified by cut and fill activities. The material properties of the fill can be inferred to some extent, from the geology in the cut area. For sites where the excavated material was used as fill it may be problematic to differentiate the fill from *in situ* material. Therefore, investigation using trial pits that give relatively large areas of exposure should be considered. In contrast, development platforms that have been formed with imported fill can contain geological material quite different from that elsewhere on the site.

The full three dimensional extent of hillside fill platforms may be difficult to determine if the made ground pre-dates the earliest aerial photographs. Nevertheless, aerial photograph interpretation can usually provide a measure of the plan area of such features, while the geomorphology of the surrounding terrain can provide some indication of the pre-filling topography (Shaw & Owen, 2000).

Bodies of fill material on sloping ground are prone to internal erosion over time as a result of sub-surface groundwater flow, and in this respect the location of pre-existing drainage lines in relation to the made ground is very important. Unless careful drainage measures were constructed prior to placing fill in a natural drainage line, progressive sub-surface erosion can be expected to remove finer material from lower layers of the fill. Later, downward migration of fines into the voids can result both in settlement and more importantly, blockage of the drainage path with consequent problems of high pore pressures developing in the fill. Therefore, deposits of fill on hillslopes require careful investigation of the sub-surface groundwater regime and any associated erosion (see Section 5.9.4).

Construction Techniques

The engineering characteristics and performance of the fill will vary depending on its suitability and the compaction techniques used during emplacement. Historically, although sourcing suitable fill was occasionally difficult, inadequate compaction is the main cause of failures of hillside fill slopes. Most of the older hillslope platforms were formed by end-tipping of material onto the slope so as to progressively build up the desired platform. Basal drainage blankets were not routinely installed. End-tipping tended to result in layering parallel to the slope and, if there are differences in material grading between layers, preferential drainage paths can develop parallel to the slope. The most serious disadvantage of the end-tipping is that the only compaction achieved is by self-weight. Therefore, a prime objective of ground investigations of old fill slopes is to determine the nature of the material and the degree of relative compaction.

Potential Problems

Stability problems result from the combination of uncompacted fill and lack of adequate drainage. Infiltration of surface and groundwater can cause sliding, localised washouts and static liquefaction of the material (Wong *et al.*, 1997). Figure 5.9.1 illustrates possible triggers and contributory factors in fill slope failures. Fill slope failures can result in the sudden development of large volumes of very mobile debris and the consequence of such failures can be disastrous, as occurred at Sau Mau Ping as described below.

5.9.4 Case Studies

Sau Mau Ping Fill Slope Failure in 1976

On 25 August 1976, a fill slope immediately behind Block 9 of the Sau Mau Ping Estate failed and the resulting debris buried the ground floor of the block, killing eighteen people (Figure 5.9.2). After the disaster, a detailed investigation had the following findings:

- The fill, composed of completely decomposed granite, was in an extremely loose state to a depth of at least 2 m below the slope surface, the dry unit weight being in the range of 12.5 to 15.5 kN/m^3 (average 13.5 kN/m^3), corresponding to about 75% of standard compaction.
- The fill was layered parallel to the slope surface, with layers between about 100 and 300 mm thick.
- Beyond the crest of the slope, the dry unit weight of the materials was low but variable to a depth of 7 m, dropping from about 16.5 kN/m^3 to about 12 kN/m^3 (90% to 70% of standard compaction), showing a gradient of densities with depth consistent with the soil having been placed in layers of 1 m to 3 m thick. At greater depths the dry unit weight was about 15 kN/m^3.

These findings indicated that the fill had been end-tipped with no compaction. Such conditions can result in the following:

- The soil strength being very much less than would be obtained with well-compacted fill.
- Rainwater infiltration wetting the soil to an appreciable depth and reducing the strength even further.
- The loose soil structure is vulnerable to collapse during shearing, thereby resulting in significant increase in pore water pressure and reduction of shear strength when failure occurs under a high degree of saturation.

The investigation considered various possible

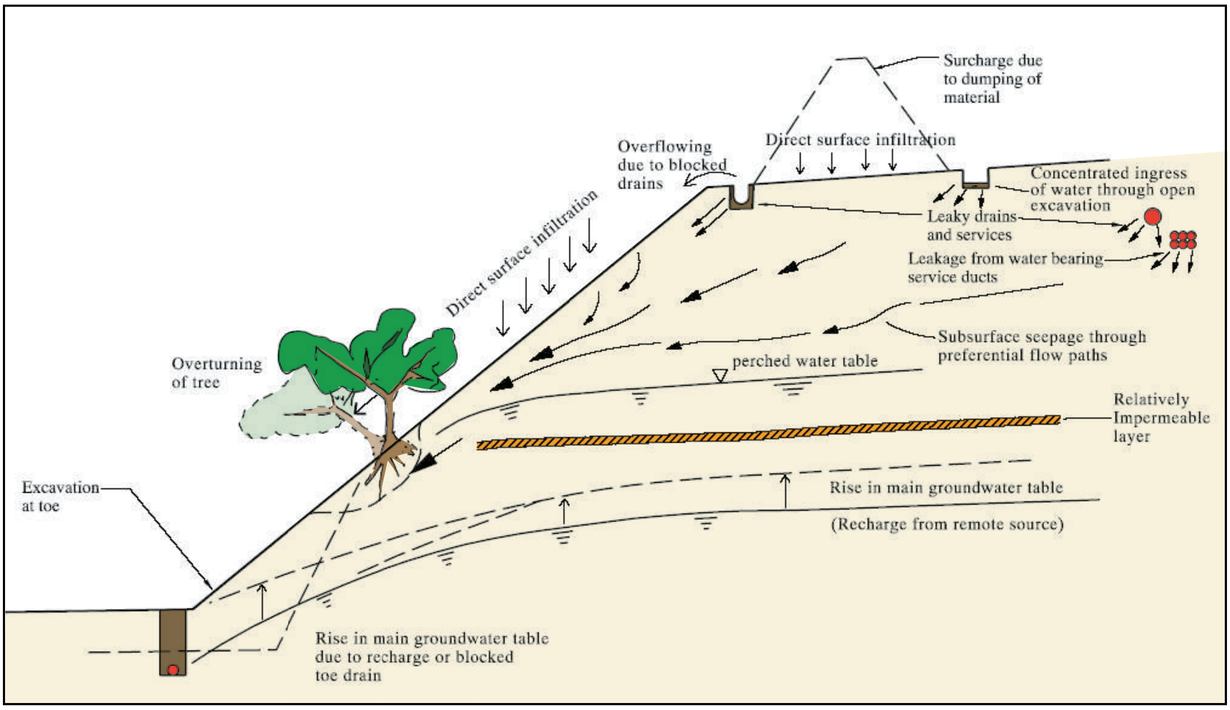


Figure 5.9.1 – Triggers and contributory factors for fill slope failure (Sun, 1998)

sources of water ingress, including direct surface infiltration, rising groundwater from below and infiltration from drainage pipes. The overall conclusion was that direct surface infiltration, possibly aggravated by slight leakage from surface drainage, was the prime cause of the high degree of saturation of the fill. Full details of the investigation are given in B&P (1976), Hong Kong Government (1977) and Knill *et al.* (1999), the latter being a reprint of the original 1977 report.

Loss of Grout Associated with Voids in a Fill Slope Below Tai Po Road

The fill slope located below Tai Po Road was constructed some time between 1949 and 1963. The slope was upgraded in the early 1980s, including re-compacting the top 3 m of the slope and reducing the gradient of the slope by constructing a retaining wall at the toe (Figure 5.9.3). In 2004 upgrading works were carried out under the Landslip Preventive Measures (LPM) Programme. These works included

Figure 5.9.2 – View of the fill slope failure at Sau Mau Ping Estate in 1976

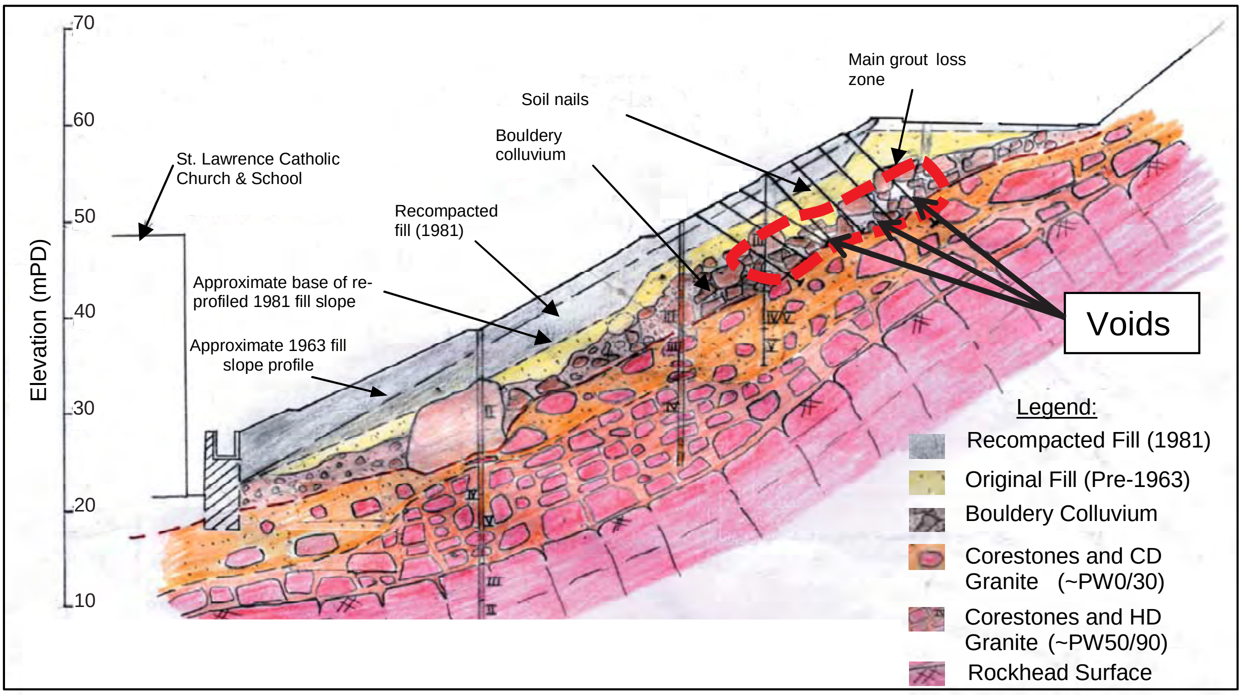


Figure 5.9.3 – Cross-section through a fill slope below Tai Po Road showing location of soil nail grout loss zone

installation of soil nails into granite saprolite below the fill slope.

During installation of soil nails (10 m to 13 m in length) in the upper level of the slope extensive loss of grout was experienced within a cluster of several nails at approximately 10 m depth, where *in situ* material was expected. A CCTV survey revealed several large voids of up to 1 m across.

Following the grout loss a detailed API was carried out. The 1949 photograph (Figure 5.9.4), taken prior to construction of the original fill slope, indicates that the site of the slope is the lower portion of a drainage line which extends up the hillside above Tai Po Road. The hills surrounding the site are comprised of medium-grained granite saprolite with significant numbers of large corestones (>5 m in size). Below the road the drainage line broadens out into a deposition zone with accumulations of bouldery colluvium. Because of the fluvial environment, most of the fines had been removed.

The original fill slope was constructed for the widening of Tai Po Road and the contemporary practice was to end tip locally derived saprolite soil (probably including boulders and cobbles) to form the slope. Thus the fill material forming the slope was probably placed directly on top of the bouldery colluvium. Subsequent upgrading works in the 1980s only affected the top 3 m of fill and thus the underlying original fill and bouldery colluvium probably contained a high percentage of voids (Figure 5.9.3).

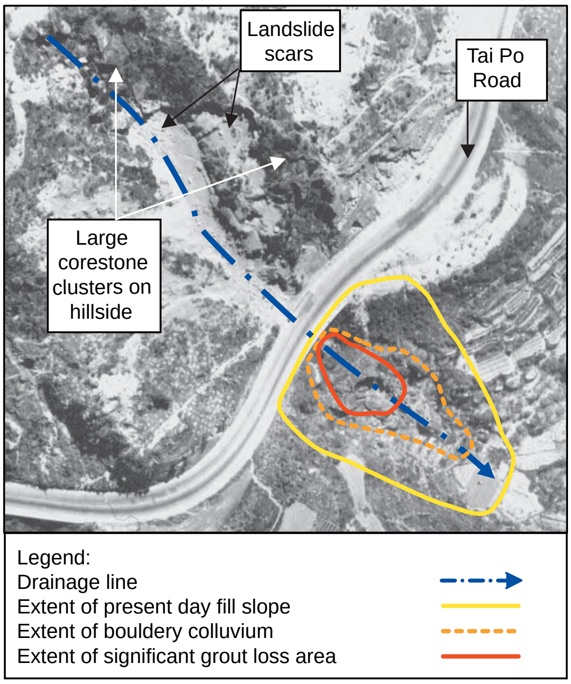


Figure 5.9.4 – 1949 aerial photograph showing the location of the fill slope below Tai Po Road prior to construction

Many old fill slopes were constructed across drainage lines to form roads or building platforms, and ground conditions similar to that described above probably occur elsewhere in Hong Kong. A geological model which incorporates a detailed API, rather than just a simple account of the site history, can assist in the identification of sites that may have similar potential problems. Consideration of the geomorphology of the area, including the catchment above the site and the groundwater system prior to deposition of fill, may assist in assessing how the groundwater system might have changed after the infilling took place, including any development that might have modified the surface water drainage regime. If a potential problem is identified, investigation methods to confirm the presence of voids may include resistivity surveys of the slope to locate potential areas of voids (see Section 6.4.7) and confirmatory drillholes through the slope where voids are suspected.

Where such ground conditions are not foreseen, they can have significant implications on cost and programme. This highlights the need for engineering geological input at an early stage. Consideration should also be given to the hydrogeology of the site when assessing the most appropriate remedial measures. Relatively loose soil material with significant boulder content is susceptible to internal erosion via groundwater seepage and flow. Excessive erosion could result in subsidence (see Section 6.4.7). On the other hand, any measures that block or hinder groundwater flow could also potentially have adverse effects on the stability of the slope.

6. ENGINEERING APPLICATIONS

6.1 INTRODUCTION

This chapter highlights the application of engineering geological skills, knowledge and practice to facilitate the resolution of key geotechnical issues relevant to different engineering applications in Hong Kong.

Applications which require engineering geological input include:
- natural terrain hazard assessment and mitigation works,
- site formation,
- slope engineering,
- foundations,
- deep excavations,
- tunnels and caverns,
- marine works and reclamation,
- landfills and contaminated land, and
- natural resource assessment.

This chapter focuses on the need to produce realistic geological and ground models to identify and address the key geotechnical issues which are most relevant to each engineering application.

Essential elements in the development of geological and ground models include:
- early definition of the likely geological and geotechnical complexity,
- identification of key areas of geotechnical uncertainty for further investigation,
- characterisation of the ground in terms that are relevant to the engineering application,
- assessment of relevant external factors which may affect or be affected by the project, and
- review of the geological and ground models during construction.

Local and international experience indicates the need for development of the models as more information is obtained throughout the various stages of a project. Many of the references and examples in this chapter indicate that engineering geological input is most effective when applied at the initial assessment, site investigation and interpretation stages of a project. Many of the examples also indicate that engineering geological input during construction is important for verifying as well as updating the geological and ground models, particularly with regard to the identification of any naturally occurring safety-critical or performance-critical features.

6.2 NATURAL TERRAIN HAZARD STUDIES

6.2.1 Introduction

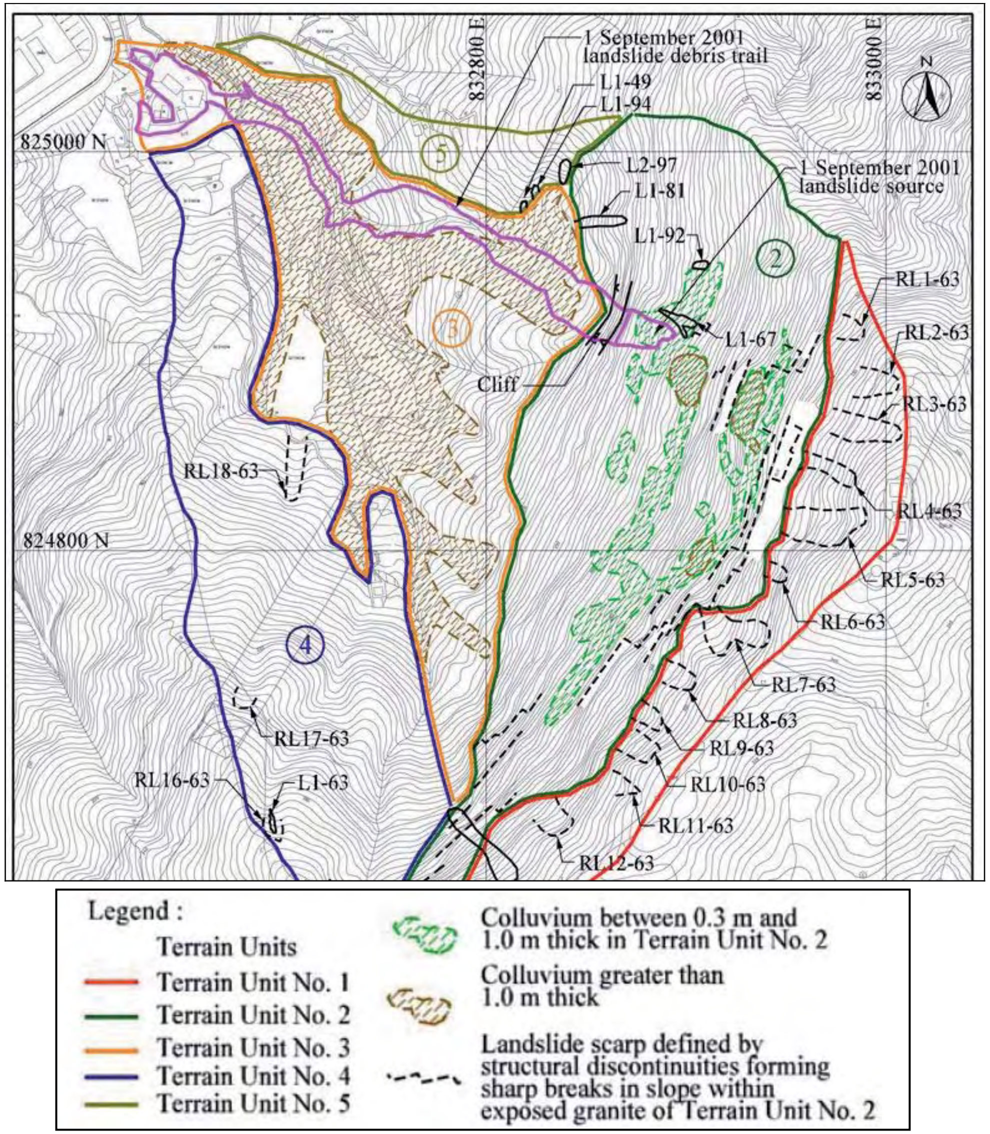
The main natural terrain hazards in Hong Kong comprise:
- landslides:
  - open hillslope landslides
  - channelised debris flows
  - deep-seated landslides
- rock falls
- boulder falls

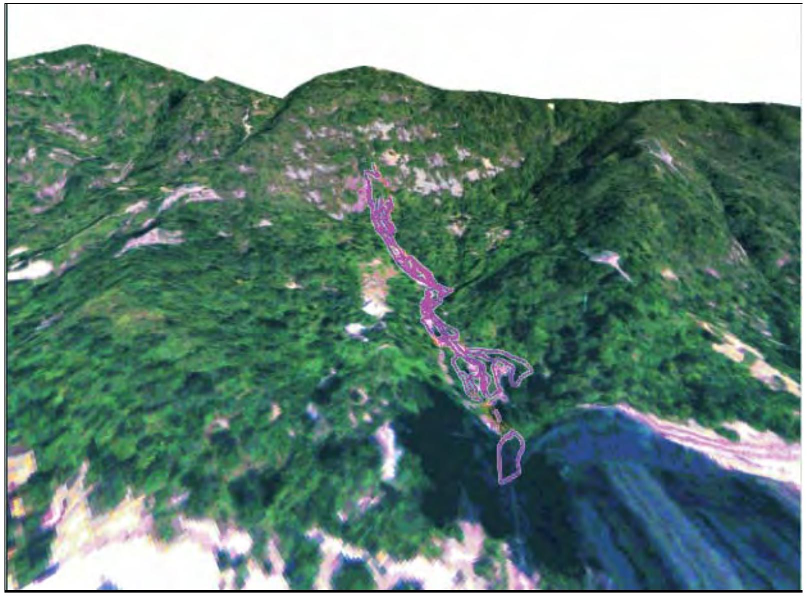
These hazards have caused fatalities, injuries and economic losses in Hong Kong (Wong *et al.*, 2004), primarily because of the close proximity of dense urban development and steep hillslopes. Natural Terrain Hazard Studies (NTHS) are carried out to:
- assess the possible hazards,
- quantify the identified hazards and analyse their risk, and
- devise mitigation strategies.

These studies are an increasing component of engineering practice in Hong Kong as new development extends into steeper terrain and existing development is assessed for potential risk. While natural terrain refers to hillsides that have not been substantially modified by human activity, hazard studies may need to consider anthropogenic features such as unregistered fill bodies and abandoned agricultural or squatter terraces.

In terms of assessing, quantifying and mitigating against natural terrain hazards, the key engineering geological activities are:
- An evaluation of the landscape evolution in terms of geological and geomorphological processes.
- Assessment of these processes in terms of their potential to generate natural terrain hazards.
- For each type of hazard identified, evaluate the likely magnitude and frequency and the potential for entrainment of regolith materials.
- For each type of hazard identified, assess factors that affect debris mobility.

The following sections outline the engineering geological input required to support natural terrain hazard studies within the framework of a 'model' approach. This is consistent with the approach to NTHS outlined in Ng *et al.* (2003a).





Lei Pui Street Terrain Units

Five distinct terrain units were identified with significant geomorphological characteristics, most importantly:

- the landslide occurred and gained momentum in Terrain Unit No. 2 (steep granite outcrop defined by sheeting joints);
- the failure entrained significant volumes of debris in Terrain Unit No. 3 (low inclination mid-slope area below granite slab outcrop with extensive and thick colluvial (talus) lobes); and
- there is evidence of hazard in Terrain Unit No. 1 (steep upper hillside and ridge above granite outcrop with deep weathered profile and numerous concave relict and recent landslide scars).

Figure 6.2.1 – Terrain units at Lei Pui Street (MGSL, 2004)

6.2.2 Geological Models

General

A fundamental aspect in assessing natural terrain hazards is understanding the geological and geomorphological processes that currently operate, as well as those that have operated in the past. Although these processes are complex, they can be interpreted using engineering geological principles to formulate a geological model such that potential natural terrain hazards arising from them can be identified and assessed. The formation of a geological model provides the basis for identifying the likely geomorphological and geological controls on the location, type, magnitude, frequency and runout characteristics of potential hazards, and generally comprises two distinct components:

- mapping and assessment of terrain characteristics and interpreting how the landscape at a site evolved, and
- evaluation of the natural terrain instabilities in the area to develop an inventory of potential hazards for the site.

Terrain Characteristics and Landscape Evolution

Terrain characteristics and landscape evolution form the basis of the geological model for natural terrain assessments and include the assessment of the spatial variations and relationships in morphology, materials, processes and age (GEO, 2004g). In order to rationalise the assessment in terms of hazard identification, terrain with similar characteristics can be grouped together to form 'terrain units'. Geomorphological processes and the derivation of terrain units are discussed in Section 4.5.

Considering the locations of hazards in relation to terrain units with similar engineering geological characteristics allows the evaluation of hazards in terms of their geomorphological setting and facilitates the prediction of potential future hazards.

Examples of terrain unit approaches include:

- The Lei Pui Street channelised debris flow study (MGSL, 2004) where five terrain units were identified (Figure 6.2.1) that related morphology, aspect, material properties (e.g. depth to rockhead, type and depth of colluvium), structural domains and process activity (e.g. landslide occurrence, weathering).
- Process-based terrain unit model (Figure 6.2.2) for the NTHS at North Lantau Expressway (OAP, 2004b).
- Terrain units identified for the Cloudy Hill NTHS (HCL, 2003b) that incorporated landslide hazard models into broader landform assemblages (Figure 6.2.3).

Hazard Inventory

The second major component of a geological model for a NTHS is the development of a site-specific inventory of instabilities and potential hazards. An inventory of recent and relict instabilities can be produced initially from existing data (Ng *et al.*, 2003a). However, this needs to be verified and complemented with a comprehensive API. Engineering geological attributes (e.g. geology, geomorphological setting, debris trail characteristics, possible initiation mechanisms, etc.) can be incorporated into the inventory. This allows examination of the influence of engineering geological characteristics on the generation of natural hazards and aids selection of field inspection and ground investigation locations.

Care should be exercised when determining the length of debris trails from API, especially channelised debris flows, as post-landslide fluvial processes may carry landslide debris further down the drainage line. This outwash can be difficult to differentiate from landslide debris by API. In addition, the appearance of the trail can change rapidly through vegetation re-growth or erosion.

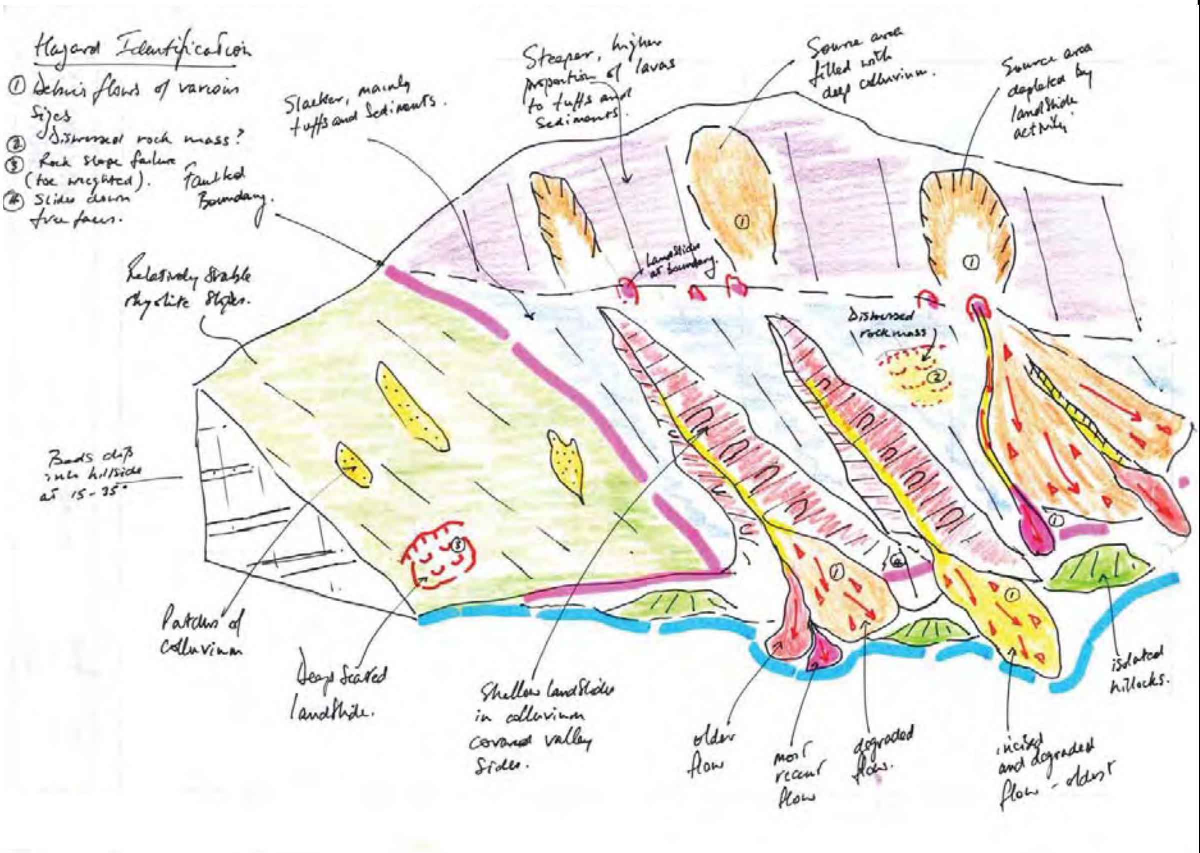
Whilst recent landslides are relatively easy to identify from API, the identification of relict landslides requires considerable interpretation and expertise as landslide scars progressively lose their morphological definition at rates depending on factors such as vegetation, landslide size, material and location. Varying levels of certainty with respect to the interpretation of individual features can be assigned to reflect this (Moore *et al.*, 2001; Parry *et al.*, 2006).

Not all landslides can be detected from API, particularly where complete detachment does not occur. Any additional landslides noted during field mapping should be added to the inventory and, if necessary, the geological and hazard models revised.

In summary, a geological model for the assessment of natural terrain hazards should combine all relevant information on terrain characteristics together with site-specific information from the inventory of

The North Lantau Expressway Natural Terrain Hazard Study (OAP, 2004b) used API to identify a number of relict and recent landslides within the study area.

Field mapping and ground investigation provided additional information on terrain characteristics. These data were assimilated into the process-based hazard models outlined in the following figure, prepared by Baynes Geologic for OAP (2004b) during a site reconnaissance:



Hazard models for this site include:

- Deep-seated rock-slope failure in rhyolite
- Possible failure of distressed volcanic rock mass
- Debris flows of various sizes and ages derived from:
  - o shallow landslides in colluvium and in lower-slope valley sides of tuff and sedimentary lithology
  - o evidence of large relict landslides in steeper slopes

The landslide inventory formed the basis of a magnitude-frequency analysis of natural terrain hazards. This analysis was a fundamental component of a Quantitative Risk Analysis for the North Lantau Expressway at the toe of the hillslopes.

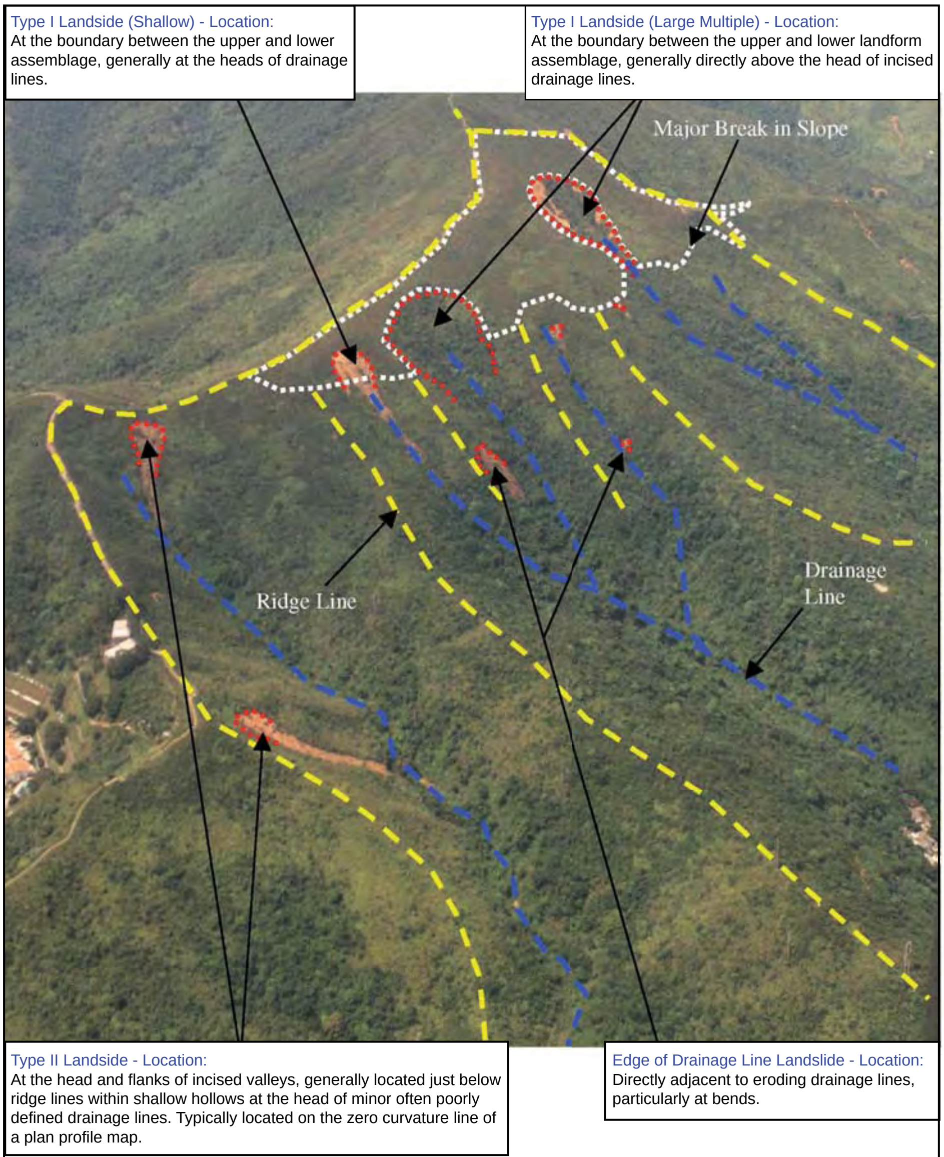
Figure 6.2.2 – Process-based terrain unit model for the North Lantau Expressway NTHS (OAP, 2004b)

instabilities and potential hazards. This provides a framework to locate site-specific hazards within their geological and geomorphological setting (e.g. Parry & Ruse, 2002; Figure 4.5.2). The model can be presented on 2-D maps (e.g. Hearn, 2002) or 3-D block diagrams (see Figure 6.2.4).

6.2.3 Hazard Models

General

Detailed information is required for each potential natural terrain hazard identified in the geological model. This provides site-specific engineering geological information which links the geological model to the hazard assessment (see Section 6.2.4).



The **Cloudy Hill Natural Terrain Hazard Study** (HCL, 2003b) described a geological model of two distinct landform assemblages separated by a major break in slope gradient:

- The **upper landform assemblage** is an area of deep weathering, often covered with a mantle of weathered colluvium and local areas of exposed rock and boulder fields. It is characterized by slightly convex, almost planar, generally smooth surfaces with few drainage lines.
- The **lower landform assemblage** is an oversteepened area formed by the processes of hillslope retreat, complex series of 'erosion fronts' of relict landslides and incised drainage lines.
- A **distinct convex break in slope** marks the boundary between the two landform assemblages, characterized by a zone of instability, which has progressively consumed the upper assemblage as it has propagated upslope. Knife-edge ridges are found where hillside retreat has reached the ridgeline.

The Cloudy Hill NTHS classification of recent landslides is summarized on the photograph above. These hazard models integrate results of landslide mapping, regolith mapping and the morphological setting characterized by the landform assemblages.

Figure 6.2.3 – Terrain units and hazard models at Cloudy Hill (HCL, 2003b)

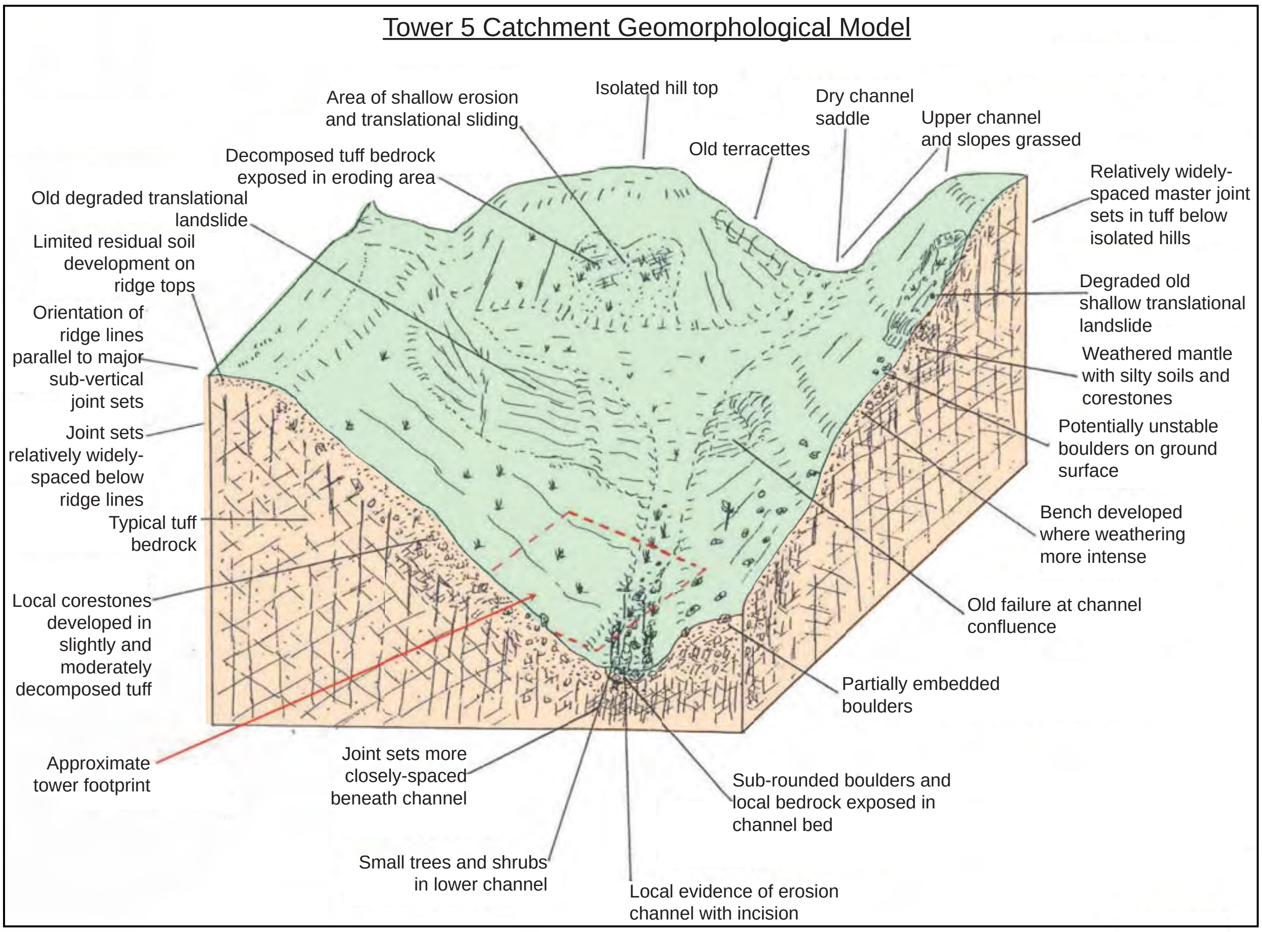


Figure 6.2.4 – Geomorphological model for Tung Chung cable car NTHS (Mott Connell, 2003)

Key factors which need to be incorporated in hazard models are summarised below for each type of hazard.

Landslide Initiation

Key initiation factors include:

- Adverse hydrogeological settings:
  - Contrasts in permeability promoting elevation of pore water pressures, e.g. colluvium-saprolite interface.
  - Preferential groundwater flow into the source area via soil pipes or open joints or fault zones.
- Geological influences:
  - Adverse geological structure – discontinuities (e.g. stress-relief joints) and fabric (e.g. foliation) which may provide potential failure planes or release surfaces (see Section 6.4.4).
  - Geological boundaries – these may influence several aspects, e.g. differential weathering of materials forming oversteep terrain and faults forming aquitards or preferential pathways for groundwater flow.
- Topographical factors - concentration of water at hillslope concavities (HCL, 2003b), heads of drainage lines, immediately below rock outcrops, and at, or close to, concave breaks in slope (MFJV, 2003a).
- Slope deterioration – diagnostic features include tension cracks, dilated and infilled joints, and slickensides.
- Human influences – water discharge from developments and leakage of water-carrying services.

Open Hillslope Landslides

Open hillslope landslide debris runout broadly increases with source volume (Corominas, 1996; Wong *et al.*, 1996). Therefore, the key engineering geological input is determination of the likely width and volume of a potential source, as this factor essentially controls the travel distance and thus hazard and risk.

Channelised Debris Flows

Debris flows develop when the landslide debris is mixed with sufficient water for slurry flow to occur. Channelised debris flows are landslides where stream water is mixed with debris and where the debris follows an established stream bed (Ng *et al.*, 2003a). Channelised debris flows typically have high mobility, i.e. they are rapid and can have a long travel distance, with the debris possibly reaching several hundred metres or more from the source area. Thus, of all the landslide types, channelised debris flows are generally the most hazardous.

Unlike other landslides, channelised debris flows have the potential to entrain large amounts of material within drainage lines, potentially increasing their volume considerably. Large channelised debris flows have occurred in Hong Kong (Figure 4.5.3) where entrainment of significant amounts of additional material along the debris trail has taken place (King & Williamson, 2002; MGSL, 2004). Entrainment may be a critical issue in the assessment of hazard where:

- the source volume is large,
- the topography is steep enough to increase debris velocity and hence erosive power, and
- readily entrainable material is present within the travel path.

Consequently a key aspect is the mapping of drainage line morphology and associated superficial deposits in order to evaluate mobility and to identify potential areas of entrainment. Superficial deposits can be entrained in large volumes if a channelised debris flow impacts the material with sufficient energy (King, 2001a,b,c; MGSL, 2004). Where the energy is less, superficial deposits may still be entrained although commonly in more restricted amounts (MFJV, 2003b). Other possible entrainment processes include liquefaction due to severe impacts of falling debris (MGSL, 2004), and undercutting of channel banks (Franks, 1998).

The mobility of channelised debris flows is also influenced by the degree of channelisation along the potential travel path. It is therefore important to evaluate the channelisation ratio (CR - width to depth ratio of the effective cross-sectional area occupied by a potential landslide) at key locations along the potential travel path. CR is dependent upon both the source volume and the topography, and is not necessarily related to the actual channel geometry, which can be relatively small in scale.

Deep-seated Landslides

Deep-seated landslides tend to be slow moving, often with small travel distance (in the order of metres to tens of metres) but involve large volumes of detached or deformed material. This type of hazard generally poses a risk to property or infrastructure rather than to life, especially where the overall slope is gentle or where the slope setting is favourable to regaining equilibrium after some slope movement (Wong & Ho, 2001). Engineering geological input required for this type of hazard includes mapping of deformation (Parry & Campbell, 2003). Ground investigation may also be required to investigate the nature of the failure surface, as well as long term monitoring to establish the conditions which control movement.

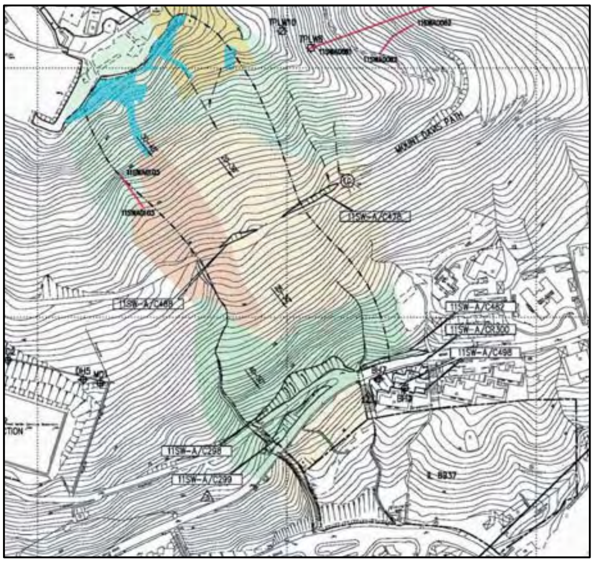
In the case of progressive movements where the setting is not favourable to regaining equilibrium after some slope movement (e.g. relatively steep slope profile below the toe, locally steep slopes directly above the toe, or where deep slope movements progressively develop upslope of a recent landslide scar), the possibility of a large and mobile detachment may need to be considered (Wong & Ho, 2001). In such cases, intensive engineering geological mapping may be necessary to assist in the assessment of potential mobility and risk.

Rock Fall

Engineering geological input for potential rock fall sites will consist mainly of mapping previous rock falls, discontinuity mapping and kinematic analysis (GCO, 1984, 1988a).

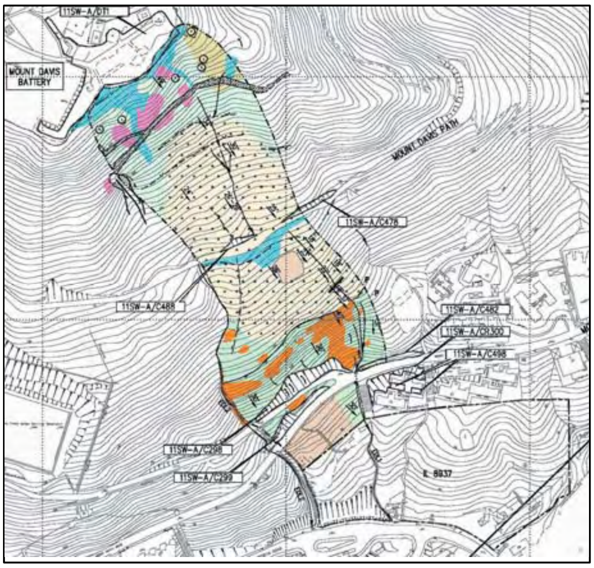
Boulder Fall

Erosion of the surrounding soil and tree root wedging are key initiation factors for boulder falls. Detailed field inspections of boulders are required to assess the degree of embedment, potential for instability and runout path. Ng *et al.* (2003b) and Shi *et al.* (2004) discuss the potential of using image processing techniques to map boulders on natural hillsides. Classifying boulders in terms of their origin is important, i.e. colluvial boulders or exhumed corestones *in situ*, which may have a bearing on the potential stability. Engineering geological characterisation of the terrain below a potential boulder fall area and shapes of the potential boulders can help in the subsequent runout assessment.



Preliminary Geological Model

- Desk study information, particularly a detailed API, permits a preliminary classification of the site in terms of morphology, material (lithology and regolith), human activity, previous evidence of hazard, etc.
- This includes looking beyond the site boundary to ensure the setting of the site has been adequately considered.
- The preliminary geological model is used to direct the field work and optimise the site investigation.



Detailed Engineering Geological Model

- An amended and more detailed model of materials, morphology, history and process is generated after fieldwork and ground investigation.
- In the case of this site, more detailed information was obtained on the distribution and classification of regolith and lithology, slope gradients, seepage zones and human activity.
- The information in the detailed engineering geological model permits identification of potential hazard.

Potential Natural Terrain Hazards

- Areas susceptible to potential hazard are identified. The areas and potential hazards can be rationally evaluated using desk study, field and ground investigation information.
- The detailed nature of each hazard becomes the input information for any subsequent hazard assessment if a risk has been identified.
- In this example, the upper slope offered little risk given its distance from the facility. Detailed analysis of the lower slope also indicated little risk to the facility.

Figure 6.2.5 – Progressive development of geological models for NTHS

Summary

Detailed field observations allow the initial geological model to be tested and built upon. Previous failures provide evidence of initiation factors and debris runout behaviour. These can be used to establish site-specific hazard models for the location, size, type and mobility mechanisms of potential failures, preferably based on terrain units determined in the geological model. The hazard models should allow realistic and focused assessments of hazard, risk and potential mitigation options. Where failures are not present, it might be possible to apply information from adjacent sites or generalised parameters from further afield, but this requires careful judgement.

6.2.4 Hazard Assessment

General

The hazard assessment builds on the geological model and hazard models in order to assess the potential risk to facilities in question, particularly by the quantification of hazard location, magnitude, frequency and mobility.

The main aims of the hazard assessment for a given facility are to establish:

- reliable design events for each hazard model identified,
- the potential frequency of such events,
- the likelihood of such events affecting the facilities in question,
- mitigation options including, where appropriate, cost/benefit analysis, and
- inputs needed to construction.

The identification of a suitable design event requires careful engineering geological judgement. The results of hazard assessments can be very sensitive to changes in the selected variables (e.g. potential volume, distance between source and facility, rates of deposition and entrainment, etc.). The key value of utilising an engineering geological approach at this stage is to ensure the range of design events are consistent with the information derived from the geological and hazard models. Allowing ongoing feedback between the gathering and analysis of geological data promotes rational hazard analysis, possibly allowing a reduction in the scale of design events with associated savings in cost.

An example of the development of a geological model is given in Figure 6.2.5, where preliminary terrain characteristics were evaluated and refined by API and field mapping, which then formed the basis for development of a hazard model.

Landslide Mapping

Where landslides have occurred previously, particularly where these are recent, engineering geological mapping can provide details of landslide initiation factors (Ruse *et al.*, 2002) and mobility characteristics for use in subsequent hazard assessment. Such details include depths and volumes of source areas and any areas of entrainment, channel morphology, geology, areas of transportation and deposition and description of materials. Ideally these should be recorded by chainage sections having similar characteristics (Figure 6.2.6). The volume balance of erosion and deposition along the trail permits estimation of an active volume at any trail section. Evidence of any debris super-elevation should be recorded to permit estimation of travel velocity (King, 2001a,b,c). Recording such data systematically is important for consistency of results and later analysis. Use of a standard pro-forma, topographical plans and ortho-rectified photographs helps the systematic recording of field mapping of landslide sources and debris trails (MFJV, 2002a).

When mapping debris trails, care is required to differentiate post-landslide alluvial erosion and outwash from the actual mobile debris of the landslide event (King & Williamson, 2002). This is because the hazard and risk presented by a mobile debris front are significantly greater than the hazard and risk presented by outwash of fine material following the event (King, 2001a,b,c).

Careful mapping of landslide debris can identify different phases of landsliding, which may be important for subsequent hazard assessment in respect of the derivation of a design volume for mitigation works. Debris flows commonly exhibit a debris front of coarser material (e.g. boulders) followed by a more mobile body of finer debris. However, subsequent retrogression at the landslide scarp or undercutting in the trail may cause secondary pulses of debris (Franks, 1998). Total debris volume can also be increased by the coalescence of near-concurrent landslides in a drainage line, e.g. MFJV (2003b). Subsequent water flow into a fresh landslide scar may promote hyperconcentrated flows, or debris floods, which exhibit sorting due to high debris fluidity (King & Williamson, 2002).

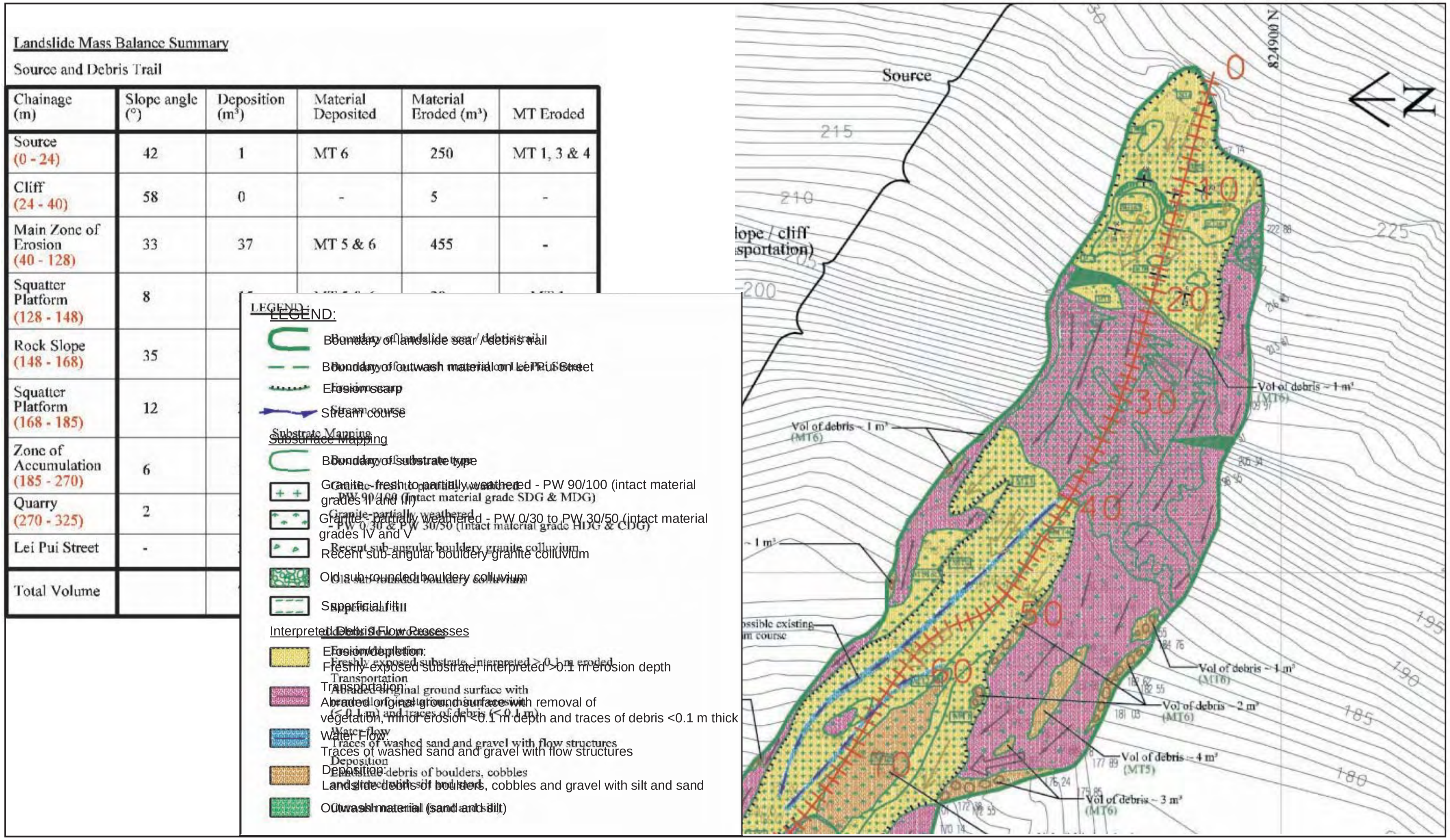


Figure 6.2.6 – Detailed mapping of the landslide scar at Lei Pui Street (MGSL, 2004)

The temporary formation and subsequent break of a dam of landslide debris within a stream course can increase debris mobility and hence runout distance, e.g. HCL (2001). In cases where subsequent alluvial erosion has occured, the apparent trail length may be exaggerated if post-failure outwash debris is not correctly identified by field mapping. Similarly, the amount of entrained material may be overestimated if post-landslide alluvial erosion is not identified.

Susceptibility Analysis

Susceptibility is the propensity of a site to produce hazards at locations of interest. Some analyses evaluate susceptibility by using a statistical approach incorporating regional landslide information with generic spatial data such as slope gradient and rock type, e.g. Evans & King (1998); Dai & Lee, (2001). Whilst such analyses can be useful indicators of potential hazard for planning purposes at a regional scale, they may be of insufficient resolution for application to site-specific risk management (Wong, 2003). Significant differences between field measured slope gradients and those generated using GIS have also been noted (e.g. Parry & Wong, 2002).

Prediction of hazard behaviour can be significantly improved at area-study scale (ca. 1:5,000) or greater, when susceptibility factors are chosen based on their causative relationship with geological and ground models. For example, the 1:2,000-scale Tsing Shan Foothills susceptibility analysis drew from five separate types of geological mapping, which provided information on landslide initiation factors, regolith and rock outcrop distribution, locations of heads of drainage lines, and lithological boundaries influencing susceptibility (Parry *et al.*, 2002; MFJV, 2003c). Susceptibility analysis in the Cloudy Hill NTHS also predominantly used topographical/ morphological factors, including slope gradient, regolith type and plan curvature (HCL, 2003b).

The hydrogeological regime is a key influence on landslide susceptibility. While it is generally not practicable to 'map' groundwater levels directly during natural terrain studies, other attributes can be mapped that may indirectly reflect areas of relatively high groundwater level. For example, of 47 possible contributory factors identified by mapping landslide scars in the Tsing Shan Foothills, only three (i.e. lithological boundaries, regolith downslope of rock outcrop and regolith at the head of a drainage line) were used in the susceptibility analysis (Figure 6.2.7), in addition to the two basic factors of regolith type and slope gradient (Parry *et al.*, 2002). With the exception of slope gradient, all the attributes used in the susceptibility analysis can be strong indicators of potentially adverse hydrogeological conditions as summarised below:

- Lithological boundaries can give rise to spring lines or areas of relatively high groundwater where contrasts in the hydrogeological characteristics of the rock masses or soils are sufficiently strong.
- Regolith immediately downslope of rock outcrop is subject to increased saturation by infiltration of the additional run-off from the rock outcrop. In addition, the regolith is likely to be relatively thin which can promote the rapid build-up of a high perched groundwater table.
- The head of a drainage line is a relatively 'active' area in geomorphological terms which may contain ephemeral springs and relatively shallow regolith which is more susceptible to landsliding due to perched groundwater build-up.
- Certain types of regolith, e.g. talus and loose depression colluvium are normally more permeable and relatively weaker than the underlying materials. In areas where these deposits are relatively thin, they may be very susceptible to landsliding due to the build-up of high perched groundwater levels.

As there may be some spatial variability in the degree to which some attributes influence landslide susceptibility, the susceptibility analysis should be based on, and subsequently checked against, the site-specific geological and ground models to ensure that the analysis is representative.

Magnitude-Frequency Analysis

A hazard assessment needs to determine the size of the potential hazard sources and how frequently hazardous events are likely to occur. To estimate frequency, a simple 'maximum age' relationship is commonly assumed to represent the hazard database, such as the 100-year period used by Evans & King (1998). Pinches *et al.* (2002) assigned suggestive ages to different types of landslides at Yam O. However, given the difficulties of determining landslide dates and the great range of landslide ages (Sewell & Campbell, 2005), this process requires considerable care.

The difficulty of assigning appropriate ages to landslides was illustrated by OAP (2004a), where a soil horizon dated to approximately 675 years old was

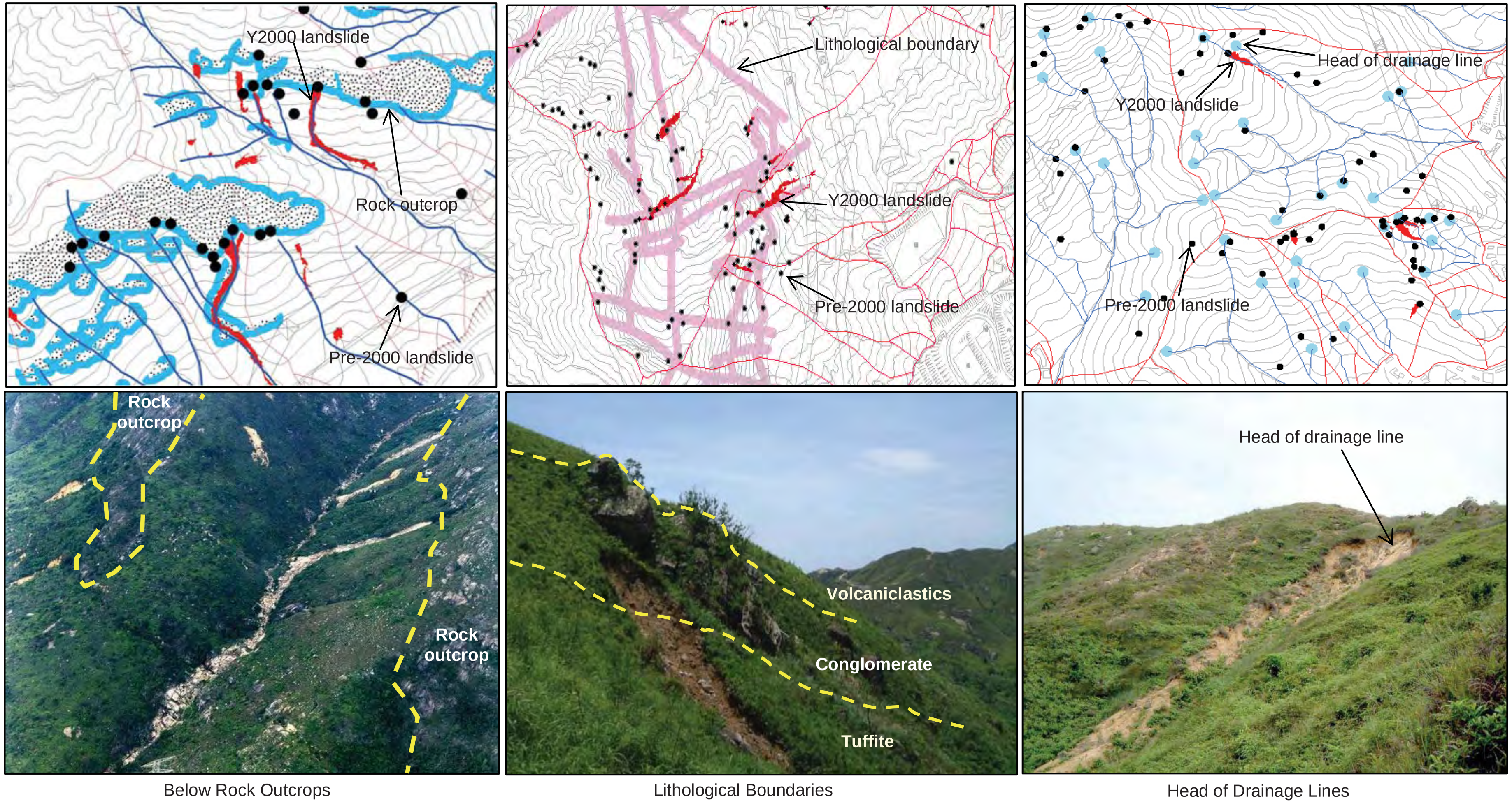


Figure 6.2.7 – Examples of landslide initiation factors (MFJV, 2004a)

buried by debris from a relict scar initially assumed to be less than 100 years old.

Engineering geological input is needed in the assessment of magnitude-cumulative frequency (MCF) relationships to ensure that the relationships reflect the findings of the ground model. The approach can provide a false sense of precision if the period selected to represent the hazard database is inappropriate, e.g. if the database is too small or is taken from a significantly different location (Wong *et al.*, 1998; MFJV, 2003c; OAP, 2005). Furthermore, the hazard inventory can have signi ficant problems of over-representing younger and lar ger landslides compared with smaller and older features that are less readily observed (Hungr, 2002).

Given the uncertainty about the ages of relict landslides visible in the landscape, a range of ages can be applied to the generated MCF graphs by assuming differing ages for dif fering volumes of landslide scars (MFJV, 2003c). An example is shown in Figure 6.2.8.

Mobility Assessment

Having determined the location of various sources of design events within the landscape, mobility analysis aims to derive estimations of the type, volume and velocity of the potential generated debris at the facility in question. The analyis can be either empirical or analytical. The main contributions of engineering geology to both approaches are:

- providing raw data about landslide debris runout and drainage line characteristics, and
- ensuring appropriate application of such data.

Input information for analytical models of channelised debris flows includes entrainability, size of catchment/ stream course, channelisation ratio, debris volume, debris path longitudinal pro file, debris height and super-elevation. The failure to distinguish between runout distances of remoulded debris and outwash could result in significant errors in the back analysis of the landslide event.

Figure 6.2.9 illustrates the importance offield data on debris distribution and drainage line characteristics for use in an analytical model. The analysis was used to gain insight into the likely sequence of events and mobility of the debrisflow for the design of mitigation measures.

Factors controlling rockfall and boulder fall mobility include block size and shape, slope gradient and surface characteristics such as vegetation cover and hardness (e.g. Chau *et al.*, 2001). Such factors have spatial distributions that can be addressed through the use of appropriate terrain units.

Example of Mobility Data from Landslide Mapping (MFJV, 2003b)

The Tsing Shan Foothills NTHS analysed 121 landslides that occurred in a single rainstorm. Data from 59 open hillslope landslides indicated that debris runout on hillslopes is inhibited by local variations of slope morphology and expulsion of water from the mobile material. No signi ficant entrainment was noted, the maximum travel distance was 90 m and the minimum angle of reach was 24º. Source volumes typically had direct relationships with travel distance and trail width, and an inverse relationship with angle of reach.

The average deposition rate per unit area was 0.27m^3 per m^2. This did not vary signi ficantly except with major local variations in slope morphology . Thus, trail width was a crucial control of runout for open hillslope landslides. The data allowed estimates of average deposition rates, maximum travel distances and maximum active volumes.

Data mapped from 24 channelised debris flows showed the importance of channelisation ratio (CR) as a key factor affecting mobility. A CR of less than 10 for at least 30% of the debris trial length was a key condition to maintain mobility in channelised landslides. Conversely, apart from locally steep slope angles, no significant entrainment occurred where the CR exceeded 10.

Hazard Mitigation

In summary, sufficient engineering geological input into a NTHS at all stages facilitates consideration of appropriate hazard mitigation strategies in terms of performance, safety, and cost-effectiveness.

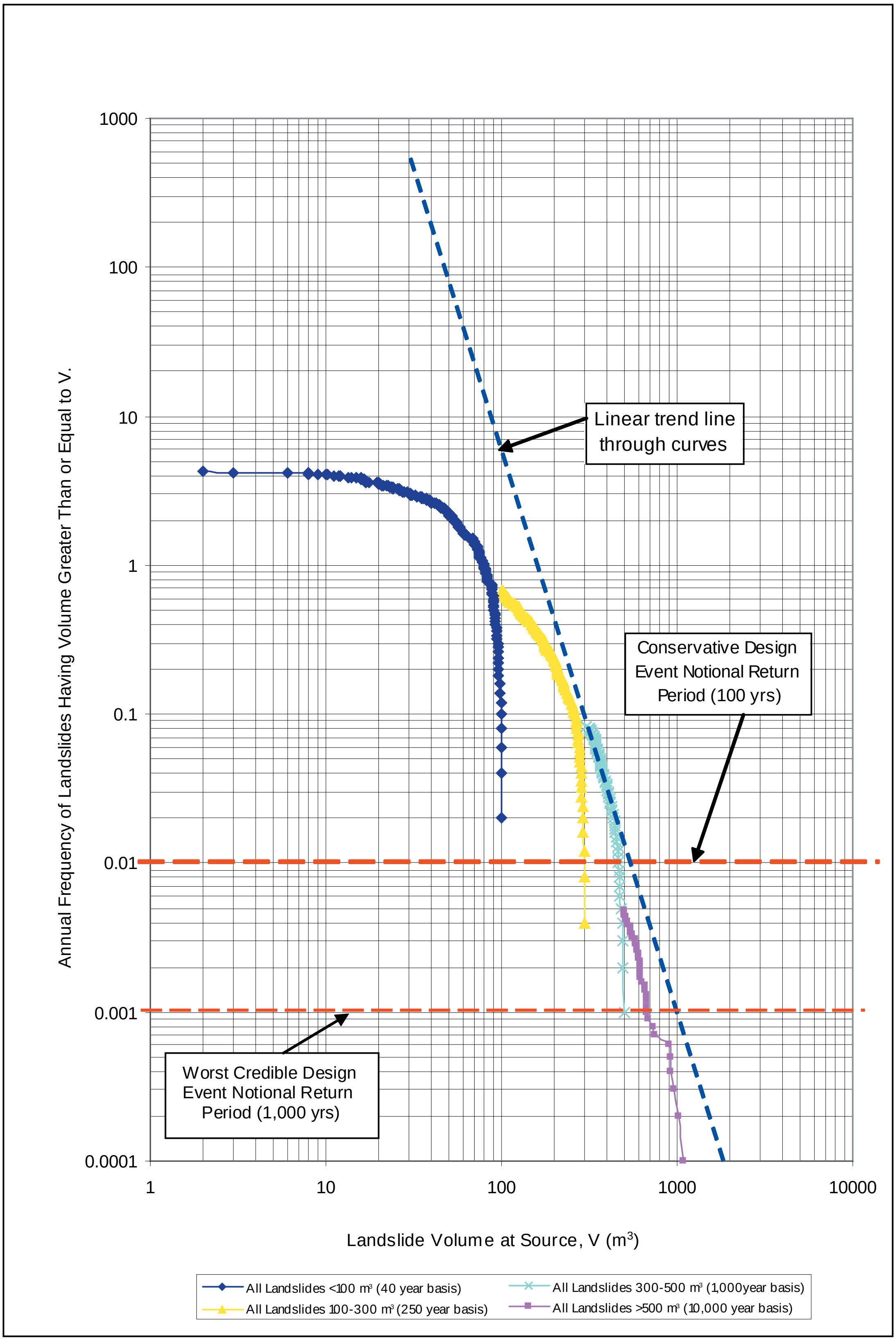
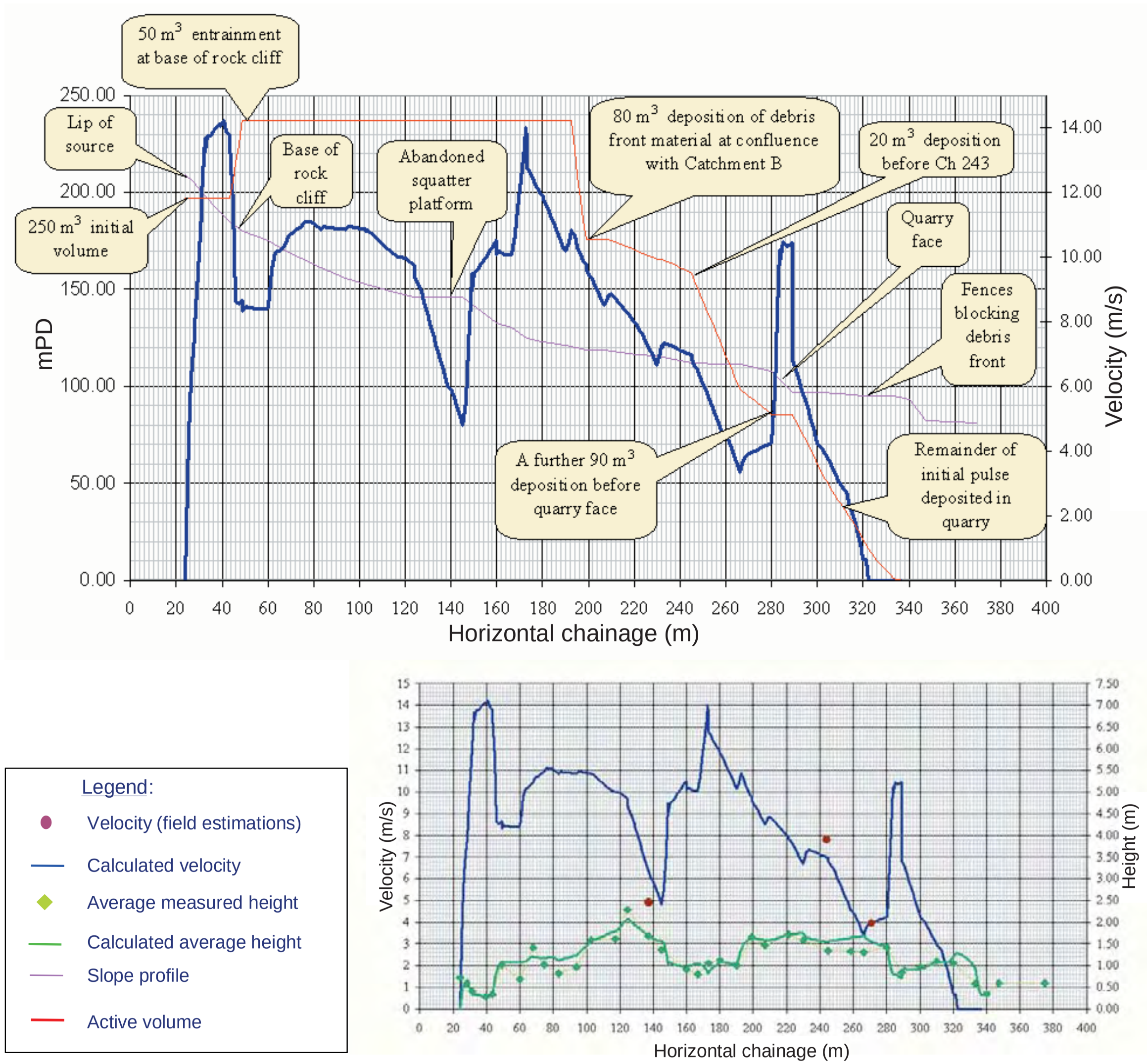


Figure 6.2.8 – Tsing Shan Foothills landslide frequency/magnitude for relict, recent and Year-2000 landslides on 100 year to 10,000 year time interval basis (MFJV, 2003c)



The Lei Pui Street debris flow mobility was back-analysed using a numerical model (MGS, 2004):

This case illustrates the importance of channel morphology on debris mobility:

- The model is based on field data for input parameters, including volumes of landslide source, entrainment and deposition, and 35 mapped cross-sections;
- Velocities, debris height and active volume were calculated and compared with field estimates;
- Entrainment of colluvial material occurred at the base of a rock cliff; significantly increased mobility
- Addition of water and deposition of bouldery debris front material at a drainage line confluence increased mobility of the wetter and finer debris.

Detailed mapping of the debris trail enabled a representative analysis to be conducted from which realistic debris flow parameters were obtained. These resulted in close matches between measured and calculated average debris heights and velocities (see lower graphical plot) and provided a useful check on the debris mobility and debris height for design of a check dam located in the former quarry area (c. CH 300-320).

Figure 6.2.9 – Analytical model of the Lei Pui Street channelised debris flow (MGSL, 2002)

6.3 SITE FORMATION

6.3.1 Introduction

From an engineering geological perspective site formation is primarily concerned with classifying the material requiring excavation in terms of its geological structure, engineering properties, excavatability, and volumes to ensure the maximum reuse of materials and the safe and economic formation of slopes. The influence of ground conditions on blasting design, rock mass disturbance and consequent stabilisation measures is also important.

The initial objective is to obtain a general picture of the site with regard to material distribution, geomorphology and groundwater regime in order to assess potential problem areas and engineering approaches including method of excavation, slope layouts and ground reinforcement, etc.

Relevant engineering geological issues relate to the following:

- natural terrain hazards,
- slope stability,
- marine reclamations,
- characterisation of materials for re-use,
- rock mass excavatability, and
- assessment of external factors which might affect or be affected by the site formation works.

As many of the issues listed above are discussed in other sections, this section concentrates on rock mass excavatability, the formation of new slopes within the site formation works, and rehabilitation of rock faces by either re-profiling or installation of stabilisation works. Section 6.8 provides a summary of the very large site formation works for the Hong Kong International Airport at Chek Lap Kok. While the focus was on marine reclamation, a substantial part the site formation works involved the levelling of Chek Lap Kok and Lam Chau islands which provided over 100 million cubic metres of fill materials (Plant *et al.*, 1998). Much of this was in the form of excavated rock (Section 6.3.3).

Reference can also be made to the Ting Kau case study in Section 6.4.4 which provides an example of the design and construction of a major rock cutting. Muir *et al.* (1986) give details of the engineering geological aspects of investigation, design and construction of large rock slopes for the Kornhill Development. Choy *et al.* (1987) provide a detailed account of engineering geological work carried out in assessing the extension of Anderson Road Quarry.

With regard to planning site formations in soil, the example of Tuen Mun Area 19 given in Section 6.3.7 highlights the importance of adverse geological materials and hydrogeology, and the need to consider natural terrain hazards during the planning stage for all site formations.

6.3.2 Material Classification

An important component of site formation works is development of a geological model (Chapter 3) to allow a carefully planned ground investigation, ensure that representative samples of the relevant types of rocks and soils are obtained, as well as information with respect to hydrogeology and overall stability.

The use of geophysical techniques such as seismic refraction surveying can allow rapid assessment of large areas for indirect estimates of weathering profiles and rock mass properties (Chapter 3). However, calibration with data from drillholes and from mapping of exposures is usually required to improve the resolution of the survey results (Muir *et al.*, 1986).

The material requiring excavation should be classified in terms of its engineering properties based on the results of ground investigation drillhole samples and laboratory tests. Generic properties of commonly occurring plutonic and volcanic rocks, given in Sections 5.2 and 5.3, can provide a basis for initial assessments of the suitability of materials for re-use and parameters for use in slope stability analysis.

6.3.3 Rock Mass Characterisation

Overview

Ground models are required to characterise blocks of ground with similar engineering properties to facilitate the planning and management of the site formation works. A key issue is the identification of rockhead, which in this context is often taken to be the level above which material is rippable, i.e. it can be excavated by hand or with power tools, and below which the ground requires blasting.

Major geological structures, structural domains and changes in rock type are particularly important, as these may influence the weathering patterns (Muir *et al.*, 1986) which affect the rockhead and possibly the overall design of the slopes (see also the Ting Kau cutting example in Section 6.4.4). The use of rock mass classification systems which relate the rock properties to engineering parameters can also be beneficial (Chapter 3).

Typical weathering characteristics of the main plutonic and volcanic rocks are given in Sections 4.4, 5.2 and 5.3. When combined with a knowledge of the geological structure and rock types within the site, initial assessments of rockhead variability, excavatability and blasting requirements can be made. Site-specific ground investigations and laboratory testing are used to refine the initial engineering geological characterisation of the site for detailed design purposes.

Rockhead

Accurate definition of rockhead is important for estimating the quantities of rock and soil materials to be excavated, and for planning re-use of the excavated materials. It is also essential for design of slopes in mixed ground, especially where the slopes are constrained by a site boundary at the crest. Overestimations of rockhead level can lead to expensive modifications being required during construction if insufficient allowance for variations has been made in the design (Muir *et al.*, 1986).

Geophysical surveys such as seismic refraction/reflection, gravity (Figure 3.4.1) and resistivity have been used to delineate apparent contrasts in material types, with varying degrees of success, depending on the complexity of the weathering profile (Collar *et al.*, 2000; Muir *et al.*, 1986) and the density of investigation drillholes. However, interpretation of geophysical records requires considerable skill and experience. For example, zones of large corestones can be mistaken for rockhead by geophysical surveys and can present problems when forming stable slopes if they are cut at a steep angle. Large corestones embedded in a soil matrix also need to be blasted or split on an individual basis.

Excavatability

The two most important factors in determining the ease by which rock masses can be excavated are discontinuity spacing and compressive strength. Choy & Irfan (1986) developed excavatability charts for the plutonic and volcanic rocks at Anderson Road quarry (Figure 6.3.1). The difference between the two rock types is primarily due to their different ranges of joint spacing at the quarry. The boundaries between the methods of excavation shown in the figure were based on Franklin *et al.* (1971) and will vary depending on the size and power of the plant available on site and the orientation of major discontinuity sets relative to the excavation surface.

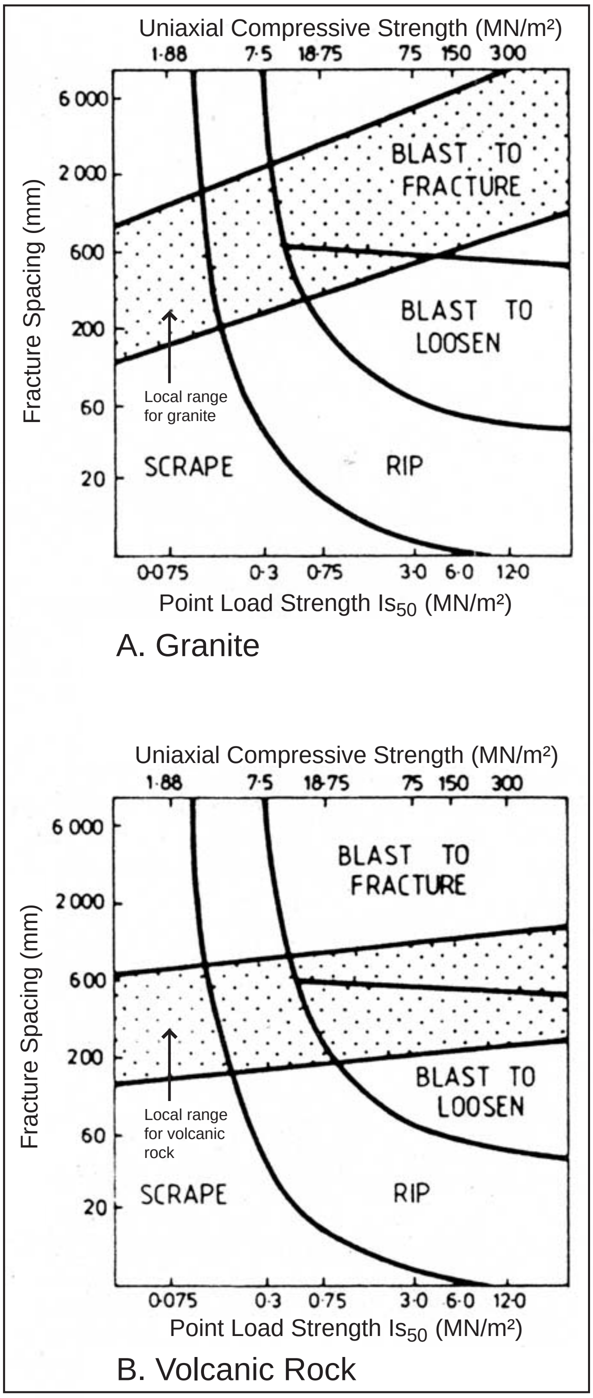


Figure 6.3.1 – Excavatability charts for granite and volcanic rock masses at Anderson Road Quarries (after Choy & Irfan, 1986)

Other rock mass characteristics that may affect excavatability include joint condition, persistence and degree of disturbance of the rock mass. However, their influence is likely to be dependent on rock type and site-specific conditions. Alternative methods of estimating excavatibility have been developed, based on rock mass classifications and/or seismic velocity (Bieniawski, 1989), but their applicability is also likely to be influenced by site-specific conditions.

During the site formation works for the Hong Kong International Airport, blasting trials were carried out to optimize efficiency, fragmentation and throw (Hawley & Keller, 1992). Engineering geological mapping was used to characterise the rock mass and provide ground reference conditions for assessment of the trials and design of the full-scale production blasts. Subsequent production blasts were normally carried out twice a day with each blast producing about 80,000 m^3 of fragmented rock (Plant *et al.*, 1998).

Slope Profile

Section 6.4 describes issues concerning slope stability. The overall design of slope profiles for site formation works is based on an interpretation of the site investigations carried out prior to construction. The design may later need to be refined if unexpected conditions are encountered during excavation. The development of a geological model for the site (see Chapter 3), updated as new information is received from additional investigations and engineering geological mapping of excavations, facilitates the design process and enables any necessary design changes during construction to be efficiently implemented (Muir *et al.*, 1986).

Determination of optimum slope profiles in rock is primarily dependent on the geological structure and rock mass quality. Particular attention should be paid to defining structural domains between which variations in geological structure or differences in the spacing and condition of joint sets occur (see the Ting Kau cutting example in Section 6.4.4).

The formation of smooth faces and regular berms is difficult where a major discontinuity set is orientated about 5°-20° to the plane of the face. This may cause overhangs where the set dips into the face, ragged re-entrant edges where the set strikes at a low angle to the face, or local failures and loss of berms where the set dips out of the face.

Depending on site constraints, it may be possible to improve overall stability by re-designing the plan layout or vertical profile of the slope, particularly when a relatively persistent joint set may cause large planar failures (GCO, 1984). This strategy was adopted during the design and construction of the Ting Kau cutting (Section 6.4.4) where a stable face was formed along the primary joint surfaces without the need to use pre-split blasting techniques.

Critical reviews are necessary to determine face monitoring requirements during slope formation works. For temporary benches, monitoring is usually limited to visual observations only. However, more intensive monitoring may be required as excavation moves closer to the final face. The degree of monitoring will depend on many factors such as:

- the intended end-use of the area,
- sensitivity of facilities behind the slope crest,
- the presence of geological features critical to stability, and
- any sign of a deterioration in stability.

Typical monitoring techniques include:

- surveyed movement monuments,
- laser scanning of the slope face,
- crack displacement measurements,
- extensometers, and
- inclinometers.

Optimisation of the monitoring strategy is facilitated by detailed engineering geological mapping and initial observations of any displacements in the vicinity of the final face.

6.3.4 Blasting

The control of flyrock is an important safety issue, particularly when blasting near populated areas. Although flyrock is primarily generated by inappropriate blasting arrangements, such as inadequate stemming or detonation delay sequences, the blasting arrangements must also take into account local rock mass conditions.

Site-specific rock mass properties need to be adequately assessed for each blast with reference to the properties and performance of previously blasted masses, including trial blast panels carried out specifically to provide a reference for the assessment of future blasts. Local changes in rock type, degree of weathering, block size and shape, disturbance by previous blasts and differences in the relative orientation of geological structure may need to be accommodated in the blasting arrangements to eliminate hazardous flyrock, minimise overbreak and optimise fragmentation.

Massey & Siu (2003) report an incident in 2003, where flyrock was ejected onto New Clear Water Bay Road up to 230 m away from the blast site. The incident was likely to have been related to a zone of weathered and fractured rock (Figure 6.3.2). It was inferred that partial toe con finement of a relatively fractured rock mass had occurred due to a high point in the sub-drill level located in the adjacent, less fractured rock. This is considered to have caused rock fragments to be ejected upwards instead of in the intended direction of throw (Figure 6.3.3).As a result of the investigation, it was recommended that for this particularly sensitive site, the Geologist and Blast Designer should assess the site conditions, verify the blasting design assumptions made, and revise the blast design if necessary , taking full cognizance of the geology of each location before blasting. It was also advised that feedback from the driller should be assessed with regard to any weak or jointed and weathered seams and that the consultation and review process should be recorded.

Another important engineering geological issue is the vulnerability of nearby slopes to disturbance caused by blasting. Failures of rock slopes in close proximity to blasting operations have been recorded in the past. Examples include:

- The 1981 Yip Kan Street rockslide involving about 1,240 m^3 of material which slid on a chlorite-coated joint dipping at about 20°. The landslide occurred in a slope adjacent to a site where intermittent blasting was being carried out. Cracking was noted after the passage of a heavy rainstorm and failure occurred about a week afterwards, following blasting which had been carried out two days previously (Hencher, 1981, 2000).
- The 1991 Shau Kei Wan rockslide involving about 2,000 m^3 of material which slid on an unfavourably orientated joint in dry weather , about one minute after a blast had been carried out within 15 m behind the slope crest. Extensive blast damage was

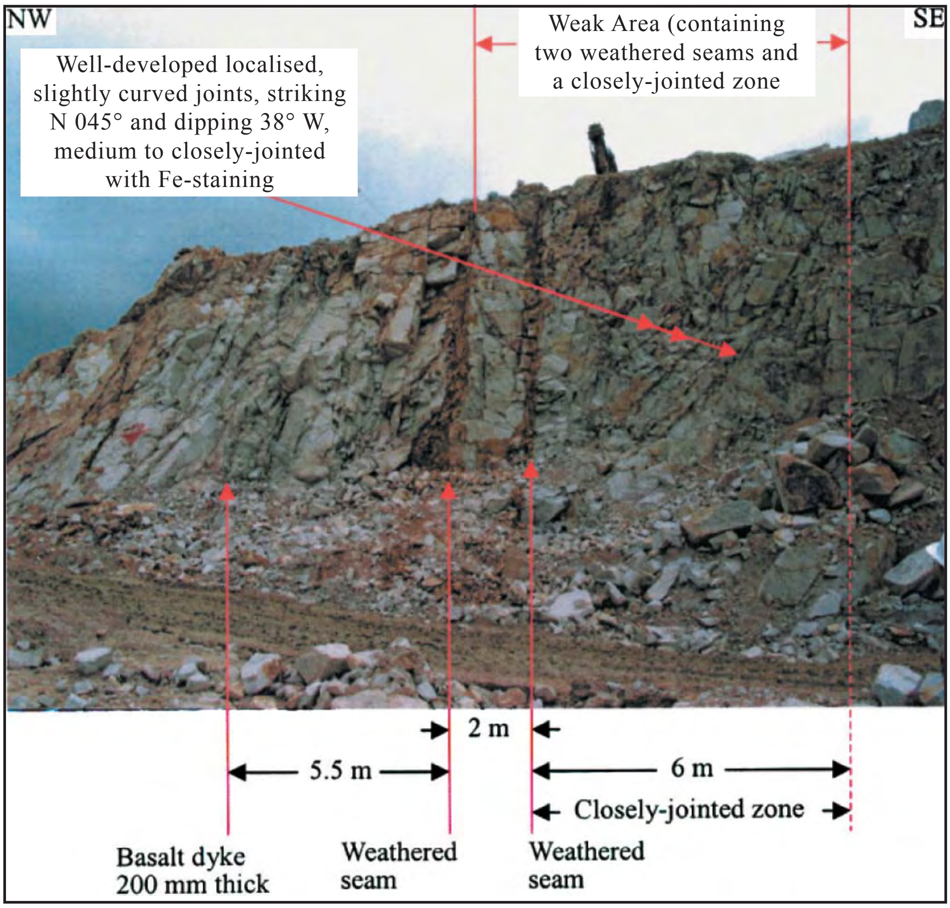


Figure 6.3.2 – Weak area 45 m southwest of the blast location near New Clear Water Bay Road (Massey & Siu, 2003)

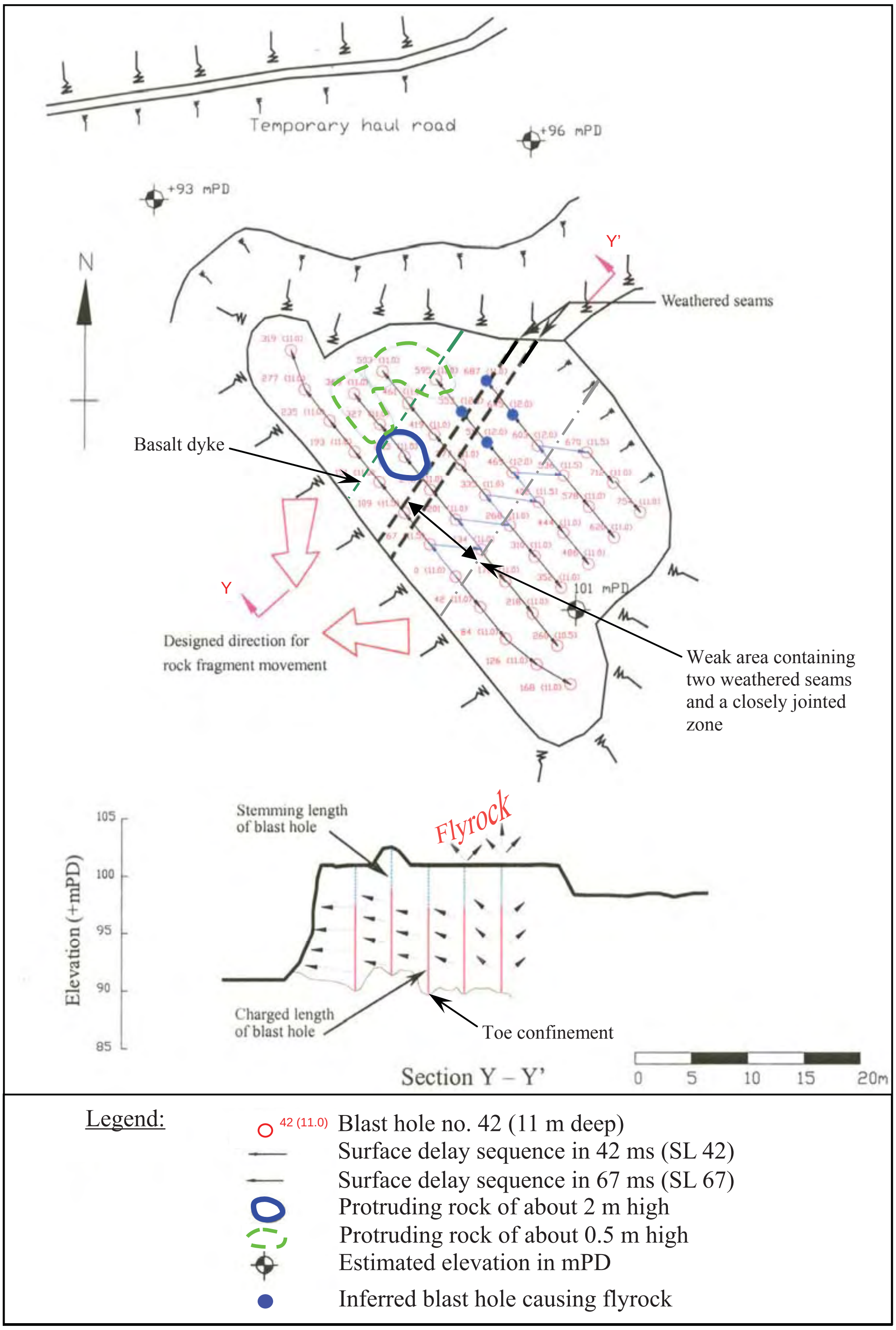


Figure 6.3.3 – Inferred mechanism of flyrock occurrence (after Massey & Siu, 2003)

noted in the rock mass, with spalling and opening of joints due to shock waves and gas pressure (Evans & Irfan, 1991; Irfan & Evans, 1998).

- The 1997 Sau Mau Ping Road rockslide involving about 1,000 m^3 of material which slid on sheeting joints dipping at about 25 ° (Figure 6.3.4) in dry weather, a few seconds after a blast had been carried out about 3 m behind the slope crest (Figure 6.3.5). Extensive blast damage was noted in the rock mass, with dilation of the joints and movement of blocks (Figure 6.3.4) due to shock waves and gas pressure (Leung *et al.*, 1999).

The effects of blasting on the stability of rock slopes are discussed in Nicholls *et al.* (1992), and methods for assessing the stability of nearby slopes subjected

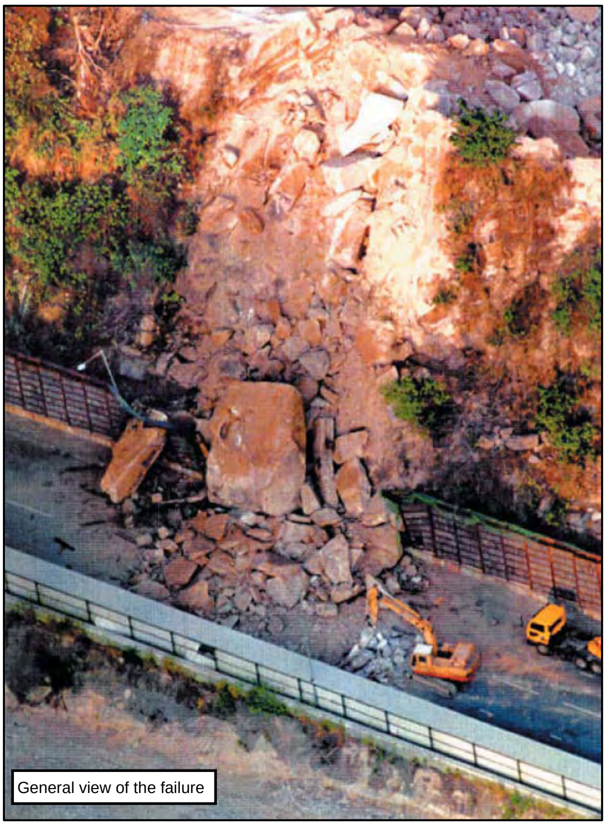


Figure 6.3.4 – Blast-induced failure during site formation works at Sau Mau Ping Road in 1997 (Leung et al., 1999)

to blasting vibration are suggested by Wong & Pang (1992). The effects of blasting gas pressures are further discussed in Blastronics Pty Ltd (2000).

Analytical approaches all require the engineering geological properties of the rock mass to be determined. These include the orientation and persistence of the discontinuities, block size and shape, and discontinuity characteristics (Barton, 1990). Wong & Pang (1992) also recommend that blast vibrations at critical slopes are monitored, with inspections being carried out to check for the presence of irrecoverable 'post-peak' displacements.

Yeung (1998) provides an account of rock blasting control at the Ting Kau Bridge site where the energy approach suggested by Wong & Pang (1992) was implemented.

6.3.5 Hydrogeology

The hydrogeological impact of large site formations needs to be carefully considered, along with the need for regional hydrogeological studies. General principles are discussed in Section 4.6 and specific issues related to slope stability are discussed in Sections 4.6.4 and 6.4.

Potential modifications to the groundwater regime caused by the construction of a large site formation include:

- Lowering of the base groundwater table in the surrounding hillside where deep excavations are carried out.
- Increase in infiltration where large areas are left unprotected, which might lead to higher groundwater occurring in slopes at lower elevations. For example, the 50,000 m^3 Fat Kwong Street landslide in 1971 was triggered by very high groundwater which was considered to be the result of increased infiltration into large expanses of unprotected ground above the cutting (O'Rorke, 1972).
- Increase in the base groundwater table where filling for site formation works raises the effective head at the lowest discharge level and/or retards groundwater drainage from the hillside (Jiao, 2000b; Jiao *et al.*, 2001). A conceptual model with regard to marine reclamations is shown in Figure 6.3.6. A similar scenario could be applicable where fill platforms are constructed in low-lying areas above sea level if an adequate drainage blanket is not constructed.

6.3.6 Rehabilitation of Rock Faces

Most engineering geological considerations with regard to the rehabilitation of quarries and former borrow areas are similar to those pertaining to site formation and slope engineering in general (Section 6.4). This is particularly so where there are few constraints for further excavation into the hillside to achieve stable, permanent slopes. In such cases, the angle of the permanent slopes can often be designed to avoid a large percentage of potential failures defined by weak geological materials and discontinuities.

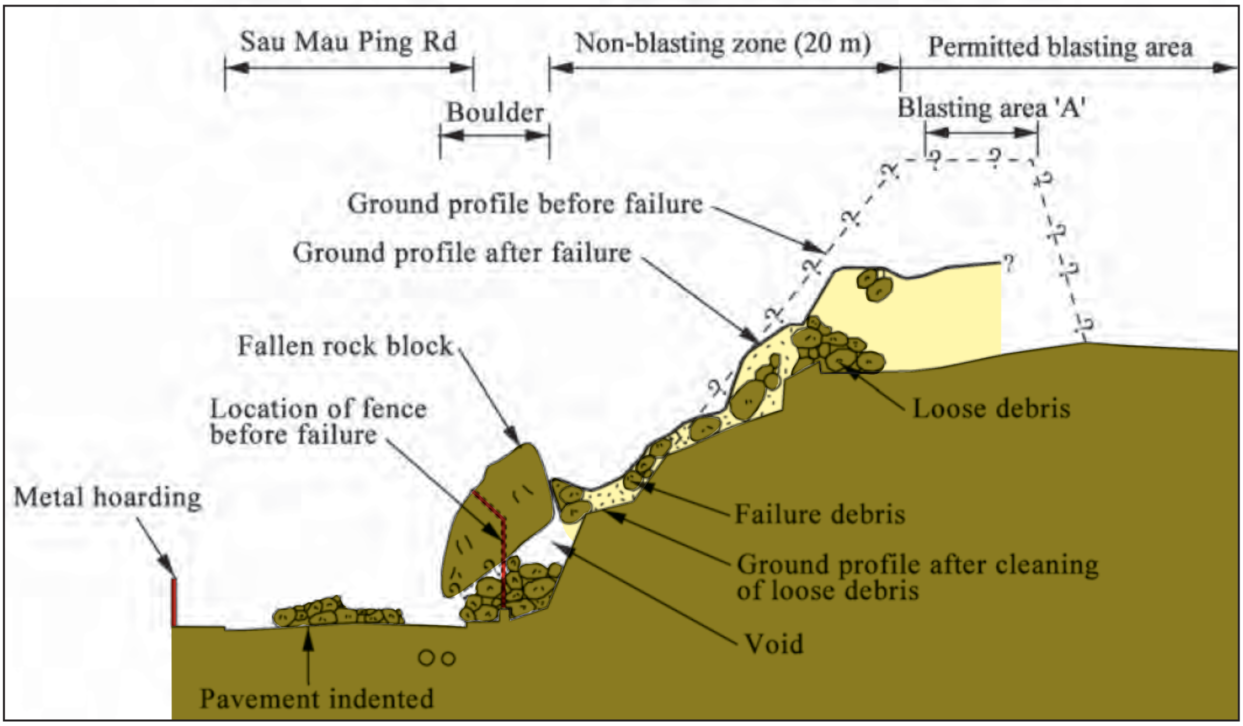


Figure 6.3.5 – Section through the blast-induced failure at Sau Mau Ping Road (Leung et al., 1999)

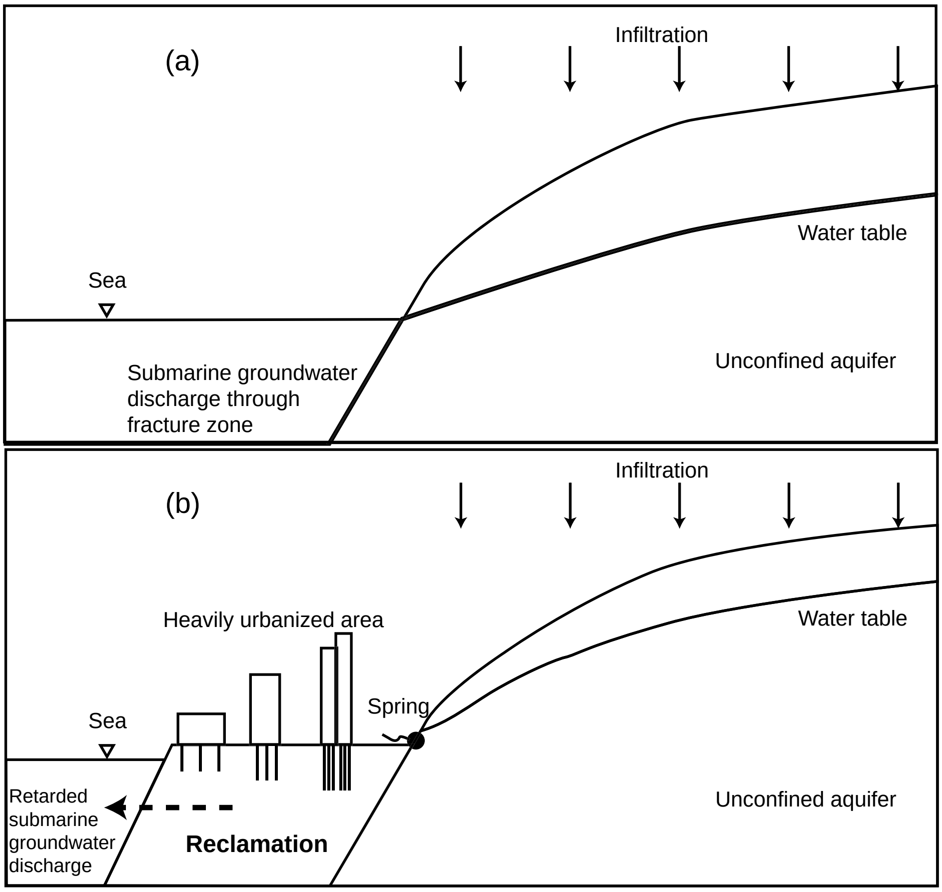


Figure 6.3.6 – Potential effect of reclamation works on raising the overall groundwater table (Jiao, 2000b)

Early examples include the design for the final face for Tai Sheung Kok quarry reported by Endicott *et al.* (1981) and the engineering geological studies carried out for the extension to Anderson Road quarries by Choy & Irfan (1986).

Lam & Siu (2000, 2002) provide reviews of more recent quarry rehabilitation strategies, with the focus being on environmental acceptability These strategies generally fall into two categories, depending on toe and crest constraints:

(i) Overall slope angle of final landform about 35º, formed by placing a vegetated soil layer over a rockfill slope against the rock face (e.g. Shek O and Lamma quarries).
(ii) Overall slope angle greater than 55º, with sloping, vegetated soil layer placed only on the rock berms (e.g. Anderson Road, Lam Tei and Turret Hill quarries).

Strategy (ii) requires more engineering geological input because it leaves most of the rock face uncovered. The potential for instability caused by locally adverse jointing and opening-up of fissures by root-wedging needs to be considered.

Where there are severe constraints such that little additional excavation can be carried out to achieve permanent stability, e.g. old rock faces formed by bulk blasting with existing developments at the toe and/or crest, the additional effects of the previous slope formation works and deterioration need to be investigated in detail. Potential effects include:

- irregular, partially unstable faces with loose blocks that have deteriorated over time, and
- over-steep faces with toppling joints, disturbed zones and cracks caused by back-break during blasting which may penetrate into the rock mass by up to about one-half of the face height.

In such cases, engineering geological studies need to be carried out to investigate the stability of individual rock blocks and discrete masses to enable reasonable estimates of the extent and cost of stabilisation works. Where close-up access to the face is difficult and/or dangerous, remote methods such as photogrammetry or laser scanning can be used to map the rock face. However, estimates of the extent and cost of stabilisation requirements may still be very approximate.

Particular attention should also be paid to defining the depth of any disturbed zones containing tension cracks, open joints and fissures which could lead to further deterioration and cause difficulties in the installation of any slope reinforcement works. Where fresh rockfalls have occurred, inspections of the scars can be carried out (with due regard to safety) to investigate the depth and extent of distress. Clearance of vegetation at the crest of the slope and investigation by trial trenches, horizontal drillholes and/or geophysical traverses may help to identify the extent of such zones.

6.3.7 Site Formation in Tuen Mun Area 19

General

This example illustrates the effect of an adverse geological setting, where about 80% of the formed site was eventually required to accommodate a succession of slope remedial works and natural terrain hazard mitigation measures (Figure 6.3.7).

The site formation was planned in about 1976, initially to provide material for reclamation, and was subsequently to be used for housing developments. At that time, little was known of the geotechnical properties of the weathered andesite of the Tuen Mun Formation (Section 5.3.6), or the potential for large scale natural terrain debris flows to affect the site. Acceleration of the housing programme and land clearance problems resulted in the lack of a detailed site investigation before commencement of excavation (Hunt, 1982). This militated against an early appreciation of the ground conditions.

Geological Setting

Area 19 is located on a gently-inclined, coastal foot slope formed in weathered andesite, and is overlooked by steeper slopes of volcaniclastic rocks, partially faulted against granite which forms the Tsing Shan ridgeline (Figure 6.3.8). The weathered profile within the andesite is relatively deep (up to about 40 m) and is overlain by colluvium up to 10 m thick.

Summary of Cut Slope Instability History

The site has been associated with instability since the commencement of excavation, and extensive regrading and drainage measures have been carried out at various times (Figure 6.3.9). Landslides were originally noted in 1977 on slopes which had been cut at 1 on 1.5, and movements continued even after the slope had been reduced to a gradient of 1 on 3.5. Figure 6.3.10 shows a section through one of the most disturbed areas.

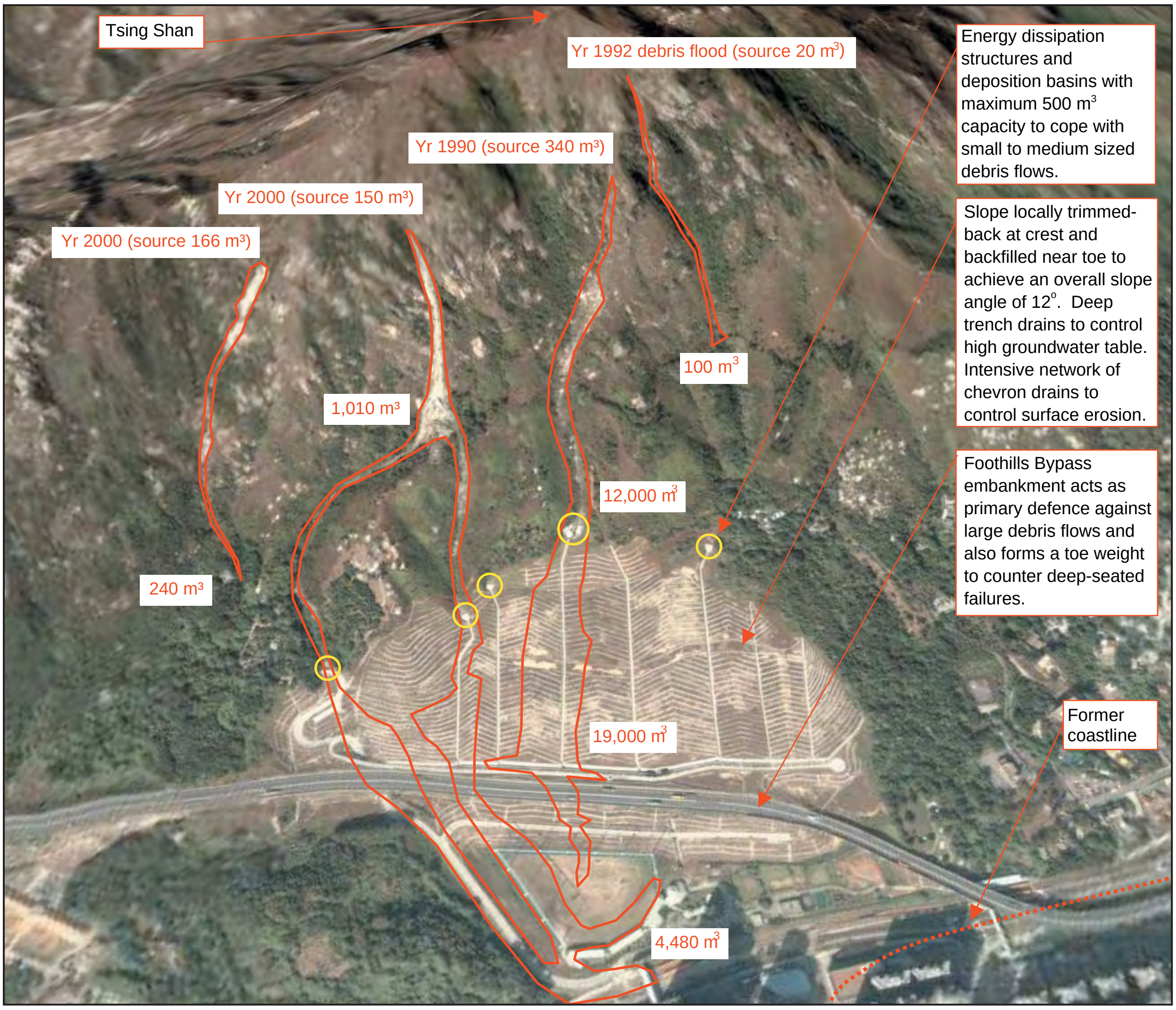


Figure 6.3.7 – Oblique aerial photograph of Tsing Shan and Area 19 showing recent major debris flow and debris flood scars, the 2003 slope treatment works and the 2003 debris flow mitigation measures (volumes refer to the cumulative erosion along the debris trails)

Investigations
Several studies were undertaken during the 1980s to determine the nature of the instabilities in the cut slopes. In 1998, construction began for the Foothills Bypass which traverses the toe of the hillsides at Area 19. Further ground investigation was carried out to characterise the ground conditions and to enhance the existing information, in particular, the nature and extent of the weathered andesite.

Movement Surfaces
The investigations revealed two types of movement surface:

- Type 1 – generally steeply-dipping, polished and slickensided tectonic joints with thin films of clay and manganese oxide, and
- Type 2 – generally low angle, slickensided shear surfaces with remoulded clay in fills and more continuous manganese oxide coatings, occurring below the colluvium/weathered andesite interface, becoming less frequent with depth (Figures 6.3.11 and 6.3.12).

Type 1 surfaces had a residual friction angle of about 18°, while Type 2 surfaces had a range in residual friction angle between 9° and 17° and appeared to form continuous shear surfaces along which the main landslide movements were occurring (Koor *et al.*, 2001).

Groundwater Regime
Monitoring of groundwater in Area 19 has been carried out since 1978. This has shown that perching of groundwater levels can develop within the thick overlying colluvium during the wet season due to the permeability contrast between the colluvium and

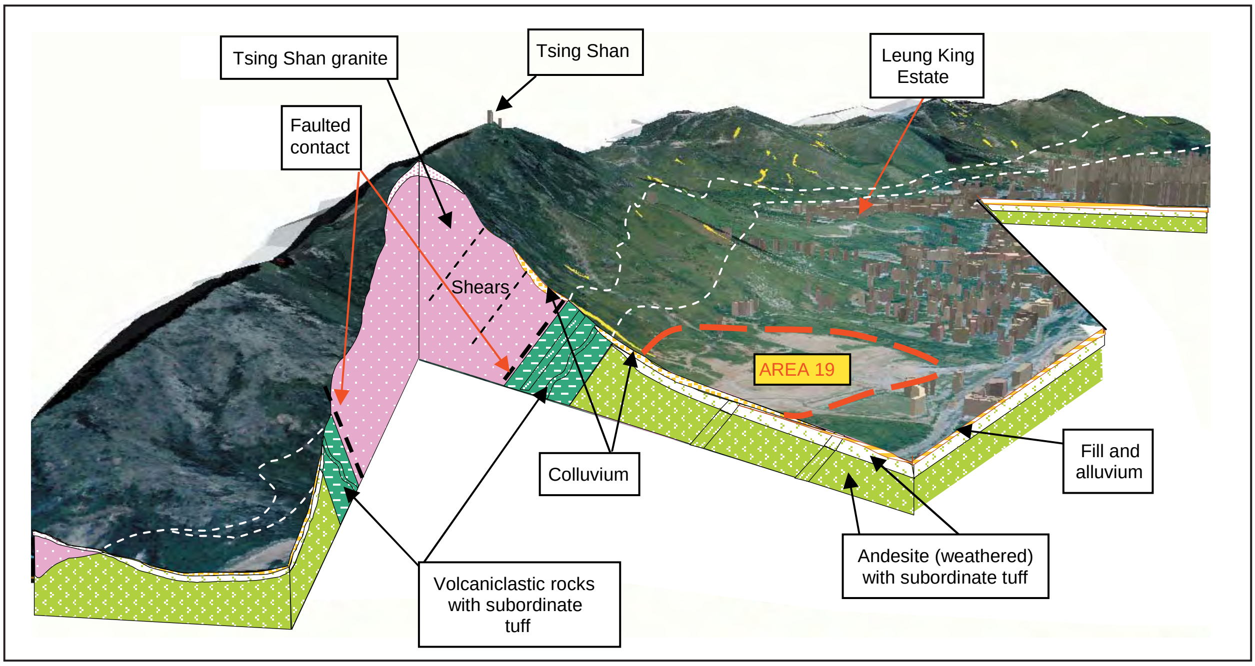


Figure 6.3.8 – Geological setting in the vicinity of Area 19 (looking northwest)

the completely decomposed andesite. In general, no large fluctuations in groundwater levels in response to individual rainstorms have been observed (Taylor & Hearn, 2000; Taylor & Hadley, 2000).

Piezometers installed at the base of the completely to highly decomposed andesite generally indicated similar maximum groundwater levels to those near the base of the colluvium. Maximum groundwater levels in all strata were very close to ground level (Figure 6.3.10). Seepage mapping (Figures 6.3.9 and 6.3.10) indicated that the slope toe area is wet throughout the year. The trench drains installed into the weathered andesite for the Foothills Bypass also showed constant discharges.

Ground Model

Most of the shear surfaces probably pre-date the site formation works. This is supported by the presence of large-scale natural terrain landslides associated with low-angle shear planes at or just below the interface of colluvium and weathered andesite in the Tsing Shan foothills (Section 5.3.6).

The low residual strength of the andesite suggests that low-friction clay infills may be significant. Halloysite and kaolinite clay infills along discontinuities in weathered granitic and volcanic rocks have been identified at the sites of several large-scale landslides in Hong Kong (Campbell & Parry, 2002). The residual friction angle of kaolin varies from about 20° to 12°, depending on the proportion of kaolinite to halloysite. As residual friction angles as low as 9° have been measured in Area 19, there is a strong possibility that lower-friction clay minerals such as smectite may be present in the shear plane infills (e.g. Fookes, 1997b).

Most of the recent failures and pre-existing shear surfaces occurred at relatively shallow levels (at the base of the colluvium and within the uppermost 5 m to 10 m of the weathered andesite) which is corroborated by inclinometer readings and seepage observations (see Figures 6.3.9 and 6.3.10). This may be related to stress relief creating and opening up discontinuities near the surface.

The instabilities in Area 19 are situated in gently sloping ground where stream courses that drain the higher slopes of Tsing Shan encounter the andesite (Figures 6.3.8 and 6.3.9). The groundwater regime is rather complex, due to the presence of the overlying colluvium, soil pipes, eroded open fissures along relict joints, and relatively impermeable pre-existing shear planes. Direct groundwater infiltration and recharge from stormwater probably led to development of local perched water tables at the colluvium/weathered andesite interface and upon the flat-lying shear surfaces within the andesite.

The low elevation of the site near the base of a high concave hillside and its close proximity to the

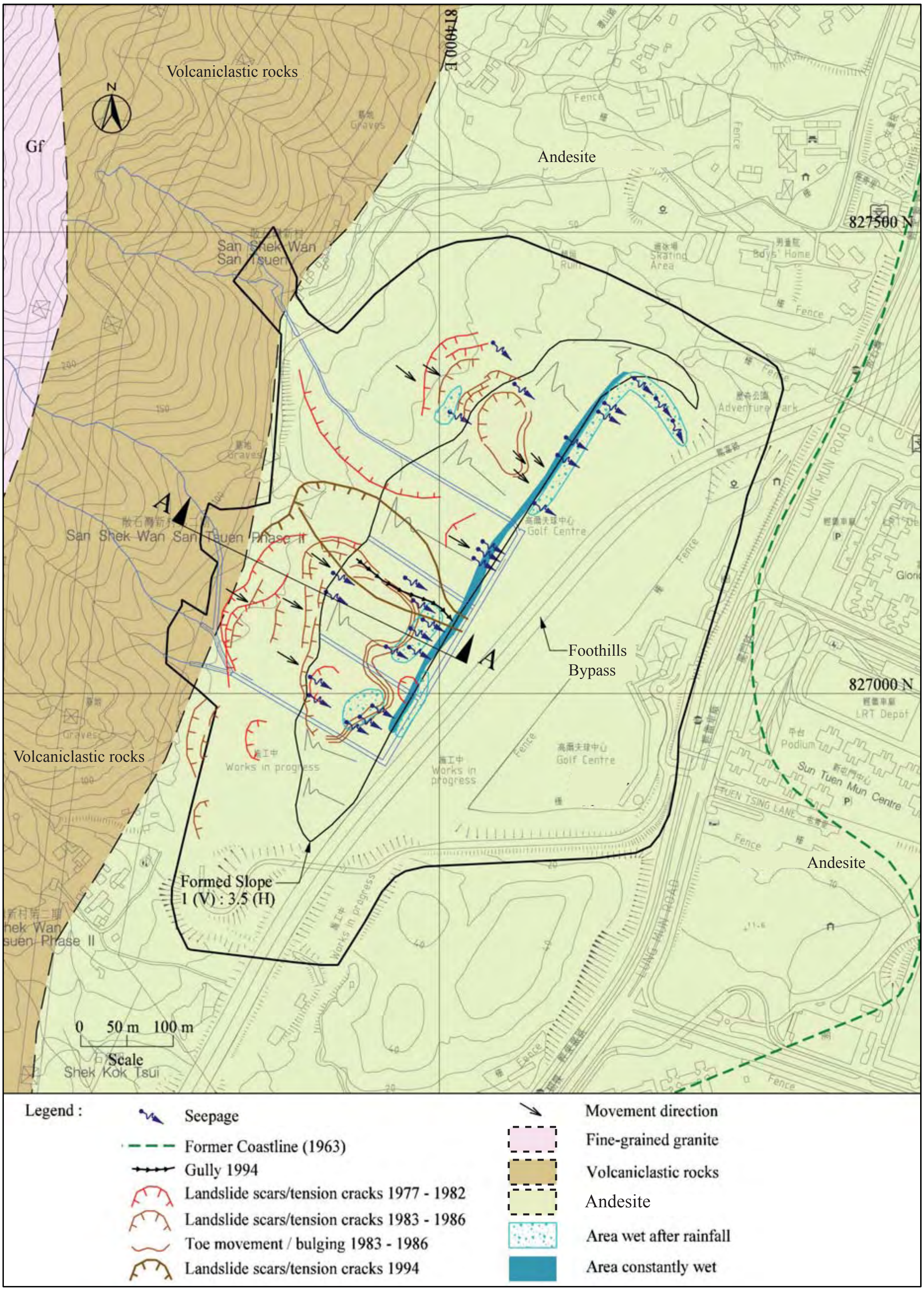


Figure 6.3.9 – Plan of Area 19 showing instability history

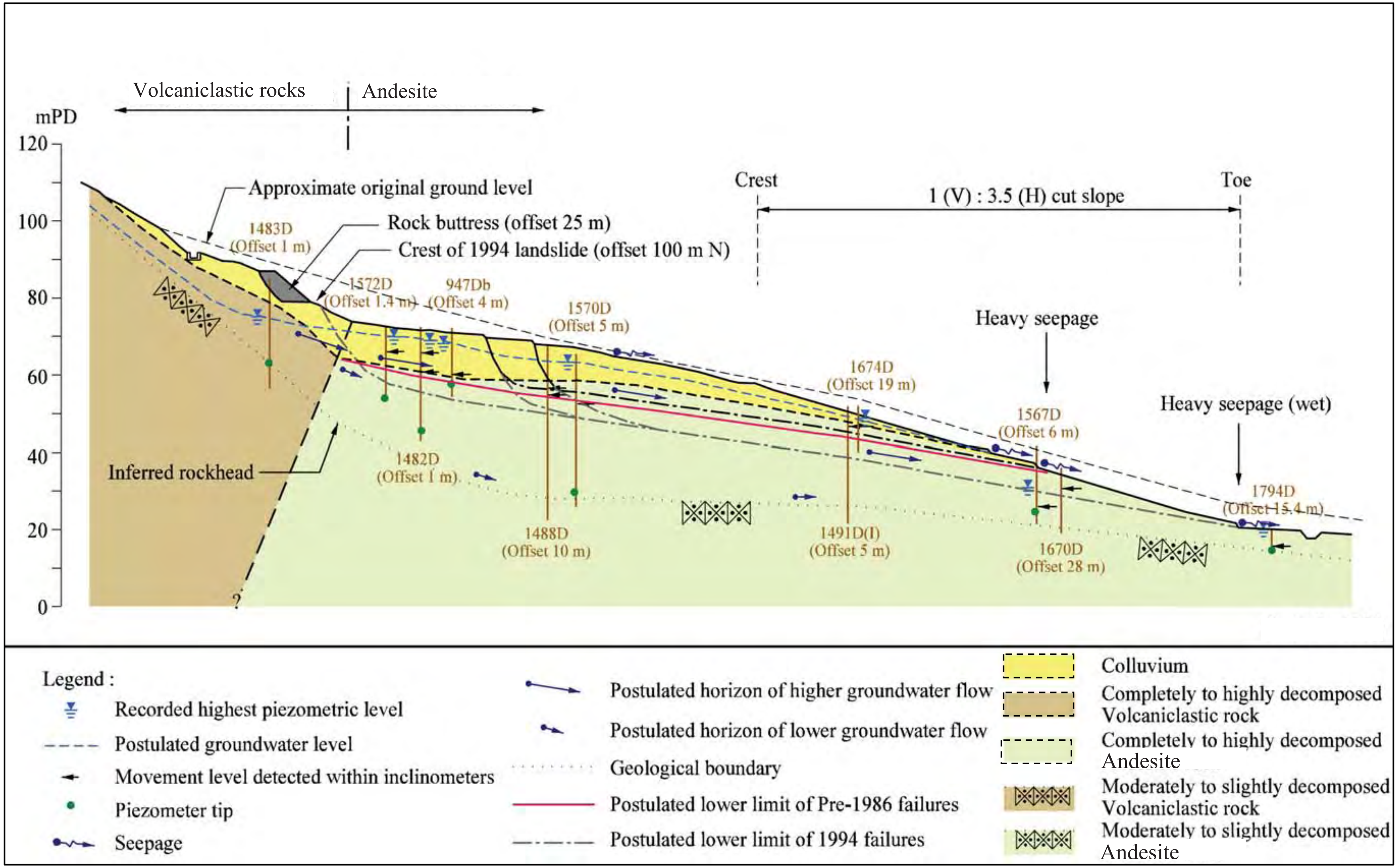


Figure 6.3.10 – Section A–A through Area 19 showing geology, deep-seated movements and high groundwater table

Figure 6.3.11 – Undulating Type 2 shear surface within completely decomposed andesite (photograph by N.P. Koor)

Figure 6.3.12 – Close-up of a Type 2 shear surface with polished and slickensided surfaces and grey clay infill (photograph by N.P. Koor)

former shoreline (Figures 6.3.7 and 6.3.9) favour the development of a high base groundwater table. This situation may have also been exacerbated by the nearby reclamation works (Jiao *et al.*, 2000b) which may have caused a permanent rise in base groundwater levels (Figure 6.3.6).

Given the sensitivity of the silt to softening when wetted, the uppermost layers of andesite were probably weakened by repeated expansion and absorption of water. The more intense weathering in the uppermost 10 m of the saprolite and groundwater flow along fissures has probably led to the concentration of low-friction clay weathering products in the undulating stress-relief joints.

Debris Flow Hazards

The extent of the debris flows which affected the site in 1990 and 2000 (King, 2001a,b,c) are shown in Figure 6.3.7. A geological model of the eastern flank of Tsing Shan showing the various terrain units associated with different geomorphological processes is given in Figure 4.5.2. The source zones of the debris flows are located in weathered granite and colluvium-infilled depressions, and have slope angles of about 35°- 45°. The initially steep terrain gave rise to fast-moving debris flows which eroded and entrained a large amount of loose colluvium accumulated within drainage lines. The debris flows also eroded significant quantities of colluvium and completely decomposed andesite lower down within Area 19. The 1990 debris flow (Figure 4.5.3), which travelled for over 1 km, involved the displacement of about 19,000 m^3 of material and is the largest recorded debris flow in Hong Kong.

Remediation

The Foothills Bypass project adopted several strategies to deal with the instabilities and debris flow hazards of Area 19. The slope was re-profiled to 12º (the design residual shear strength value), the road embankment across the foot of the unstable area was constructed to act as a toe weight, and 6 m deep trench drains were installed into the weathered andesite. A dense system of surface drains was also installed to protect the easily erodible slope surface (Figure 6.3.7).

The Foothills Bypass embankment also acts as the primary defence against large debris flows that might affect the road itself. Energy dissipation structures and deposition basins at the intersection of the drainage lines and the site boundary were constructed to mitigate against smaller debris flows up to about 500 m^3 in volume (Thorn & Koor, 2002).

This example illustrates the need to identify and manage geotechnical risks for site formations using state-of-knowledge engineering geological perspective, principles and practice, particularly at the planning/feasibility and initial investigation stages. In this case, the prime considerations related to potentially adverse geological materials, adverse hydrogeology and the need to assess systematically the impact of potential natural terrain hazards on the development.

6.4 SLOPE STABILITY

6.4.1 Introduction

Overview

An overview of the evolution of slope engineering practice in Hong Kong is given in Wong (2001). Before the 1970s, slopes were designed largely in an empirical manner with little geotechnical input. Intense urban development in the hilly terrain, combined with thick weathering profiles and heavy, seasonal rain, contributed to some notable slope failures with associated loss of life such as the Sau Mau Ping fill slope disasters in 1972 and 1976 (Section 5.9) and the Po Shan Road disaster in 1972 (Figure 6.4.1 and Section 6.4.4). In response to the mounting public concern, the Government of Hong Kong established the Geotechnical Control Office in 1977 (now the Geotechnical Engineering Ofice) with the main aims being to progressively improve slope safety and geotechnical practice in Hong Kong.

Figure 6.4.1 – 1972 Po Shan Road landslide

Over the last 30 years or so, slope engineering practice has evolved to meet society's demands. However, based on landslide statistics from 1997 to 1999, the overall failure rate of '*engineered*' slopes in terms of major failures (defined as >50 m^3) was lower than that for '*un-engineered*' slopes by only a factor of two (Wong, 2001). Whilst improvements in geotechnical control, slope management and safety awareness have substantially reduced the overall rate of fatalities resulting from landslides when compared to the rate of increase of urban development (Malone, 1998; Chan, 2003b), there is still room for improvement in reducing the failure rate of engineered slopes.

Engineering Geological Issues

There are inherent variability and uncertainties in the geological and hydrogeological conditions of slopes in Hong Kong. Experience has shown that slope stability problems in soils derived *in situ* from the weathering of rock masses (i.e. saprolite) require appropriate elements of geology, geomorphology and hydrology in greater measure than may be necessary in other circumstances. Slope engineering in the regolith of Hong Kong therefore spans the narrowly separated fields of soil mechanics, rock mechanics and engineering geology, and the engineering geological approach is generally the most satisfactory one for these materials (Brand, 1985).

Ho *et al.* (2003) document the key lessons learnt from studies of failures of man-made slopes together with observations from reviews of investigation and design practice based on examination of over 100 slope design or assessment reports. Their findings indicate that the most important factor with regard to major failures is the adoption of an inadequate geological or hydrogeological model in the design of slopes, with the main problems being associated with adverse geological features and adverse groundwater conditions. Martin (2003) and Parry *et al.* (2004b) make similar observations. These findings call for enhanced engineering geological input in slope investigation and design.

This section builds on the references cited above and highlights key engineering geological issues through the use of examples and other references.

Specific issues relevant to the engineering of cut slopes include:

- the need to recognise and consider the implications of relict instability of the natural ground (Section 6.4.2) and previous failures or deterioration of man-made slopes (Section 6.4.3),
- the recognition and assessment of adverse geological structures and lithological variations in both rock and soil (Section 6.4.4),
- consideration and investigation of the hydrogeology of typically heterogeneous ground (Sections 6.4.5 to 6.4.7), and
- consideration and investigation of materials and geological structures which may give rise to difficulties when installing soil nails (Section 6.4.8).

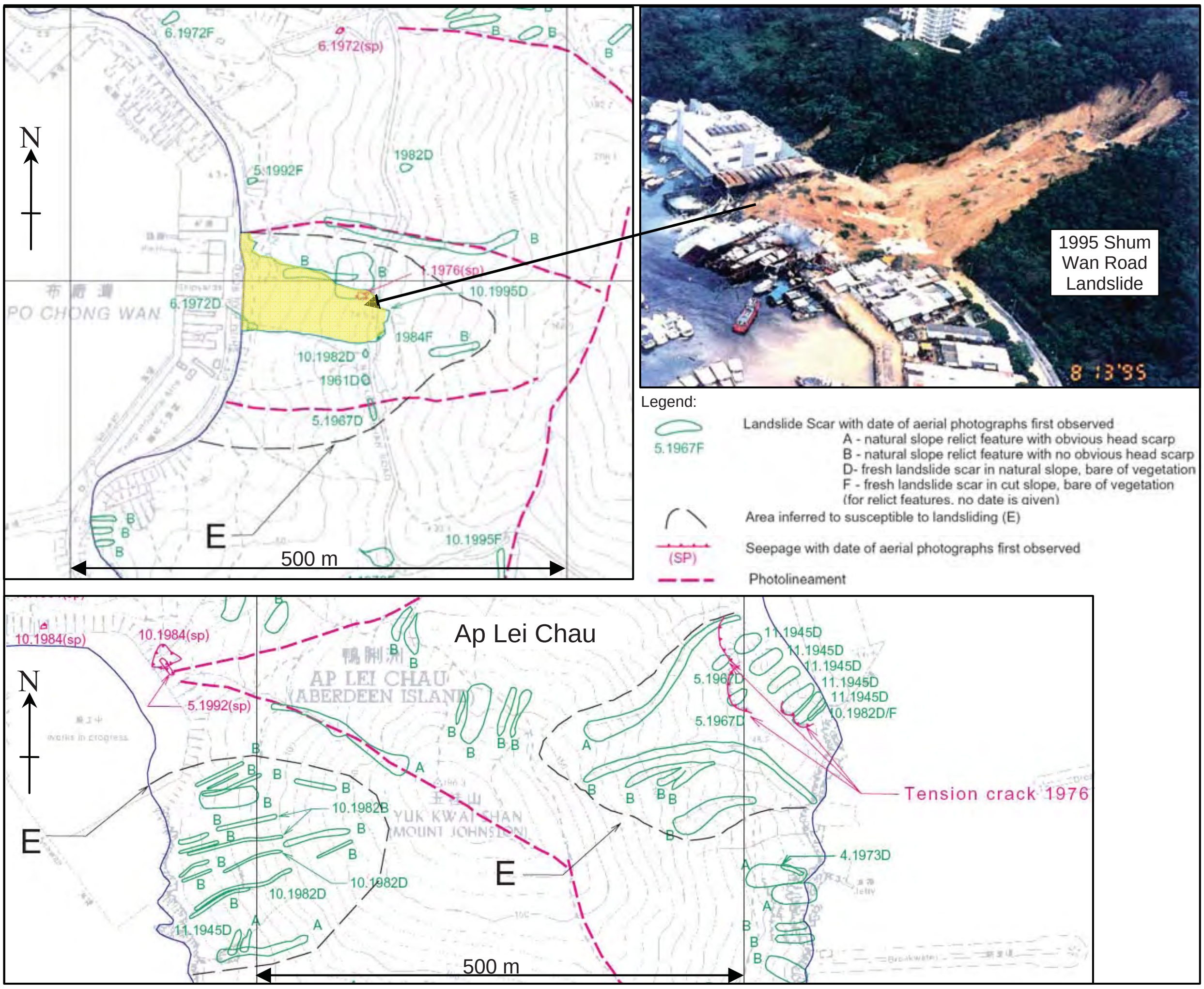


Figure 6.4.2 – 1995 Shum Wan Road landslide and clusters of relict and recent coastal instability identified from API near Aberdeen (after Franks et al., 1999)

The predominant use of slope failures to illustrate key issues is due to the availability of good data from the subsequent investigations. Although each example is used to illustrate a particular issue, most failures are the result of a combination of issues, and these are also listed where relevant. References to other examples which illustrate similar issues are also given.

Many of the examples highlight that while enhanced engineering geological input is very beneficial in the planning, investigation and design stages, it is equally beneficial during construction. Given the inherent uncertainties that may still remain after completion of the design, any adverse variations in ground conditions exposed during construction need to be identified and correctly interpreted in the context of the geological and ground models so that their impact on the existing design can be properly assessed.

Key references and guidance documents that address slope engineering issues in weathered rocks and saprolite include: Deere & Patton (1971), Patton & Deere (1971), GCO (1984, 1987b, 1988a), Malone (1985), Irfan & Woods (1988), Hencher & McNicholl (1995), Martin *et al.* (1995), Irfan (1998b), GEO (2000, 2004d,j), Hencher (2000) and Campbell & Parry (2002).

6.4.2 Relict Instability

General

Relict instability is related to previous movements of natural ground prior to construction. Typical characteristics can include:

- zones of low strength, or weakened materials,
- relict shear surfaces which may be reactivated,
- previous failures controlled by discontinuities,
- tension cracks which may act as local weaknesses or promote the build-up of cleft water pressure,

- dilation of discontinuities with secondary infilling by low-friction clay minerals, and
- unfavourable hydrogeological conditions.

Relict instability may be recognised by detailed API carried out by skilled personnel. However, field verification is required. The processes required to form a representative geological model are outlined in Sections 3.2, 3.3, 4.5 and 6.2.

Instability Near Aberdeen

Figure 6.4.2 shows clusters of relict and recent landslides identified from API during the assessment of geological features related to recent landslides in volcanic rocks in the Phase 2 Aberdeen Study Area (Franks *et al.*, 1999).

The 1995 Shum Wan Road landslide (GEO, 1996c,d) involved the displacement of 26,000 m^3 of material and is located within a zone of possible inherent instability. Natural contributory factors involved:

- a trough of more deeply weathered tuff probably associated with faulting,
- kaolin-rich material near rockhead (see Figure 4.4.16),
- adversely-orientated, kaolin infilled relict joints, and
- a generally high groundwater table with additional perching above the clay-rich surface of rupture.

1997 Ma On Shan Road Landslide Opposite Shing On Temporary Housing Area

The landslide involved the displacement of about 2,800 m^3 of natural materials and the mobilisation of about 300 m^3 of fill on the platform below the cut slope (Figures 6.4.3 and 6.4.4; HAPL, 1999). The hillside contains many relict landslides and incised drainage lines, suggesting an adverse geomorphological

Figure 6.4.3 – 1997 Ma On Shan Road landslide

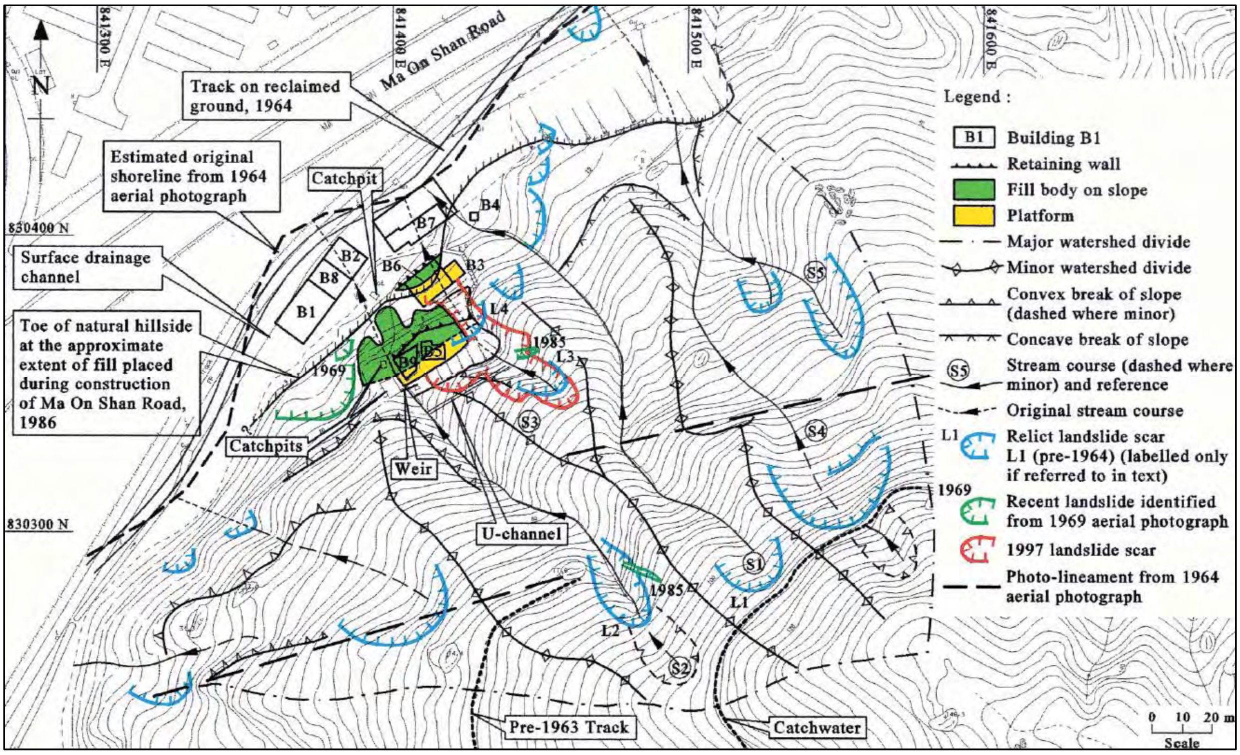


Figure 6.4.4 – Previous instabilities near the 1997 Ma On Shan Road landslide inferred from API (HAPL, 1999)

setting. Contributory factors involved:

- infiltration of surface water from a stream course and numerous soil pipes,
- shallow failure of the cut slope above the fill platform along a planar, kaolin-infilled joint dipping sub-parallel to the slope,
- concentrated surface water flow and retrogressive failure of the relict landslide scar above the cut slope, and
- over-steep former coastal slopes.

1972 Lai Cho Road Landslide Near Kwai Chung

This relict landslide comprising about 45,000 m^3 of material also involved thick kaolin accumulations near rockhead (MGSL, 2002; Thorn *et al.*, 2003) and was re-mobilised by site formation works at the toe (see Figures 4.4.17 and 5.4.4).

Figure 6.4.5 – 1999 Shek Kip Mei Landslide

1999 Shek Kip Mei Landslide

The zone of distress comprised about 6,000 m^3 of material (Figure 6.4.5; FMSW, 2000). Investigation of the landslide revealed:

- a possible relict landslide scar at the site of the 1999 area of distress (inferred from API),
- a zone of infilled tension cracks which extended up the slope beneath the chunamed surface including areas where no obvious signs of distress were noted at the surface (Figure 6.4.6),
- a 10 mm thick, low angle, kaolin and manganese-oxide infilled, slickensided planar discontinuity that extended for at least 60 m across the toe of the cut slope and marked the basal rupture surface of the southern part of the 1999 area of distress (Figure 6.4.6 and Figure 6.4.7), and
- the distress occurred about two days after intense rainfall, suggesting a delayed build-up of groundwater pressure.

Whilst this slope was formed many years ago with only minimal geotechnical input, the example illustrates the importance of identifying past instability from API in order to effectively target ground investigations to locate possible relict tension cracks, rupture

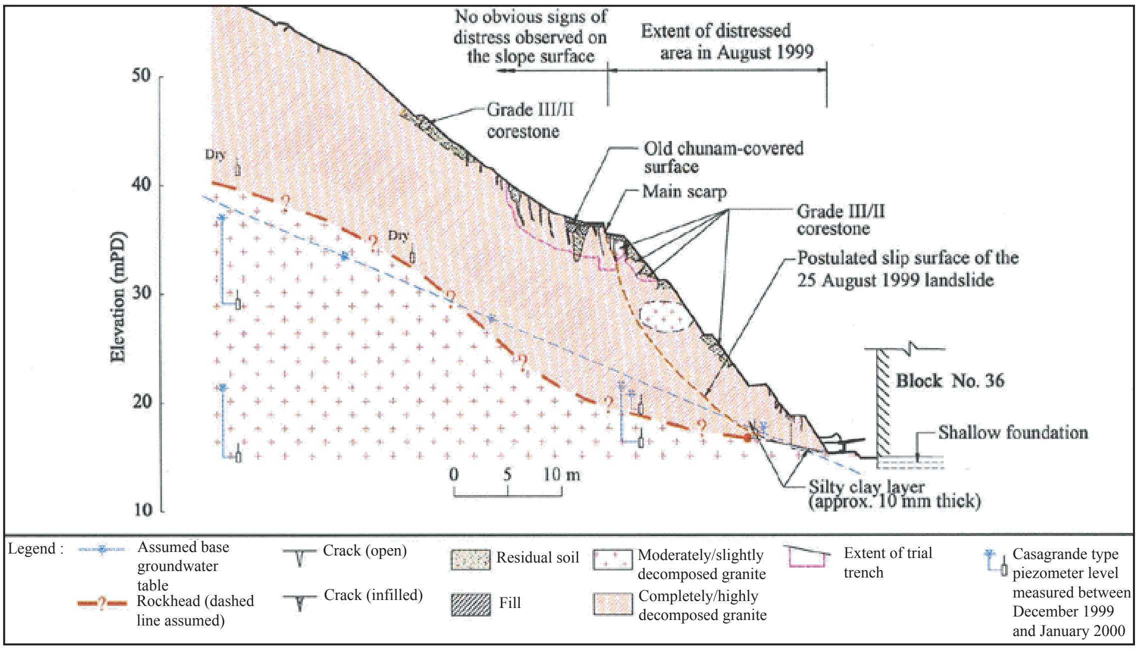


Figure 6.4.6 – Section through the Shek Kip Mei landslide showing old, infilled tension cracks, deep weathering and weakness plane near toe (FMSW, 2000)

Figure 6.4.7 – Laterally persistent discontinuity infilled with slickensided kaolin and manganese oxide deposits at the toe of the Shek Kip Mei landslide (FMSW, 2000)

surfaces and groundwater flow paths/responses. Although the true significance of the 10 mm thick, laterally persistent discontinuity near the toe of the slope might have been difficult to ascertain from a ground investigation involving only a few drillholes, awareness of previous relict instability and detailed mapping of the slope face during excavation would have increased the probability of recognizing the importance of this feature.

Although the examples illustrated above are from coastal areas where undercutting and over-steepening of the hillsides by the sea has probably taken place, relict instability in upland areas is also common. Other examples of relict instability include: Ville de Cascade (HAPL, 1998a), Fei Ngo Shan (FSWJV, 2001c), Sham Wat (FMSW, 2001b), Queen's Hill (FSW, 2001d; Hughes *et al.*, 2002) and Lin Ma Hang Road (see Section 6.4.4).

6.4.3 Previous Failures and Deterioration of Man-made Slopes

General

Previous failures in man-made slopes, or evidence of deterioration or deformation may indicate adverse geological or hydrogeological conditions and should be a focus during any stability appraisal or investigation. Examination of previous landslide incident records and construction records, detailed API of all available photographs and field inspections, particularly in the area above the slope crest, facilitate the recognition of potential problem sites. They also help to determine likely causes so that further engineering geological studies and ground investigations can be effectively focused. Typical indicators of deformation include cracked drains and deformed stairs, e.g. FMSW (2001a); FMSW (2001d).

1988 Landslides at Island Road School

These landslides were described in detail by Irfan (1989, 1994b). The main landslide (Figure 6.4.8) involved about 800 m^3 of material. There had been much previous instability (Figure 6.4.9), with many landslips associated with an old borrow area dating back to at least 1924. Significant regression of the backslope of the borrow area had also occurred between 1924 and 1949.

Subsequent investigations suggested that the main landslide was probably caused by undercutting of a kaolin-rich, creeping layer of tuff resting on a pre-existing shear plane (Figure 6.4.10), combined with infiltration of water on the unprotected slope which had been stripped bare of vegetation during construction. The weak tuff and pre-existing shear

Figure 6.4.8 – 1988 landslide at Island Road School (Irfan, 1989)

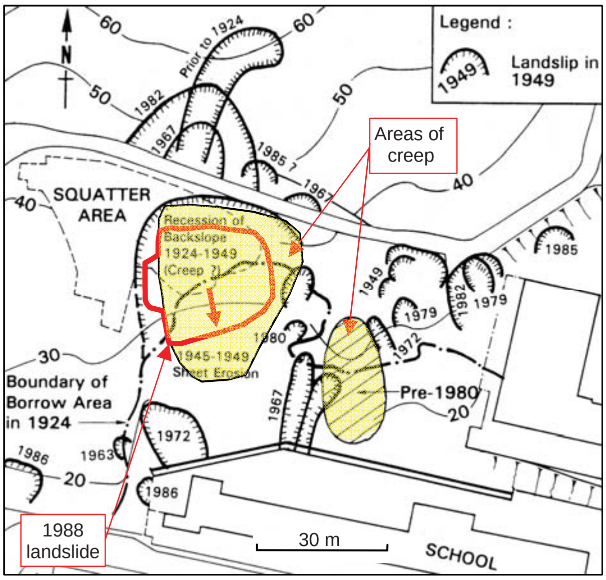


Figure 6.4.9 – Previous instabilities near site of the 1988 landslide at Island Road School (after Irfan, 1994b)

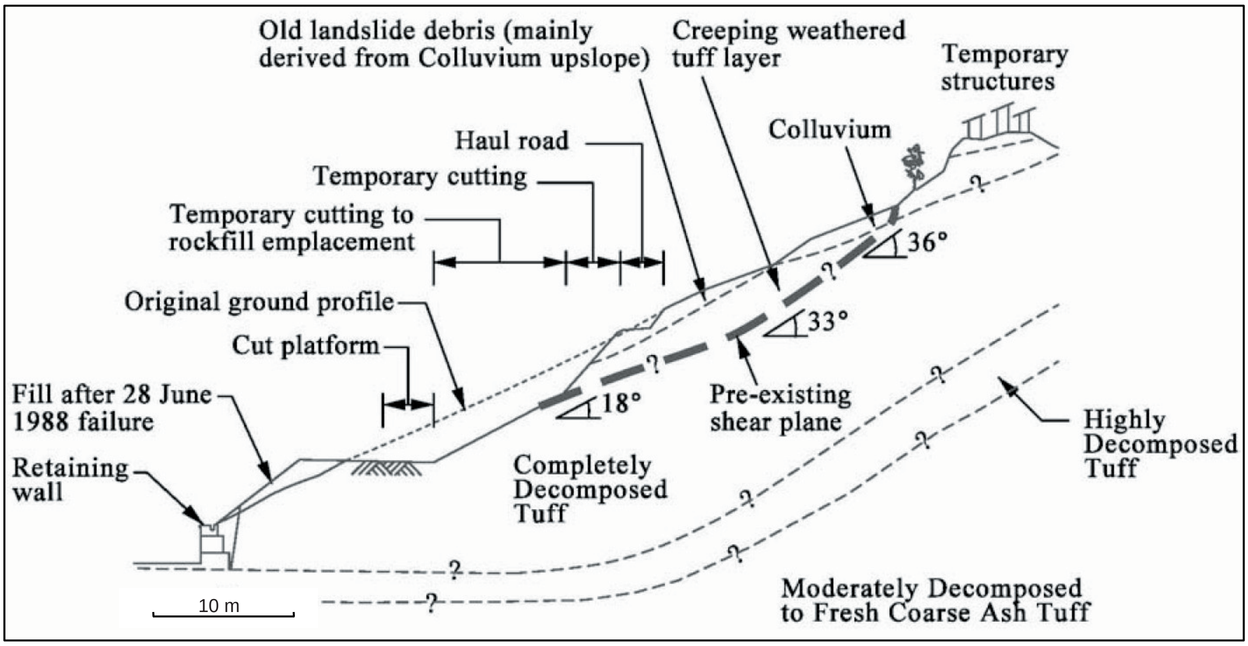


Figure 6.4.10 – Section through the 1988 landslide area at Island Road School (Irfan, 1989)

plane had not been recognised during the original ground investigation.

1995-1998 Landslides at the Junction of Sai Sha and Tai Mong Tsai Roads

The locations of the landslides are shown in Figure 6.4.11. The 1987 landslide was primarily controlled by slickensided relict joints with probable kaolin and manganese-oxide in fills (Figure 6.4.12; Premchitt, 1991). Probable relict joints can be seen in Figure 6.4.13 which shows the 2,000 m^3 landslide in 1995.

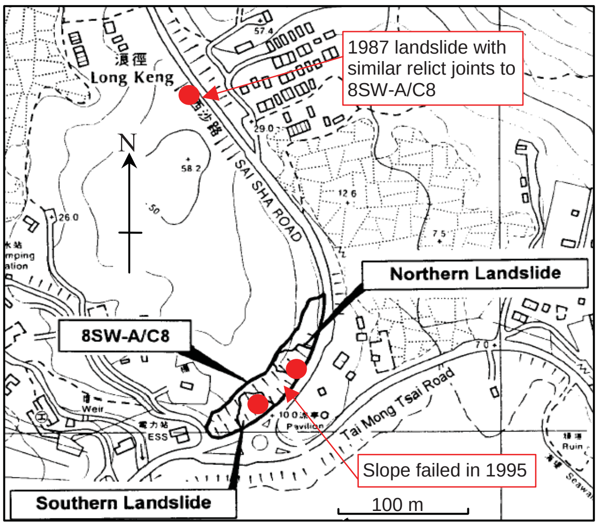


Figure 6.4.11 – Location of previous failures near the 1998 northern and southern landslides which occurred during LPM works (after Premchitt, 1991; Wong, 1997; FSW, 2001a)

During subsequent LPM works, a steep, temporary soil nailed slope was formed to accommodate the proposed permanent works which consisted of a reinforced concrete retaining wall. In 1998 during excavation, the slope failed in two places, involving about 2,600 m^3 of material (FSW, 2001a).

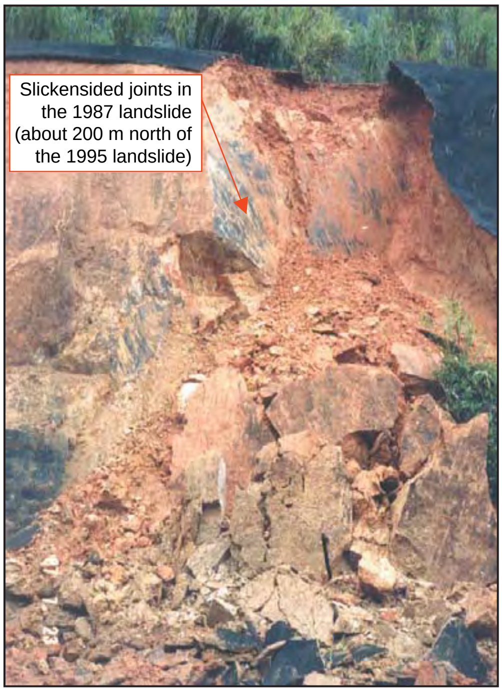


Figure 6.4.12 – 1987 landslide at Sai Sha Road with kaolin and manganese oxide coated relict discontinuities (Premchitt, 1991)

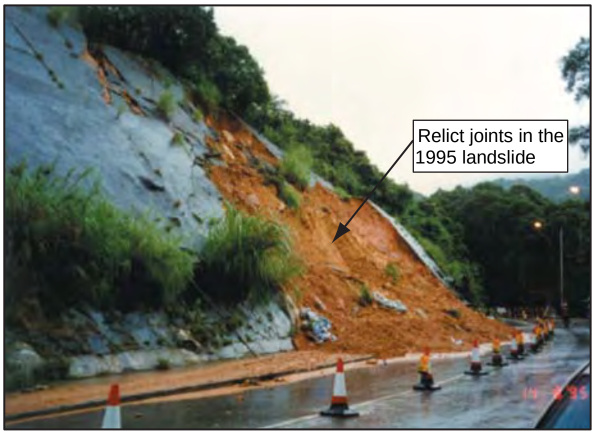


Figure 6.4.13 – 1995 landslide at Sai Sha Road with relict joints (Wong, 1997)

Figure 6.4.14 – Slickensided, manganese oxide coated, sub-vertical relict joints forming the back-scarp to the 1998 landslide which occurred during LPM works to the 1995 landslide at the junction of Sai Sha and Tai Mong Tsai roads (FSW, 2001a)

Contributory factors included:

- the presence of persistent, slickensided, kaolin infilled and manganese-oxide stained relict joints (Figure 6.4.14),
- a probable rise in groundwater of about 10 m during heavy rain which was much higher than had been anticipated (Figure 6.4.15), and
- incomplete temporary soil nailing works.

This example indicates the value of reviewing the failure history of an area from an engineering geological perspective. The presence of slickensided, kaolin infilled and manganese-oxide stained relict joints may be particularly problematic in the general area. The hillside above Sai Sha Road is a low knoll, surrounded on three sides by low-lying ground (Figure 6.4.11), which would not be expected to be conducive to large rises in base groundwater levels.

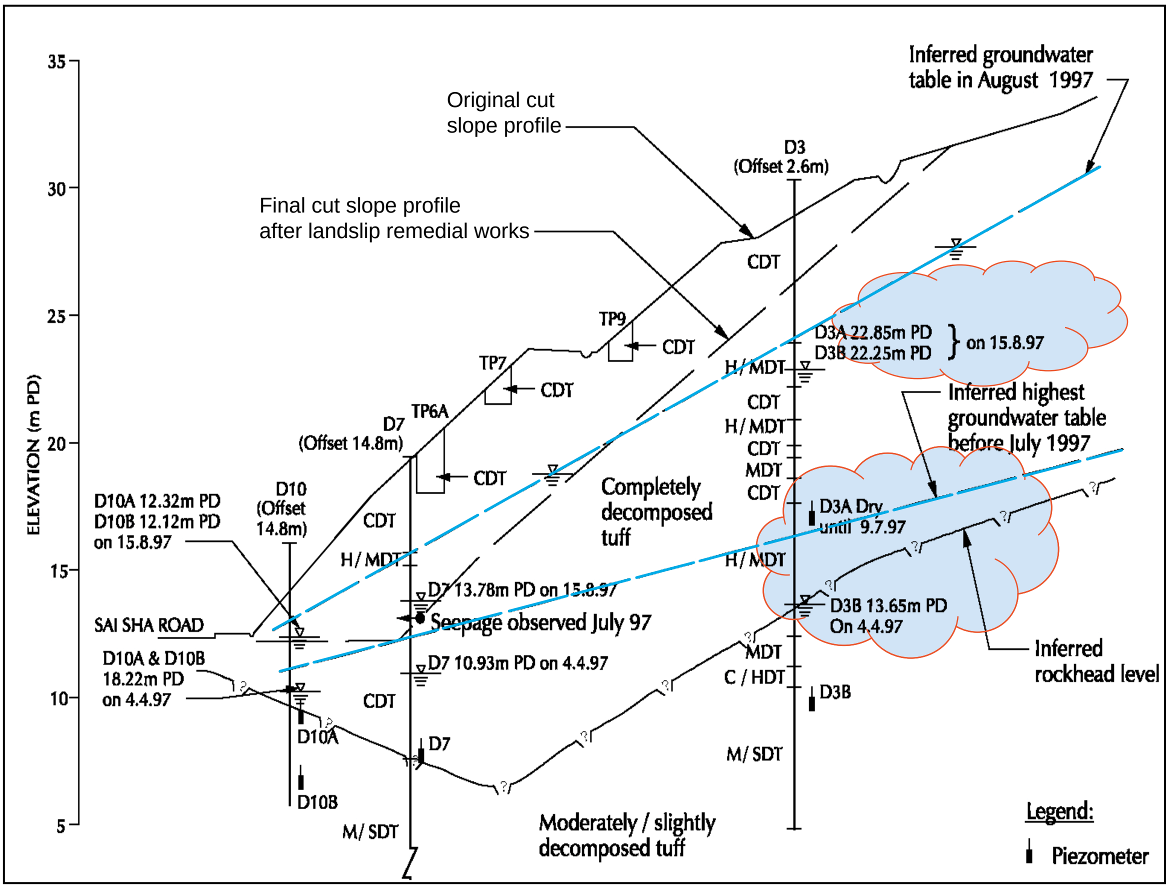


Figure 6.4.15 – Section through the 1998 landslide highlighting the large rise in groundwater level recorded by Halcrow buckets between July 1997 and August 1997 (FSW, 2001a)

Although detailed hydrogeological studies would be required to conclusively determine the cause of the large rise of groundwater level, surface water infiltration into steeply-dipping relict joints which may have been dilated due to stress relief caused by the original steep cutting might have been a contributory factor.

Other Examples Involving Previous Failures or Evidence of Deterioration

- The 1954-1997 Ching Cheung Road landslides involving repeated failures of over 10,000 m^3 of material (see Section 6.4.7).
- The 1978-1997 Lai Ping Road landslides involving retrogressive displacement of about 100,000 m^3 of material (see Section 6.4.7).
- The 1982-1999 Tsing Yi (1) landslides on Tsing Yi Road involving displacement of over 13,000 m^3 of material (see Section 6.4.6).
- The 1986-2000 Hiu Ming Street rockfalls involving a total of about 230 m^3 in at least four separate events. Contributory factors included progressive opening and soil infilling of sub-vertical joints leading to deterioration, tree root wedging and the presence of wavy sheeting joints that gave rise to local steep inclinations which were not recognised during previous assessments (HCL, 2002).
- The 1989-2002 Cha Kwo Ling rockfalls involving a total of about 1,020 m^3 in at least five separate events, with a single failure in 1995 accounting for 1,000 m^3 of the total volume. Contributory factors included the progressive dilation of sub-vertical release joints and the presence of undulating and persistent sheeting joints (MGSL, 2005).
- The 1995 Fei Tsui Road landslide which had a history of small failures above the extensive kaolin-rich seam along which the 14,000 m^3 landslide occurred in 1995 (see Section 6.4.4).
- The 1999 Route Twisk landslide involving failure of about 1,000 m^3 of material, with old tension cracks near the cut slope crest and cracked drains, indicating previous deterioration (FMSW, 2001a).

6.4.4 Adverse Discontinuities

General

Martin (2003) reports that evidence from systematic landslide investigations, together with earlier case histories, shows that most of the sizeable (>50 m^3) landslides and all the large landslides (>500 m^3) in cut slopes have failure surfaces formed wholly or partly along discontinuities in saprolite or less weathered rock. Assessment of the geological structure is therefore important in the investigation, design and construction of cut slopes.

1982-1985 'Pepco' Slope Distress, Nam Wan, South Tsing Yi

Hencher (1983e) investigated a zone of distress (Tsing Yi (2)) that occurred after a heavy rainstorm in 1982 on a 100 m high slope above Tsing Yi Road opposite the Pepco power station (now demolished). Figure 6.4.16 shows the configuration of the slope in 2003 and the locations of previous instabilities. In 1982, blocks of completely and highly decomposed granite were displaced along a set of joints infilled with silty sand dipping at about 20-25° out of the slope. A set of sub-vertical, slickensided, manganese-oxide and kaolin infilled joints formed the back-scarp of the failure and also divided the displaced mass into a series of slices with sub-vertical relative displacements (see Figure 6.4.17). Another set of joints formed side-release surfaces orthogonal to the set forming the back-scarp. The volume of the displaced mass was about 3,200 m^3.

The relict structure was similar to that found in the underlying moderately to slightly decomposed rock slopes which had been previously mapped. Further deterioration of the area occurred during heavy rain

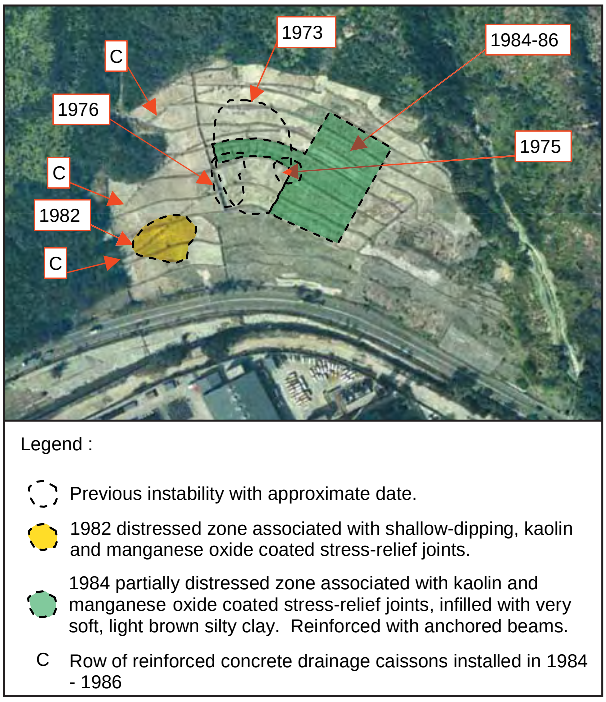


Figure 6.4.16 – 2003 Orthophoto of the 'Pepco' slope near Nam Wan, South Tsing Yi showing previous distressed zones

Figure 6.4.17 – Distressed 'Pepco' slope in 1982

in 1983, leading to substantial disintegration and almost complete detachment of the distressed zone as shown in Figure 6.4.18.

Subsequent remedial works included extensive re-grading of the slope and the installation of three rows of inter-linked drainage caissons (Figure 6.4.16). During excavation of the caissons in 1984, it was noted that in some areas, many low angle joints and subvertical joints close to rockhead were slickensided and coated with manganese-oxide deposits, with very soft light brown clay infills up to 200 mm thick.

Figure 6.4.18 – Outwash from complete failure of the 1982 distressed zone of the 'Pepco' slope blocking Tsing Yi Road in 1983 following heavy rain (Choot, 1993b)

These were very similar in appearance to that shown in Figure 4.4.15. Shortly after the clay infilled joints had been identified, movements of large blocks of completely and highly decomposed granite occurred during heavy rain (Buttling, 1986). The volume of the displaced masses was about 800 m^3. Although there were some variations in the angle of dip of the low angle joints across the site, the orientations of the relict joints in the displaced masses were similar to those in the 1982 zone of distress.

A re-examination of unopened mazier samples and rock cores from previous investigations indicated that even where thick clay infills had been proven by slope and caisson mapping, only zones of core loss with occasional traces of clay on joints testified to their presence. Further drillholes using continuous triple tube sampling with air-foam flushing were carried out, but identification of the clay infills still proved to be problematic due to contrasts in drilling resistance that led to erosion of the very soft silty clay.

Following detailed mapping using survey markers as reference points and enlarged aerial photographs as base plans, synthesis of the available data was carried out to create a 3-D geological model using isometric drawings and closely-spaced geological sections. These enabled sensitive zones to be identified that were subsequently reinforced with a grillage of anchored beams (Buttling, 1986).

This example highlights the following key engineering geological issues with regard to saprolite:

- Saprolite retains the structure of the parent rock mass, but low-friction weathering products are usually more prevalent in the discontinuities, particularly near rockhead. Slickensiding may also be common.
- Zones of clay infill are not easy to detect by drilling, particularly where there is a high contrast in drilling resistance between the clay and the intervening blocks.
- Although a saprolite mass may be classified as a soil in engineering terms, it is prudent to treat it as a weathered rock mass, with stability being at least partly controlled by relict discontinuities. The low strength of the intact blocks is an additional disadvantage in that failure of intact material can occur much more readily than in the case of strong rock.
- Where the relict structure defines outwardly

dipping, tabular blocks or wedges, stability may be much lower than might be predicted from assuming a homogeneous continuum where the mass shear strength has been based on the results of triaxial tests on 'intact' samples.
- Well-developed sub-vertical joints form release surfaces that allow surface water infiltration where they are opened by stress relief, leading to potentially high cleft-water pressures and potentially high pressures in basal discontinuities where they are interlinked.

Engineering geological inputs which can help to reduce geotechnical uncertainty in large saprolite or mixed rock and soil cuttings include:
- examination of the engineering geological characteristics of previous, nearby failures,
- mapping of the geological structure revealed in the parent rock mass for assessment of potential failure modes in the saprolite, and
- detailed engineering geological mapping of temporary exposures and the cut face during construction in order to increase the probability of the recognition of adverse structures.

1982 South Bay Close Landslide

This landslide, described by Hencher (1983b) and Clover (1986), occurred in a steep volcanic rock cutting during heavy rain. About 3,800 m^3 of moderately and highly decomposed rock failed on a very persistent, undulating discontinuity which dipped out of the slope face at an average angle of about 28° (Figure 6.4.19). A further 700 m^3 of adjacent material failed on the same discontinuity during another rainstorm in the same year.

The infill to the discontinuity was up to 700 mm thick and consisted of moderately decomposed gravel fragments in a matrix of sand and silt with pockets of light brown and pink clay (Figure 6.4.20).

The infill was highly variable and was thought to be tectonically disturbed, although no associated slickensides are noted in the above-mentioned references. Hencher (1983b) notes that a continuous broken zone of weathered material was also found in better quality rock at a deeper level. Figure 5 of Hencher (1983b) shows thick tree roots penetrating a loosely interlocked rock mass which may indicate previous movement.

Figure 6.4.19 – 1982 South Bay Close landslide scar (Hencher, 1983b)

700 mm thick infill along rupture surface overlain by highly to moderately decomposed tuff with soil pipes that flowed with water for several days after the failure (indicated by arrows)

Completely to highly decpomposed tuff fragments in a silt/clay matrix overlain by very closely jointed moderately decomposed tuff

Local area of tuff fragments in a clay matrix

Figure 6.4.20 – Variable appearance of the infill along the rupture surface of the South Bay Close landslide (Hencher, 1983b)

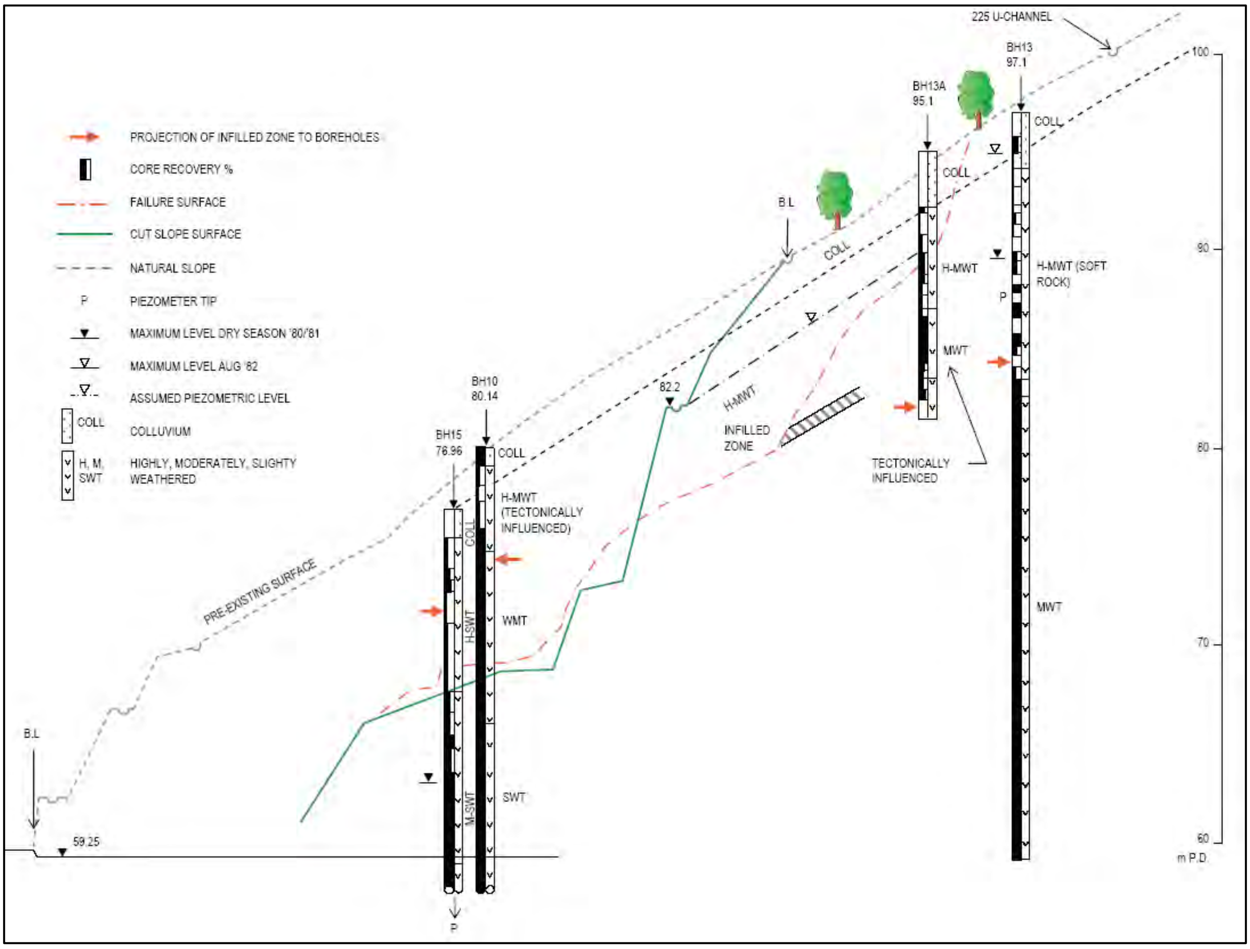


Figure 6.4.21 – Section through the 1982 South Bay Close landslide showing projection of the infilled zone through boreholes (Hencher, 1983b)

A section through the landslide area is shown in Figure 6.4.21, with the infilled zone projected through zones of poor recovery in drillholes and/or zones described in the drillhole logs as being 'tectonically influenced'. However, the generally poor core recovery made it difficult to infer the presence of a single zone of weakness. Clover (1986) reports that the zone of weakness was recognised during excavation, but the slope failed during the design of the stabilisation measures.

After the failure, water was seen issuing from soil pipes above the infill (Figure 6.4.20) and flowed for several days. 'Halcrow buckets' were subsequently installed in BH13 (Figure 6.4.21) which recorded a much higher groundwater level during the August 1982 rainstorm than had previously been assumed from manually dipping the piezometer in the preceding dry season. It is likely that a high perched water table developed above the clay-rich, infilled zone during heavy rainstorms.

This example illustrates the following key engineering geological issues and considerations that may help to reduce geotechnical uncertainty:

- Extensive, adversely orientated, clay-rich weakness zones can be encountered in rock masses, overlain by substantial thicknesses of moderately and highly decomposed rock.
- Although inferred to be a low-angle thrust fault, the critical zone of weakness has many similarities to more recently encountered weak zones at other sites with kaolin-rich accumulations.
- High quality coring in rock would increase the probability that similar zones would be recognised at the ground investigation stage.
- If core recovery is consistently poor at a certain level or along a certain plane, consideration may be given to carrying out additional drillholes inclined into the hillside or excavating trenches where the inferred zone is expected to daylight at the ground surface. Consideration could also be given to carrying out down-hole geophysical logging as outlined in GEO (2004h).
- Consistently poor core recovery or anomalies in water returns during drilling through substantial

thicknesses of moderately or highly weathered rock masses on sloping ground may indicate dilation due to previous movements. Permeability testing may help to confirm this.

- Where an adversely-orientated weakness plane strikes sub-parallel to a proposed cut face, it will form a sub-horizontal trace along the cutting when exposed. If the excavation is carried out from the top down, with no deeper excavations or exposures at the sides, the feature may not be detected during face mapping until it is too late to provide effective stabilisation measures.
- High perched water levels may develop in fractured ground underlain by relatively impermeable clay seams. Piezometers should be located near the base of such zones and equipped with automatic recording devices or 'Halcrow buckets' in order to record peak groundwater levels. These should preferably be monitored for a complete wet season before the final design is confirmed.

1995 Fei Tsui Road Landslide

About 14,000 m^3 of highly and completely decomposed tuff failed along a laterally persistent, kaolin-rich seam dipping between 10° and 20° out of the slope (GEO, 1996a,b). Extensive, steeply-dipping, kaolin infilled relict joints formed the back-scarp to the landslide (Figures 6.4.22 and 6.4.23). Details of the kaolin-rich altered tuff layer and the slickensided relict joints in the back-scarp are shown in Figure 6.4.24.

Shear box testing on the kaolin rich tuff layer yielded average and lower-bound shear strength parameters of $c' = 0$, $\phi' = 29°$ and $c' = 0$, $\phi' = 22°$ respectively. Back analysis of the landslide indicated that a perched water table only 2 m above the rupture surface yielded a factor of safety of 1.0 if the operative angle of friction on the kaolin seam was assumed to be 28°.

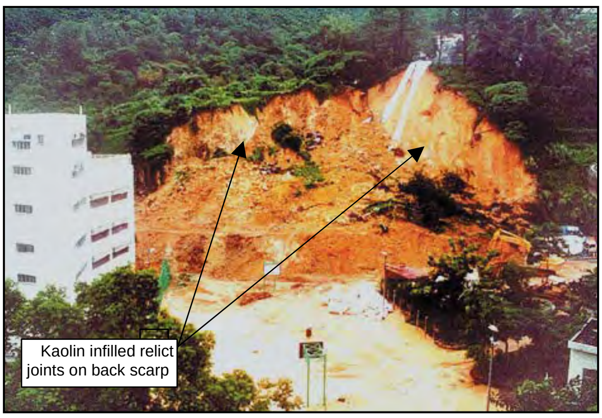


Figure 6.4.22 – 1995 Fei Tsui Road landslide

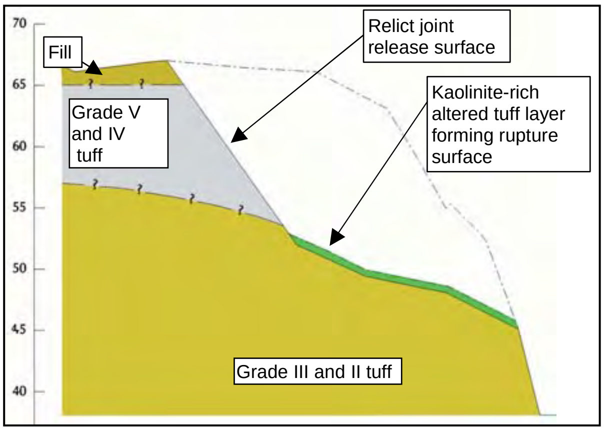


Figure 6.4.23 – Section through Fei Tsui Road landslide (GEO, 1996b)

Figure 6.4.24 – Appearance of the kaolin-rich altered tuff in the rupture surface of the Fei Tsui Road landslide (GEO, 1996b)

A photograph of the slope taken in 1977 (Figure 6.4.25) clearly indicates the exposed and unprotected nature of the kaolin-rich seam, but in several studies undertaken before the failure, its true implications for slope stability had not been recognised (GEO, 1996a,b).

Key engineering geological issues arising from studies of this failure include:

- Previous small-scale failures had occurred above

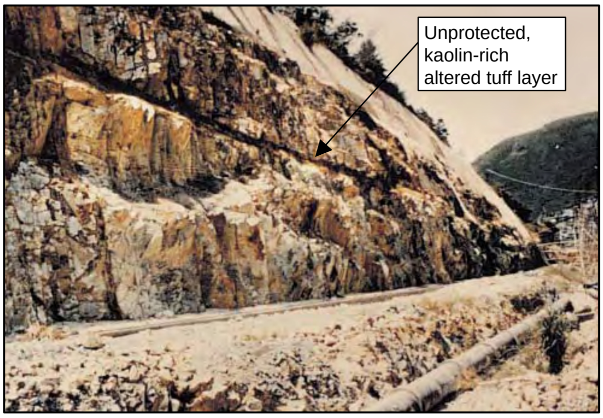


Figure 6.4.25 – Location of the 1995 Fei Tsui Road landslide in 1977 (B&P, 1977)

the seam between 1985 and 1993. Such failures, or deterioration, may indicate signs of an impending larger instability.

- Kaolin-rich seams can have very low shear strength, and no conclusive stability assessment can be made unless appropriate stability analyses are conducted.
- Representative geological and ground models will help to define all potential failure modes and controlling factors including discontinuity control, shear through the overlying soil mass and the effects of groundwater.

1998 Tai Po Road Landslide Opposite Chak On Estate

This landslide involved the displacement of about 1,400 m^3 of material during its first wet season, about six months after the slope had been constructed (FSW, 2001b). The failure was controlled at its base by a persistent, adversely orientated sandy silty clay layer dipping out of the slope at about 14 ° (Figure 6.4.26).

Sub-vertical, manganese-oxide coated, persistent discontinuities formed lateral release surfaces sub-parallel to photolineaments which could be seen on the original cut slope in the 1949 aerial photographs. The western release plane and the sandy silty clay layer that were exposed in trial pits (Figure 6.4.27) during the landslide investigation were not considered in the original design and were not identified during construction when a large quantity of material was excavated to cut back the slope.

Although it may have been difficult even with continuous mazier sampling to identify the persistent sandy silty clay layer, partially underlain by slightly decomposed coreslabs, this example indicates the importance of:

- development of the geological model, starting with a thorough API,
- targeting the ground investigation to identify adverse geological structures, and
- mapping the excavation during construction to further refine the ground and design models.

1981-2000 Lin Ma Hang Road Landslides

The instabilities in the cut slope at Lin Ma Hang Road, Lok Ma Chau illustrate some of the problems of slope engineering in tropically weathered, intensely folded

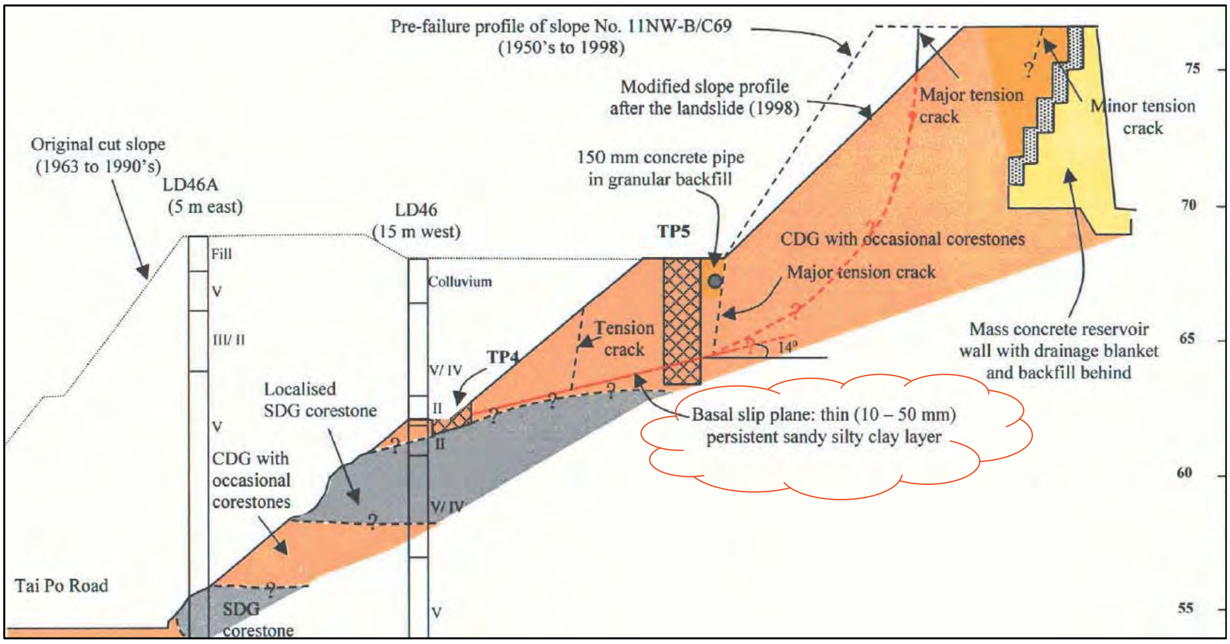


Figure 6.4.26 – Section through 1998 Tai Po Road landslide opposite Chak On Estate (FSW, 2001b)

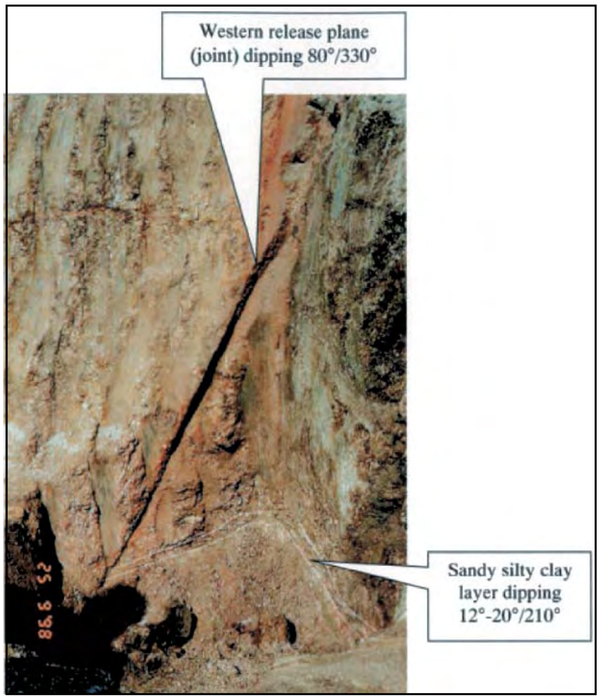


Figure 6.4.27 – Release plane and sandy silty clay layer exposed in trial pit during investigation of the 1998 Tai Po Road landslide (FSW, 2001b)

meta-sediments. Figure 6.4.28 shows a general view of the slope and instability history.

The slope was originally constructed in meta-sedimentary rocks of the Lok Ma Chau Formation (see Section 5.7) with 60° benches in early 1981, after an initial ground investigation which comprised two trial pits and two drillholes. The excavations revealed highly complex geology with recumbent folds plunging towards the slope. An undulating schistosity was also present which dipped towards the slope at an average angle of about 45°.

After several localised failures, the benches were re-graded to 45° in mid-1981, but some failures still occurred, defined by wedge-shaped joint intersections and undulations in the foliation planes (Figure 6.4.29). An extensive tension crack also developed in the middle of the slope below a topographical depression which is considered to be the site of a relict landslide (see Figure 6.4.28). Following further stability appraisals, the benches were re-graded to 35° and the slope hydroseeded, but some small failures still occurred in 1982 (Figure 6.4.28).

The landslide in 2000 involved the displacement of about 2,100 m^3 of material which moved about 1.5 m outwards from the slope toe. Investigations revealed extensive tension cracks and cavities of various ages, and a relict rupture surface near the slope crest with associated clayey infills (Figure 6.4.30). The metamorphic rocks were also found to be

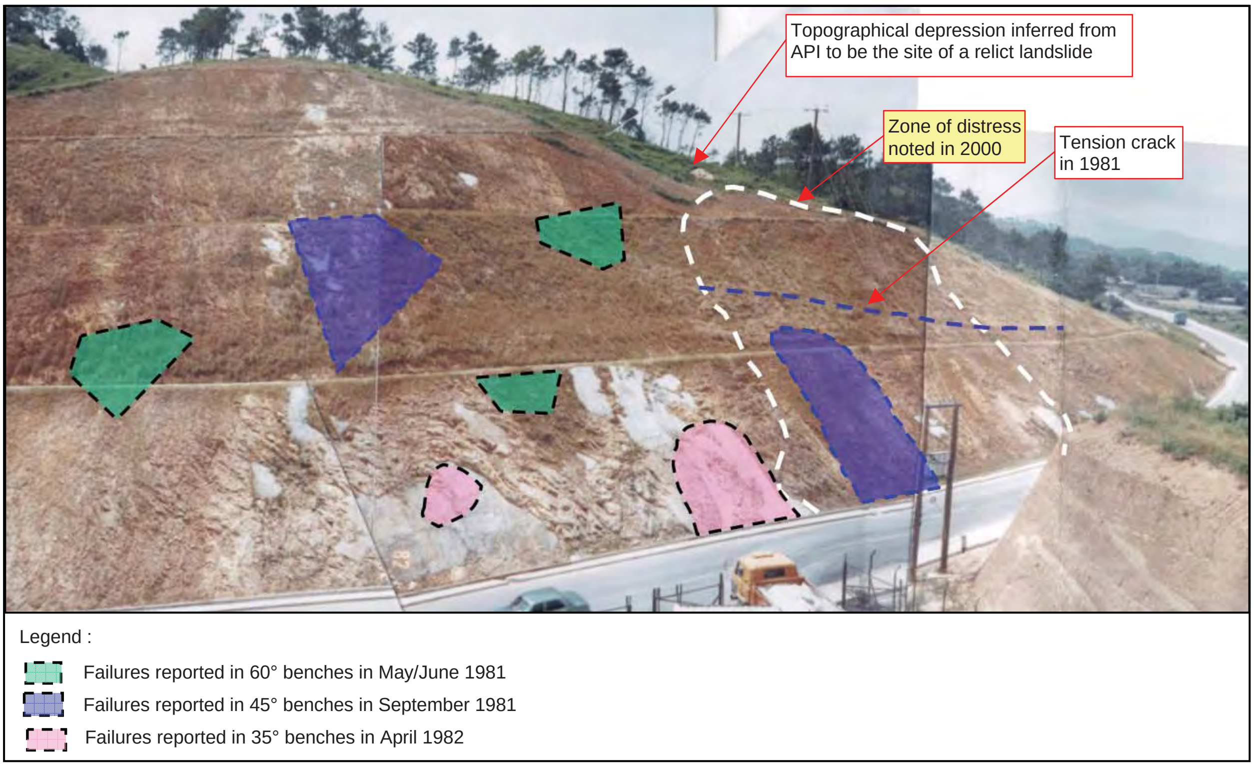


Figure 6.4.28 – 1982 photograph showing history of cut slope instability at Lin Ma Hang Road

Figure 6.4.29 – 1981 failure on undulating foliation planes plunging steeply towards cutting at Lin Ma Hang Road

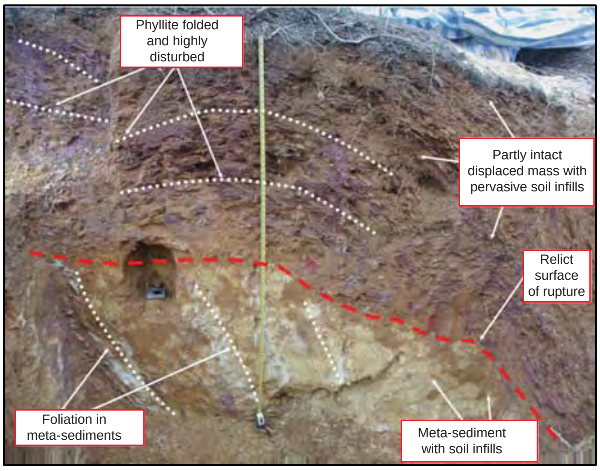


Figure 6.4.30 – Relict rupture surface exposed in trial pit in 2001. Overlying phyllite is highly disturbed and clayey sediment infill in underlying meta-sediments is concentrated along the foliation fabric and near the relict surface of rupture

more intensively weathered and weaker than had been described in 1981, and adversely orientated, undulating and dilated, foliation fabrics were seen to control the basal shear surface in the slope. The landslide was probably the result of reactivation and enlargement of the relict landslide, exacerbated by progressive dilation of the very weak, intensely foliated rock mass due to excavation of the cutting, tension crack development and ingress of surface water.

In rock slope engineering, it is common practice to use a variant of the 'Observational Method', where the structure is mapped in detail as excavation proceeds, and stabilisation works are installed as necessary. This is usually successful, provided that large scale, adverse geological structures and potential large-scale failure mechanisms have been recognised and incorporated into the ground and design models during the investigation and design stages, or at least at a very early stage during excavation when more detail first becomes apparent. Recognition of such structures is equally important for engineering in saprolites, but can be more difficult due to less obvious contrasts between the adverse structures and surrounding weak material. Skilled engineering geological input is especially valuable in such cases.

Figure 6.4.31 shows an extract from the original geological face map of the cutting which illustrates the complex structure and weathering. Although this gives a record of the basic geological materials exposed, localised failures still occurred due to lack of timely remediation, and there is no indication that the pre-existing shear planes associated with the large relict landslide had been identified. This underlines the need to regularly review the resources necessary for verification of the ground and design models during excavation to ensure that they are commensurate with the geological and hydrogeological complexity of the site.

Other Examples Involving Significant Failures Controlled by Adverse Discontinuities

- The pre-1924 to 2001 Queen's Hill progressive landslides involving displacement of about 2,000 m^3 of predominantly colluvium along a basal interface with completely to highly decomposed meta-tuff (FSW, 2001d; Hughes *et al.*, 2002).
- The 1963-1985 Tin Wan Hill slope movements involving the complex displacement of about 19,000 m^3 of colluvium and weathered tuff controlled by a stepped network of kaolin infilled joints (Section 6.4.6).
- The 1971 Fat Kwong Street landslide involving failure of about 50,000 m^3 of material partially controlled by relatively shallow-dipping relict joints in granite (O'Rorke, 1972).
- The 1972 Po Shan Road landslide (Figure 6.4.1) involving about 40,000 m^3 of weathered tuff and colluvium may have been at least partially controlled by a large relict joint noted by B&P (1981) and Cooper (1992). However, the lack of a detailed engineering geological assessment of this feature in 1972 precludes any definite conclusion to be drawn about its influence.

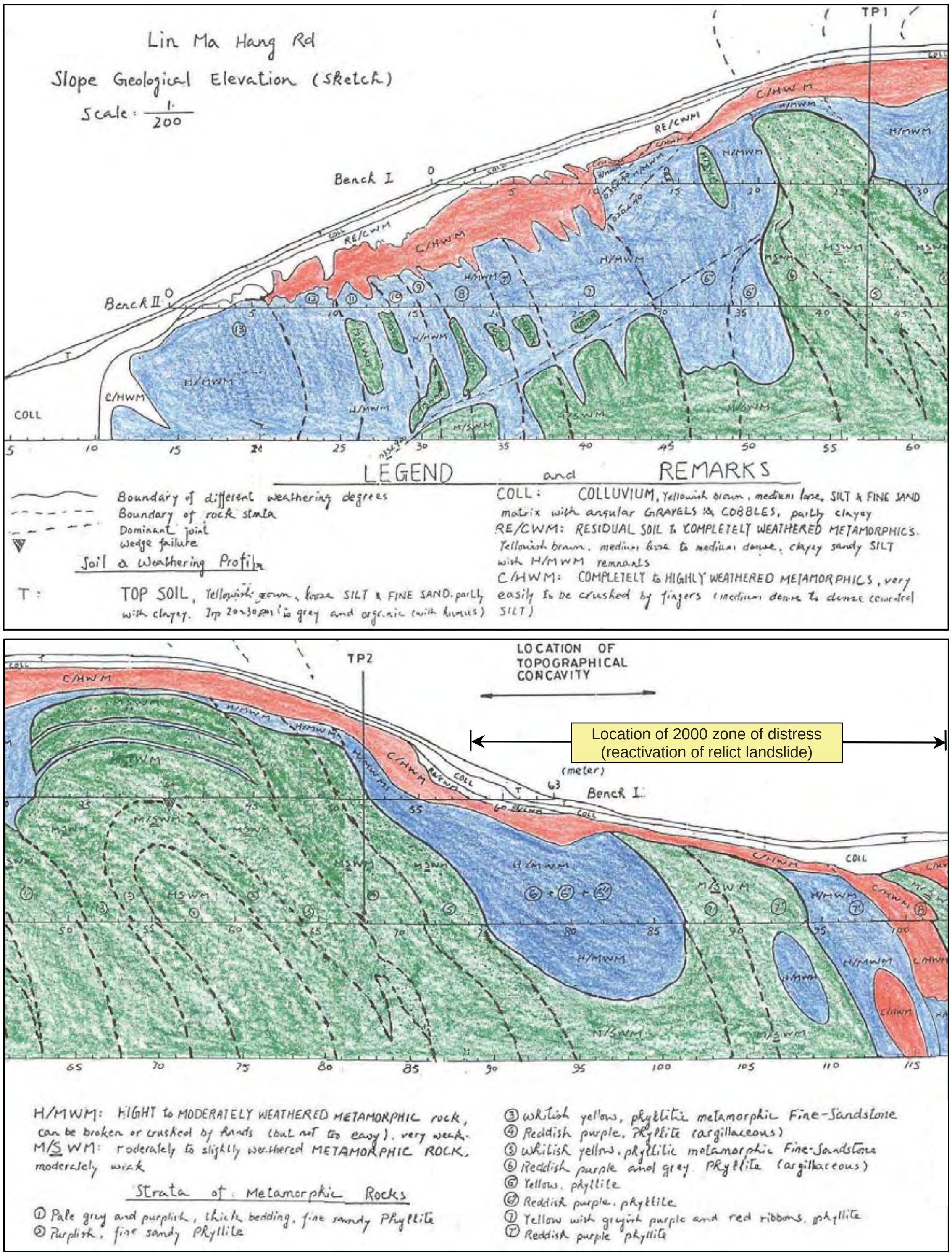


Figure 6.4.31 – Extract from the original 1981 geological face mapping record of the Lin Ma Hang Road cutting showing complex structure and weathering

- The 1982 Junk Bay Road landslide involving failure of about 1,300 m ^{3} of granitic saprolite on a steeply-dipping, kaolin in filled joint (Hencher, 1983a).
- The 1986-87 Wu Kau Tang landslide involving displacement of about 800 m ^{3} of predominantly colluvium along a clay-rich basal interface with underlying sedimentary rocks of the Port Island Formation (see Section 5.6.3).
- The 1987 Cho Yiu Estate landslide involving failure of about 1,200 m^3 of weathered granodiorite, largely controlled by kaolin in filled relict joints

(Siu & Premchitt, 1988; Massey & Pang, 1988; Siu & Premchitt, 1990).

- The 1987-1998 landslides at Sai Sha Road involving volumes ranging up to 2,600 m^3 which were partially controlled by slickensided, manganese oxide and kaolin infilled relict joints in tuff (see Section 6.4.3).
- The 1988, 800 m^3 landslide at Island Road School in colluvium and weathered tuff (see Section 6.4.3).
- The 1993 Allway Gardens landslide involving failure of about 250 m^3 along relict joints in volcanic saprolite (Chan *et al.*, 1996a).
- The 1995 Cha Kwo Ling rockslide involving about 1,000 m^3 of granite on undulating sheeting joints (MGSL, 2005).
- The 1997 Ching Cheung Road landslside involving failure of about 3,200 m^3 of weathered granite controlled by kaolin in filled relict joints (see Section 6.4.7).
- The 1997 Kau Wah Keng landslide involving failure of about 360 m^3 of weathered granite, largely controlled by a kaolin infilled discontinuity (HAPL, 1998c).
- The 1997 Ten Thousand Buddhas landslide involving failure of about 1,500 m^3 of weathered granite largely controlled by kaolin in filled joints (HAPL, 1998d).
- The 1999 Sham Wat Road landslide involving displacement of about 1,700 m^3 of colluvium with a network of soil pipes along a basal interface with completely decomposed volcanic rocks (FMSW, 2001b).
- The 1999 Shek Kip Mei landslide involving displacement of about 6,000 m^3 of weathered granite partially controlled by a manganese oxide and kaolin in filled basal discontinuity (see Section 6.4.2).

1995-1998 Southwest Slope of the Ting Kau Cutting, Route 3

This case provides an example of the use of an observational method during the design and construction of a major rock cutting where potentially adverse geological structures needed to be incorporated into the ground and design models. These models were then regularly updated as new information from systematic face mapping was obtained.

Geological data obtained prior to construction indicated the possibility of a relatively shallow dipping joint set affecting the southwest side of the cutting. However, some orientated core indicated favourable conditions, which was later found to be due to the core from one drillhole being wrongly orientated around its axis by 180° (Figure 3.4.3).

At an early stage during construction, an excavation was made at the northwest end of the cutting which helped to clarify the geological structure. This revealed a continuous shear plane that would intersect the southwest face at an acute angle, and persistent, shallow-dipping joints (Set 1) that were potentially adverse for stability (Figure 6.4.32). Consequently, the southwest slope was re-designed with 20 m high benches at 48° with nominal 4 m wide berms, forming an overall slope angle of 42° (Figure 6.4.33).

Careful mapping of the discontinuities was carried out, with detailed descriptions of the characteristics recorded. This information was plotted onto large-

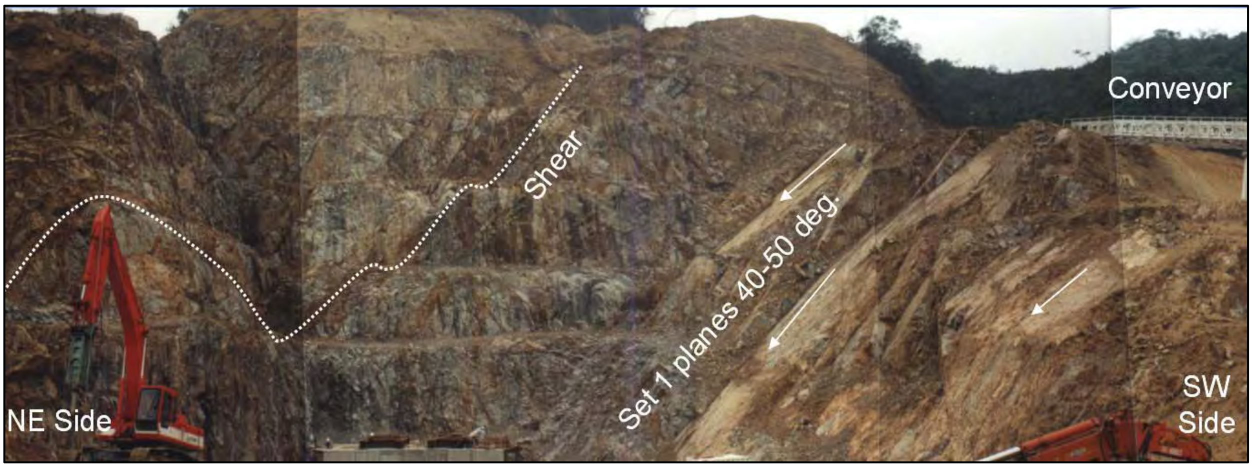


Figure 6.4.32 – Initial 'box-cut' of Ting Kau cutting showing continuous shear and persistent Set 1 joint planes (photograph by J.W. Tattersall)

Figure 6.4.33 – Route 3 Ting Kau cutting showing moderately-inclined southwest face

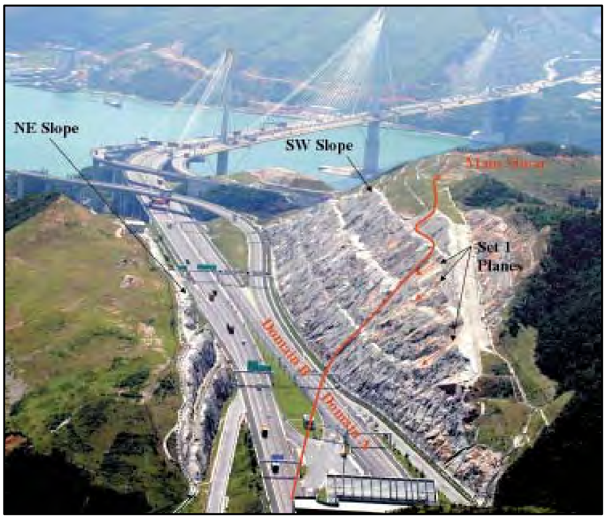


Figure 6.4.35 – View of completed cutting showing trace of main shear and Domains A and B

scale plans and sections to maintain a 3-D overview of the developing geological model. Two structural domains were identified (Figures 6.4.34 and 6.4.35), which were separated by the shear plane. Domain A contained a persistent set of joints dipping between 38° and 50 ° whereas Domain B contained more closely spaced but less persistent Set 1 joints rarely dipping flatter than 48°.

Of most concern was the possibility of failure of parts of Domain B closest to the ‘leading edge’of the shear (Figure 6.4.34) which might involve a combination of three failure mechanisms: a) movement along the shear plane, b) movement along an adversely dipping joint set, and c) failure through the rock mass near the toe. These concepts are discussed in Section 3.5 (Figures 3.5.3 and 3.5.4). The eventual design model used for sensitivity analysis is shown in Figure 6.4.36. The range of dip angle and maximum length of the shallow-dipping joint set were regularly reviewed, based on statistical analysis of the mapping data received from site.

Sensitivity analyses were carried out as part of the design modelling assuming dif ferent ‘intact’ rock compressive strengths to which the Hoek-Brown failure criterion is generally the most sensitive. The rock mass quality (Q’) assumed was very

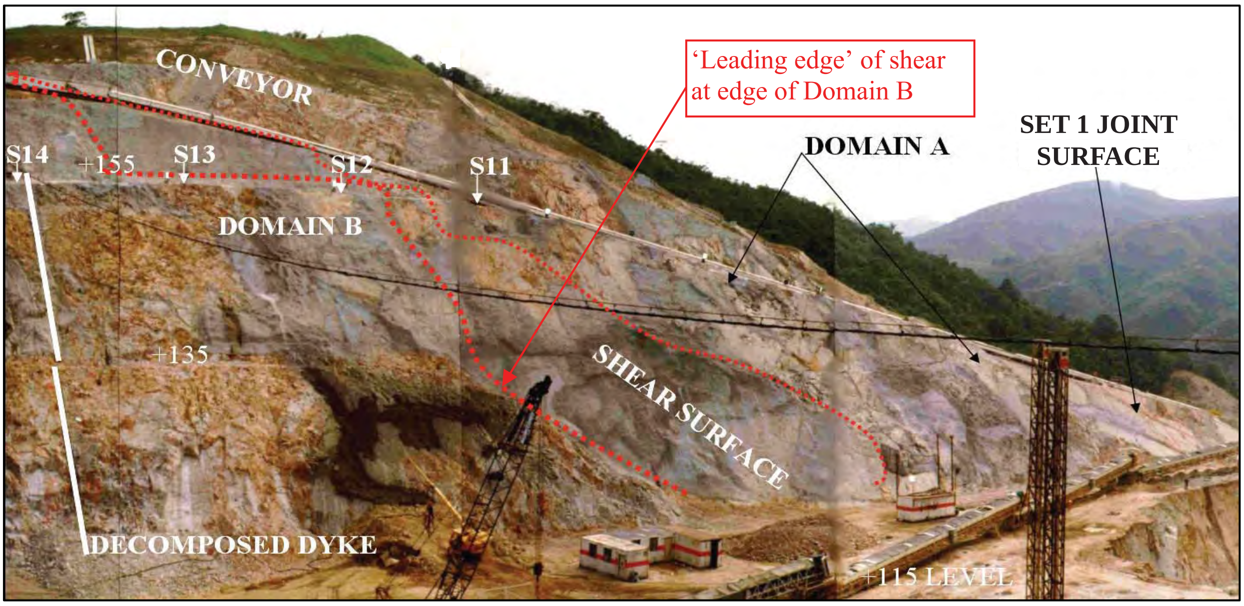


Figure 6.4.34 – Upper part of face during construction showing shear surface and Domains A and B (photograph by J.W. Tattersall)

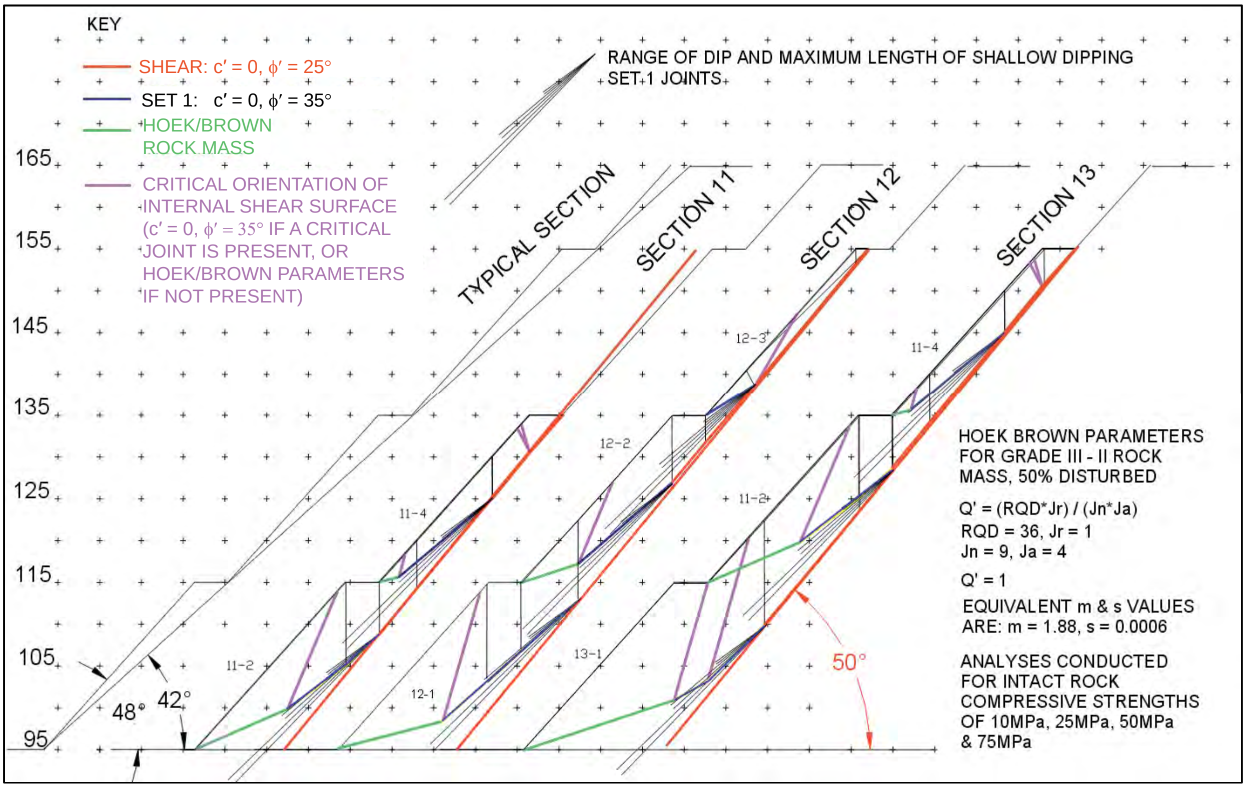


Figure 6.4.36 – Analytical models for sensitivity analysis assuming a composite failure surface along the main shear plane and Set 1 joints, passing through the rock mass at the toe (Tattersall, 2006)

conservative for the conditions revealed on site. Different cases were analysed and reinforcement charts were produced to allow assessment of the need to install large rock nails as a precaution against adverse conditions being encountered deeper in the excavation. Other potential effects on stability which could be assessed on site were also considered, such as:

- the presence or absence of critically orientated Set 1 joints,
- the presence or absence of other joints allowing internal shear,
- degree of 'keying-in' and proximity of major lateral release surfaces,
- lower shear strength of the shear plane or the set of joints,
- more adverse groundwater conditions,
- greater persistence of the critically orientated set of joints,
- flatter-dipping critical joints, and
- flattening of the main shear plane with depth.

Although initial assessments of stabilisation requirements were necessarily conservative, updating of the geological and ground models as excavation progressed indicated that the actual conditions were favourable for overall stability, and only minor stabilisation works were necessary on the lower parts of the slope. Piezometer monitoring for two wet seasons, seepage mapping and displacement monitoring of a large number of survey markers were also carried out during construction as a check on, and ultimate confirmation of, the effectiveness of the stabilisation strategy.

Similar concepts and strategies, with appropriate adjustments for material types and geological structure, could also be applied during the construction of major saprolite and mixed rock and soil cuttings where geological uncertainties exist.

6.4.5 Hydrogeological Boundaries

Most of the examples of failures due to adverse discontinuities cited in Section 6.4.4 involved the build-up of perched water at hydrogeological boundaries, promoted by direct infiltration during rainstorms, notably via sub-vertical discontinuities, and/or recharge from uphill through more permeable materials such as colluvium and near-surface dilated masses, and groundwater flow associated with open relict joints or soil pipes.

An additional example is the 1982 failure at Chainage 6750 on Tuen Mun Road (Hencher , 1983c; Hencher & Martin, 1984). A block diagram of the inferred hydrogeological model is shown in Figure 5.4.5 and illustrates the perching of groundwater above a shallow-dipping completely decomposed dyke which partially controlled the rupture surface.

Sub-vertical, completely decomposed dykes or clay infilled joints striking across a slope can also cause localized damming of the groundwater which can lead to failure. An example involving weathered dykes is the 300 m^3 failure at the 14 ½ Milestone on Castle Peak Road which occurred in 1994 (Franks, 1995; Chan *et al.*, 1996b).

The key to effective identification of zones susceptible to the build-up of perched water is the development of a representative geological and ground model, based on an understanding of the natural and anthropogenic processes that have formed the site, through geological mapping, seepage mapping and a focused ground investigation (see Sections 3 and 4.6). As noted in Section 6.4.4, piezometers in soil or dilated rock masses should be located a short distance above hydrogeological boundaries and equipped with automatic recording devices or 'Halcrow buckets' in order to record peak groundwater levels.

6.4.6 High Groundwater Levels in Deep Weathering Profiles

Some aspects of high groundwater levels in deep weathering profiles are discussed in Section 4.6, including the potential for sub-artesian or artesian conditions. Such occurrences may be associated with combinations of large upslope catchments, outcrops with high secondary permeability upslope (allowing rapid infiltration into the rock mass), low elevation (where upward flow is more likely), and possible depressions in rockhead which may act to concentrate groundwater and may be indicative of strong upwards flow, leading to more intense weathering over geological time. Examples include:

- The 1954-1997 Ching Cheung Road landslides involving delayed failure of over 10,000 m^3 of material (Section 6.4.7).
- The 1963-1985 Tin Wan Hill Road landslides involving progressive displacement of about 19,000 m^3 of material (Section 6.4.7).
- The 1975-1983 Pun Shan Tsuen landslides on Tuen Mun Road involving delayed progressive displacement of about 80,000 m^3 of material (Slinn & Greig, 1976; Choot, 1984; Jiao *et al.*, 1999).
- The 1976-2004 Tuen Mun Area 19 landslides involving the progressive displacement of over 200,000 m^3 of material (Section 6.3).
- The 1978-1997 Lai Ping Road landslide involving the progressive displacement of about 100,000 m^3 of material (Section 6.4.7).
- The 1982 Tsing Yi (1) landslide involving progressive displacement of about 12,500 m^3 of material (Hencher, 1983d; FMSW, 2001d). In this case, the original investigation drillholes had been terminated in very large corestones at a relatively high level above the soil/rock interface. This gave a false impression of the thickness of engineering soil, and the relatively high level of groundwater in the soil below the corestones was not detected.
- The 1989-1992 Siu Sai Wan landslides involving progressive displacement of about 4,000 m^3 of material (Ho & Evans, 1993; Sun & Tsui, 2003).
- The 1990-1995 Sham Shui Kok landslide involving progressive displacement of about 40,000 m^3 of material (Lau & Franks, 1998).

One of the key lessons learnt from studies of landslides is that the effect of regional hydrogeology on the base groundwater table should always be considered when planning investigations and interpreting the data. An example of large scale hydrogeological modelling of a delayed rise in groundwater level, where topographical and large scale geological influences were considered, is given in Jiao *et al.* (1999) for the 1983 Pun Shan Tsuen landslide.

6.4.7 Complex Hydrogeological Conditions

General

As noted in Section 4.6, local factors such as the presence of drainage lines, soil pipes, open discontinuities, and zones with large differences in hydraulic characteristics often give rise to complex hydrogeological regimes at the scale of most slopes and landslides, including many of the examples listed in Section 6.4.6. The potential hydrogeological complexity of a site needs to be anticipated through an understanding of the range of hydrogeological processes commonly encountered in Hong Kong (Section 4.6) in conjunction with the site-specific characteristics in order to effectively plan the ground investigation and interpret the results.

1963-1985 Tin Wan Hill Road Landslide

The complex, progressive displacements at Tin Wan Hill Road (Figure 6.4.37) were described by Irfan

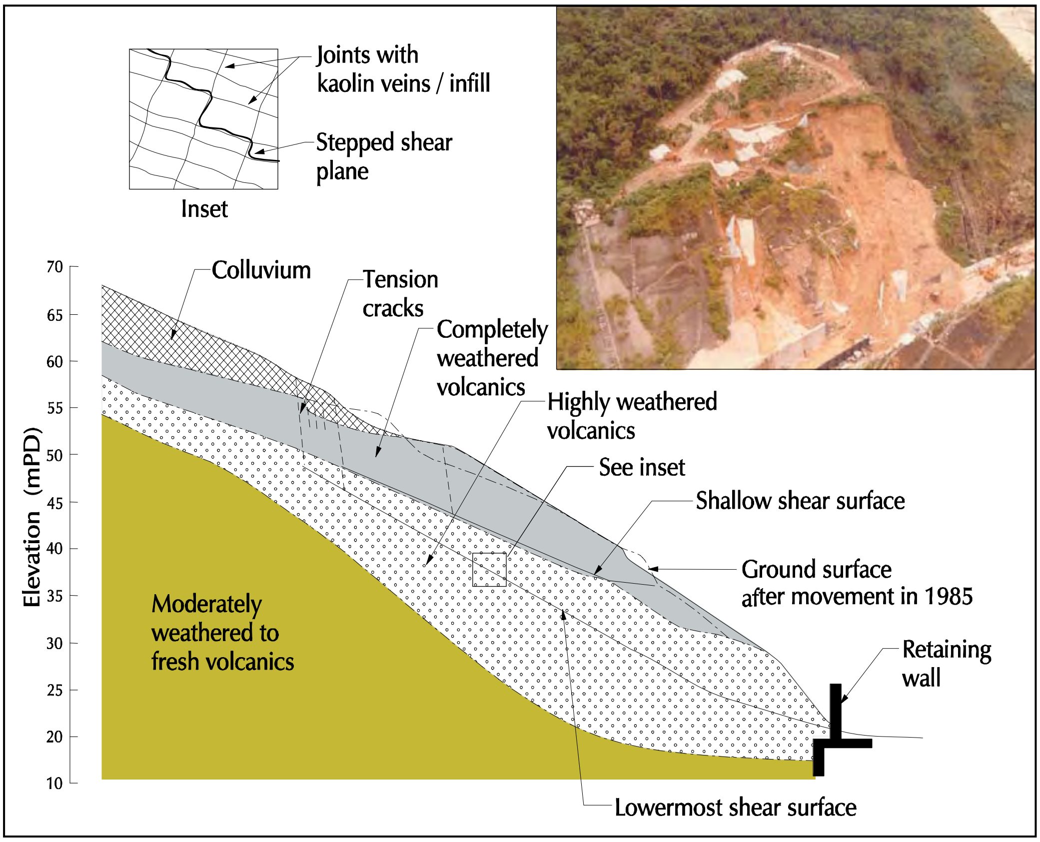


Figure 6.4.37 – Intermittent failure and progressive, deeper-seated movements at Tin Wan Hill Road resulting from complex geological structure, weathering and hydrogeological regimes (after Irfan, 1986)

(1986). As part of the remedial works to the slope, a system of long horizontal drains was installed to control the groundwater.

Whiteside (1987, 1996) reports on the drainage characteristics of the slope, based on continuous monitoring of piezometers and out flows from the drains over a period of 65 hours during multiple rainstorm events. The results showed that groundwater flow was taking place preferentially , with only 20% of the drains accounting for about 85% of the total flow (Figure 4.6.9), which reached a peak of almost 1,000 litres per minute for a short time. The characteristics of the out flow pattern indicated that some drains only carried water during periods of high flow. It was concluded that the drains were effective in controlling the overall groundwater regime by reducing the piezometric head in the more permeable zones which were responsible for raising overall groundwater levels. High permeability zones and local variations in groundwater drawdown have also been reported in other cases where horizontal drains have been installed (Figure 4.6.9).

1978-1997 Lai Ping Road Landslide

The shallow landslides of 1997 involving about 4,000 m^3 of material and progressive movements of a deeper seated landslide of about 100,000 m^3 are documented in Sun & Campbell (1999). The detailed engineering geological characterisation of the landslide is documented in Koor & Campbell (2005). An oblique aerial photograph of the site with key features identified is shown in Figure 6.4.38, and a geological section through the distressed zone is shown in Figure 6.4.39.

Investigations carried out after the 1997 landslide indicated that some piezometers responded rapidly to rainfall, with groundwater rises of about 10 m during rainstorms. The investigations also showed that a depression in rockhead tended to concentrate groundwater flow into the zone of distress and increased the relative groundwater levels above rockhead.

Voids and possible soil pipes or sheared zones were encountered in the investigation drillholes (Figures 6.4.39 and 6.4.40). A large soil pipe was

Figure 6.4.38 – 1997 shallow failures at Lai Ping Road and location of main scarp of older landslide (Sun & Campbell, 1999)

also exposed near the western end of the failure (Figure 4.4.10), and further soil pipes up to 1.5 m wide and 2 m high were encountered during the subsequent excavations for the remedial works.

The hydrogeology was also affected by dilated relict joints striking across the slope and many deep tension cracks up to 750 mm in width. The most representative groundwater model, simulated using 2-D transient seepage analysis for the 1997 rainstorm, took account of a highly permeable, sheared zone just above rockhead, and the high permeability of the open relict joints and tension cracks. The results of the analysis (Figure 6.4.41) show that the very high final groundwater profile modelled is similar to that inferred to exist at the time of the shallow landslides.

The depression in rockhead and complex hydrogeology with an extensive network of soil pipes probably existed before the cut slope was formed in about 1978. Changes to the profile of the hillside, and possible blockage or collapse of soil pipes causing build-up of groundwater pressures, may have led to the formation of the deep-seated zone of distress.

This example illustrates the difficulties in formulating realistic hydrogeological models in a complex area even when a large amount of investigation data is available. In general, the presence of large soil pipes, deep weathering and a rockhead depression may indicate potentially adverse hydrogeological conditions. However, soil pipes are particularly difficult to identify in areas where the soil profile is not exposed.

Figure 6.4.40 – Anomalous zone in saprolite interpreted as being either an infilled soil pipe or high permeability sheared zone (Koor & Campbell, 2005)

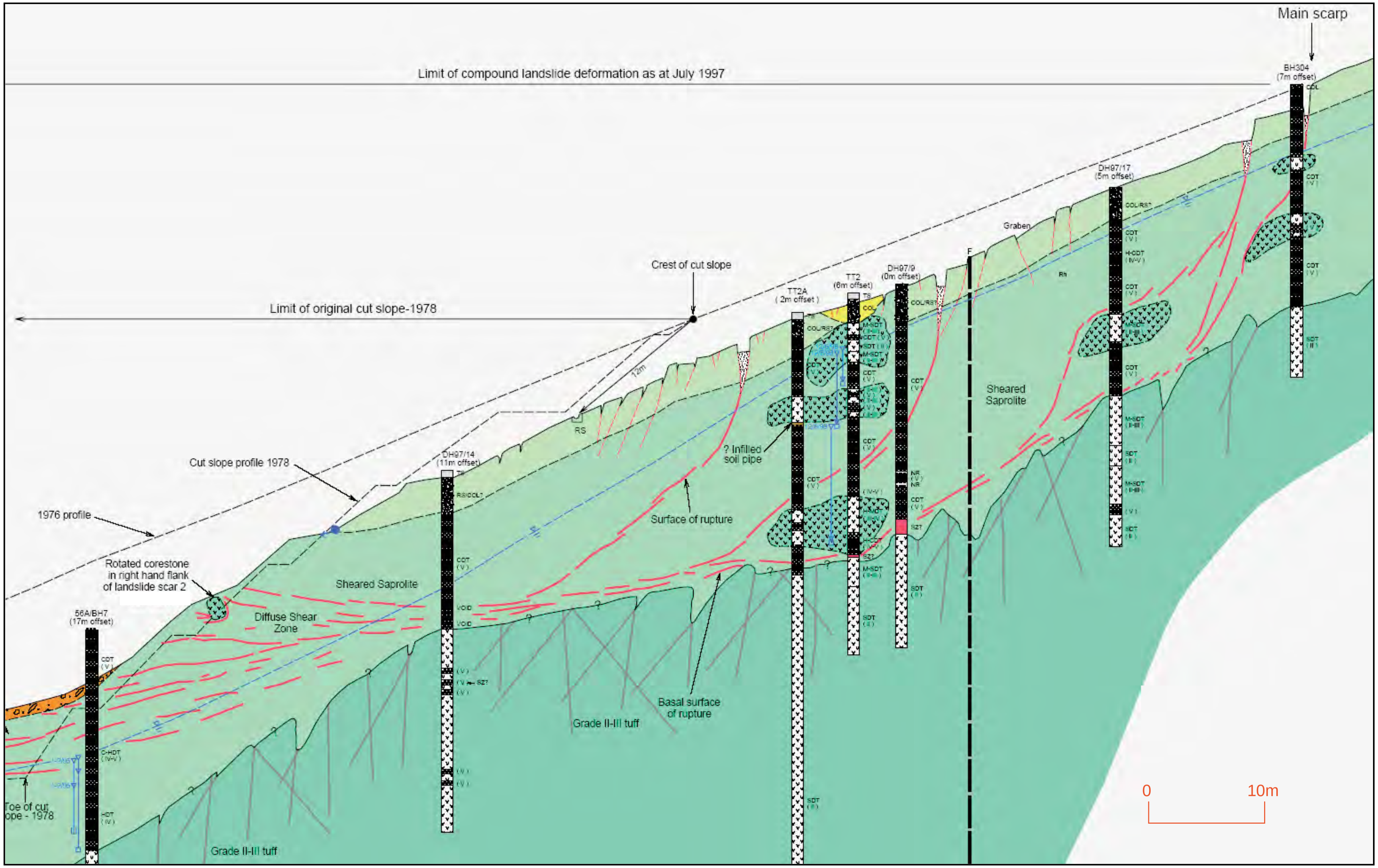


Figure 6.4.39 – Extract of section through the Lai Ping Road landslide showing deep weathering, high groundwater and shear planes associated with deep-seated movements (Koor & Campbell, 2005)

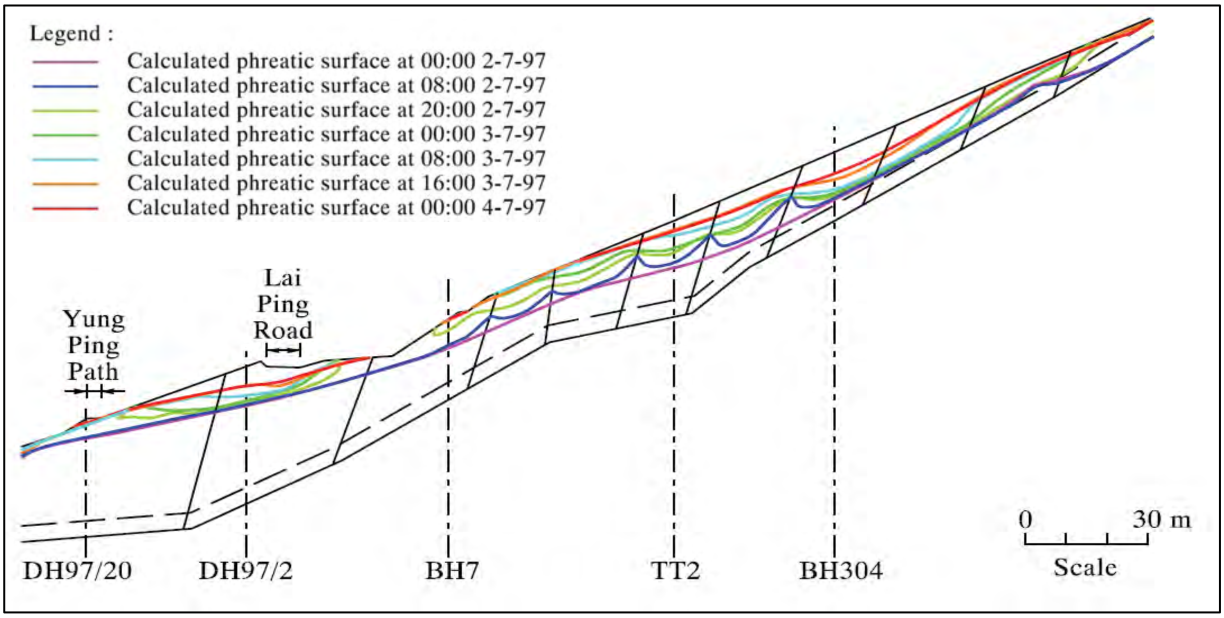


Figure 6.4.41 – Seepage analysis of the 1997 Lai Ping Road landslide assuming higher permeability of sheared zone above rockhead and accounting for the effect of open relict joints and pre-existing tension cracks (Sun & Campbell, 1999)

1954-1997 Ching Cheung Road Landslides

Investigations of these landslides in 1997 are documented in HAPL (1998b). This case provides an example of delayed failure and high groundwater, suggesting adverse hydrogeological conditions.

Figure 6.4.42 shows the 1997 landslide with remedial backfilling operations underway. Also shown is a surface depression which was reported to remain dry even during heavy rain. Figure 6.4.43 shows a section through the failed area and the location of inferred soil pipes intersected by the drillholes during investigation of the landslides. Although some zones containing soil pipes may be related to previous shear planes or tension cracks, the density of similar features must be very high for five drillholes of 100 mm in diameter to intersect as many as 13 of these features.

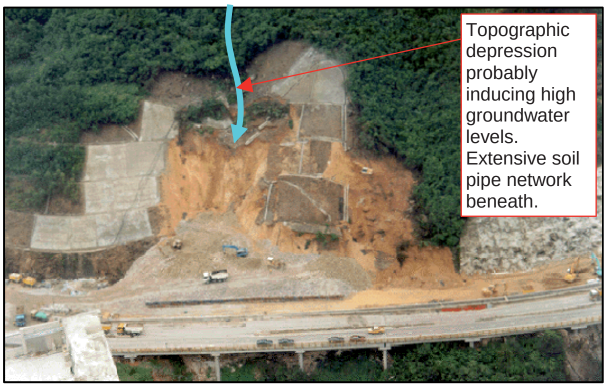


Figure 6.4.42 – 1997 Ching Cheung Road landslide intersected by natural drainage line (HAPL, 1998b)

It was inferred that most of the groundwater that is channelled towards the surface depression is normally carried through the slope via a dense network of soil pipes. Blockage of these soil pipes by natural collapse or small ground movements is thought to have led to a build-up of groundwater pressure and caused failure of the slope.

Anderson *et al.* (1983) note that where drainage lines or depressions are intersected by cut slopes, special attention should be paid to the development of conservative yet representative hydrogeological models and the need for detailed investigation and groundwater monitoring.

2002-2003 Investigation of Soil Pipes at Yee King Road

Following the collapse of a footpath into a cavity upslope of the crest of a cut slope along Yee King Road, Hong Kong Island, the general area was investigated to determine the extent of under ground voids and drainage pathways. The development of geomorphological and hydrogeological models of the area from API, field mapping and ground investigations is described in HCL (2003a). The collapse revealed a cavity of about 1.5 m by 3 m in plan and about 6.5 m deep.

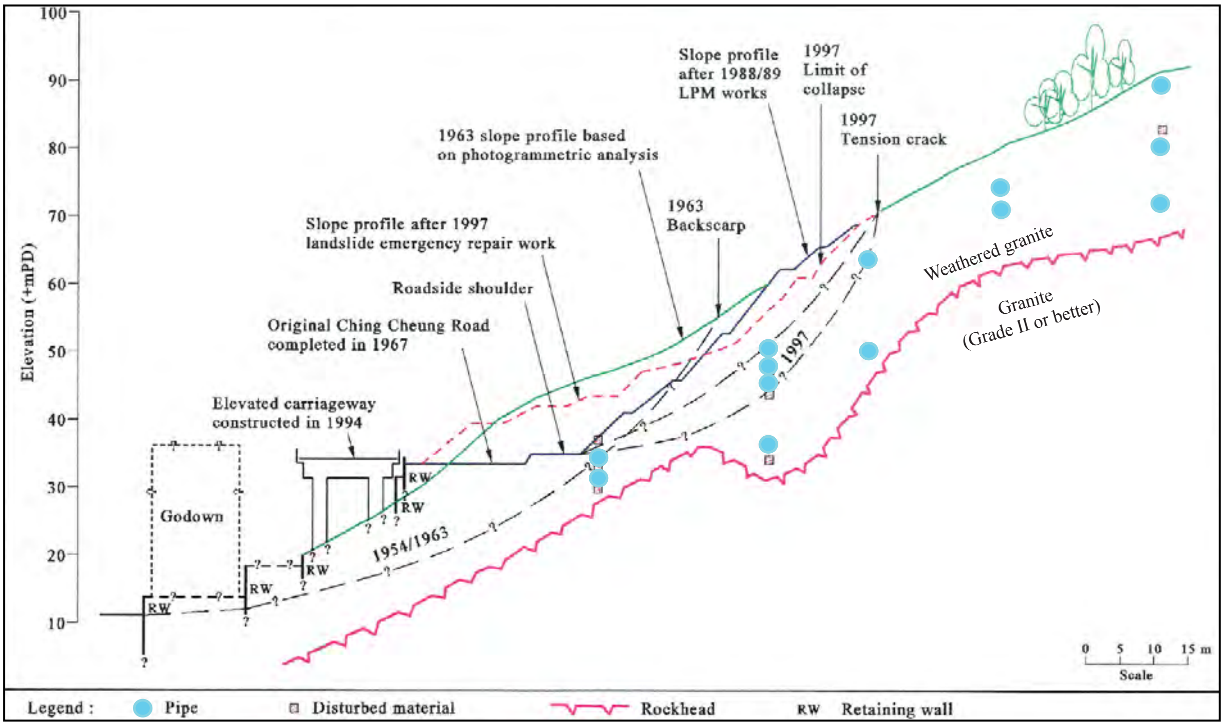


Figure 6.4.43 – Section through 1997 Ching Cheung Road landslide showing previous failures and the large number of soil pipes encountered in drillholes (HAPL, 1998b)

Figure 6.4.44 shows the inferred sequence of events which led to the development of underground stream flow and erosion tunnels (very large soil pipes) beneath the boulder field, formed by undercutting of the more resistant, finer-grained Mount Butler granite which overlies the more decomposed and coarser Kowloon granite.

During the investigation, some of the erosion tunnels were physically examined, and a geophysical resistivity survey was carried out across the hillside in an attempt to locate similar features. Two apparently continuous tunnels were located and three apparently shorter tunnels were identified.

The development of such process-based models based on detailed API and geological mapping combined with engineering geological knowledge and judgement is beneficial, particularly where complex hydrogeological conditions are suspected.

The example also serves to indicate the potential significance of areas where drainage lines lose their definition in bouldery areas, particularly below a concave break in slope. In such cases, the surface water which formed the drainage line in the relatively less permeable, steep terrain may sink into the more permeable deposits forming the flatter lying ground.

Soil pipes are more likely to form where the deposits are heterogeneous. Accumulation of bouldery deposits may also choke a surface drainage line over time, leading to sub-surface flow within buried channels.

Pipe Erosion Model for Cut Slopes in Saprolite

Figure 6.4.45 shows a model developed by Nash & Chang (1987) to explain the widespread pipe erosion and slumping in the surface of the completely decomposed tuff of the remediated Pun Shan Tsuen slope. Pipe erosion along relict joints has also been observed in other slopes containing easily erodible material such as Tuen Mun Area 19 and Ma On Shan (Gray, 1986).

Development of similar models to Stages 1-4 shown in Figure 6.4.45 may be a useful exercise when investigating the effects of stress relief on sub-vertical joints, cleft water pressure and general ingress of water into vulnerable slopes where 'sinkholes' or eroded relict joints have been detected.

6.4.8 Difficulties during Installation of Soil Nails

A review of cases where problems were encountered during soil nail construction between 1993 and 2003 is documented in Ng *et al.* (2004a). Several cases involving difficulty in drilling or excessive loss of

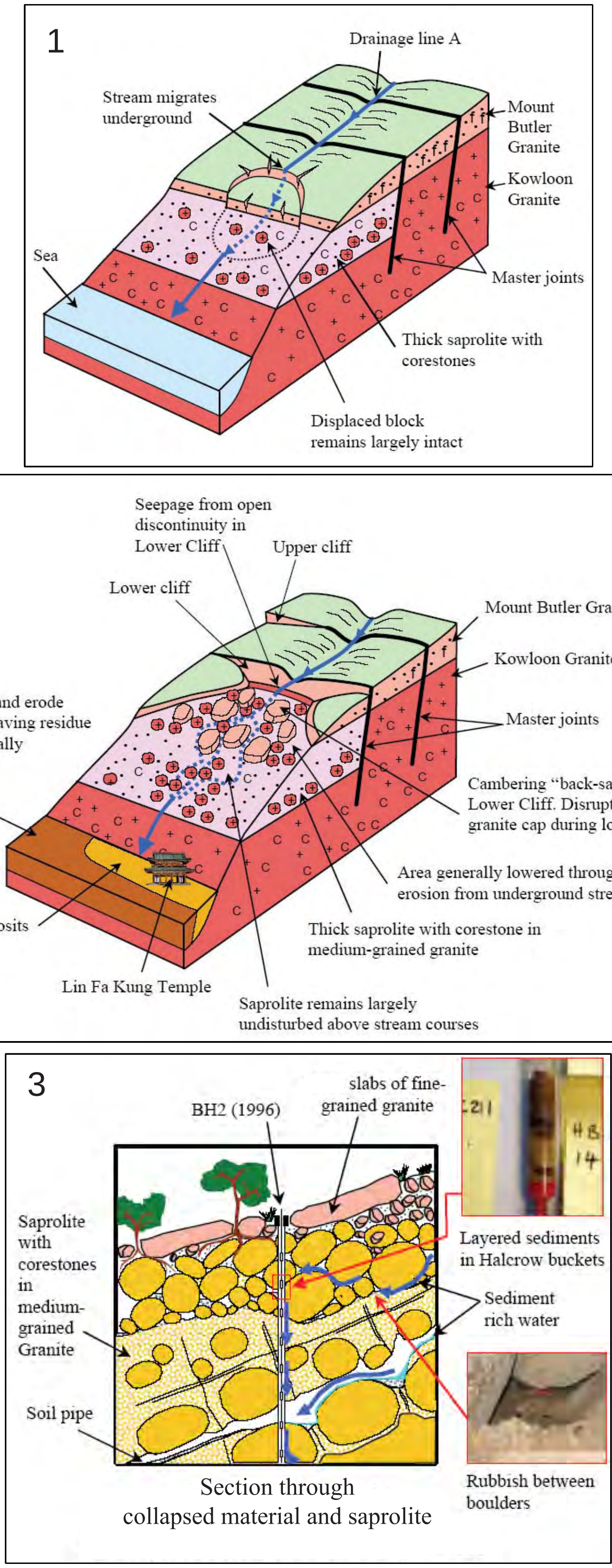


Figure 6.4.44 – Hydrogeological model for development of erosion tunnels/soil pipes at Yee King Road (after HCL, 2003a)

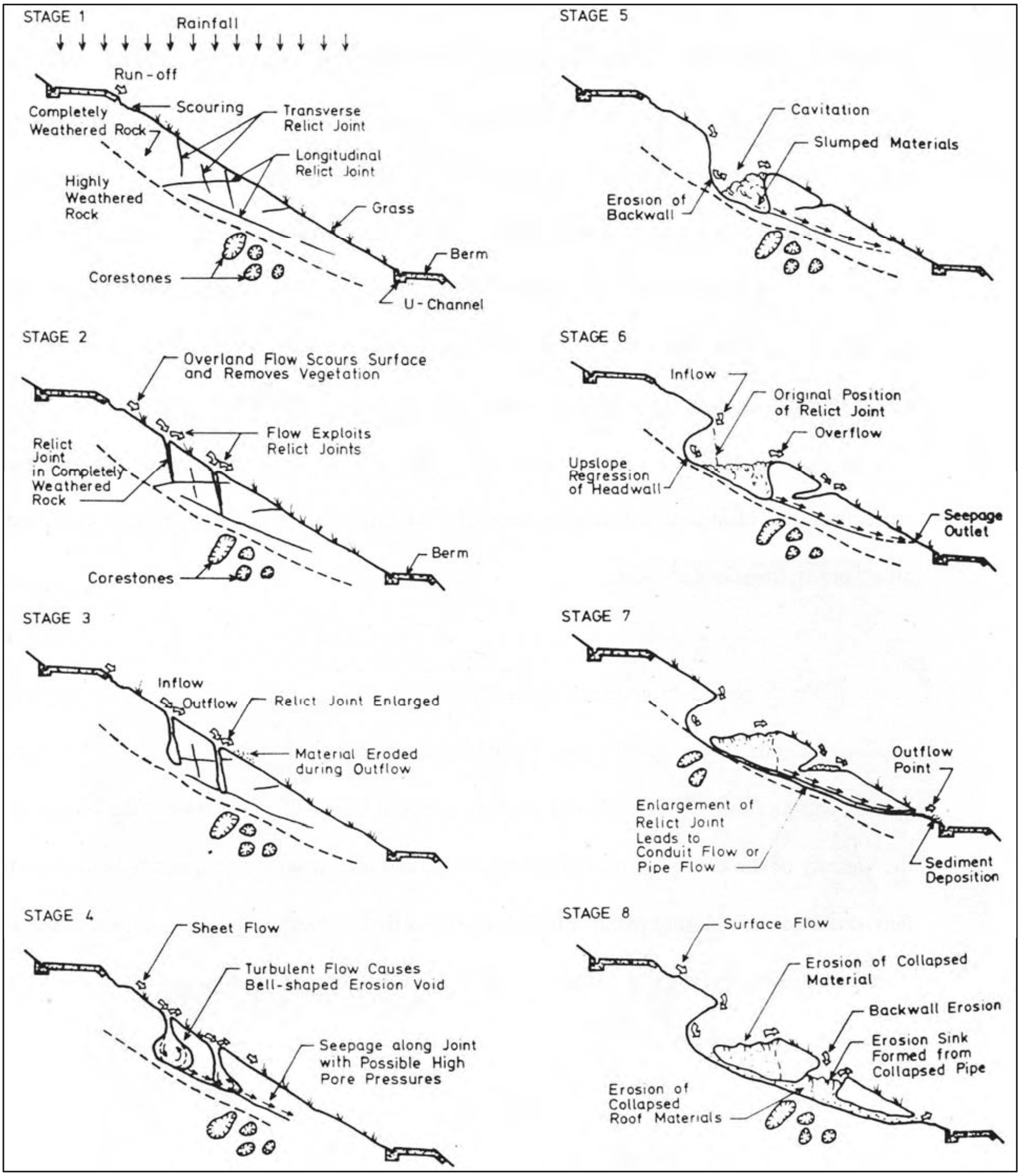


Figure 6.4.45 – Pipe erosion model for cut slopes in saprolite based on observations of slumped areas and tracer dye testing on the remediated large instability at Pun Shan Tsuen in 1985 (Nash & Chang, 1987)

grout were primarily caused by adverse geological conditions. In particularly difficult areas, the soil nailing works were abandoned and alternative slope upgrading works were carried out.

Based on the review, the following geological conditions that may contribute to excessive grout loss were identified:

- Generally permeable coarse materials with a relatively low silt/clay content in the matrix and moderate to high intergranular porosity, including:
  - fill, containing a significant proportion of coarse material, i.e. boulders, cobbles, gravel and sand, and
  - colluvium and fluvial deposits, containing

a high proportion of coarser material, i.e. boulders, cobbles, gravel and sand.
- Features and geological settings within which discrete pathways enable enhanced fluid through-flow, including:
  - erosion pipes, which may be partly infilled by porous and permeable material,
  - material boundaries within colluvium, and between colluvium and *in situ* material, and within corestone-bearing saprolite, especially at the margins of corestones, open joints, faults and shear zones, and other discontinuities (e.g. zones of hydrothermal alteration, etc.) that are weathered and eroded, and so are open,
  - landslide scars, tension cracks, and other features related to slope deformation, as these may include voids within transported and *in situ* materials, and
  - drainage lines intersecting slopes, within which colluvium may be present, erosion pipes may be developed, and preferred groundwater through-flow indicated by seepage locations/ horizons, may also occur.

The presence of the above conditions may serve as an initial screening to help assess the possibility of grout loss problems, although their existence may not necessarily mean that excessive grout loss will occur at a particular site, and vice versa. In general, drainage lines with colluvium are potentially most vulnerable to grout loss during soil nail construction. Sites with open joints and/or erosion pipes are also vulnerable to grout loss.

In order to better identify the potential for problems during slope upgrading works, it is useful to refer to the geological model developed for the assessment and design of the slope, and to critically examine the need for any further development of the model.

6.5 FOUNDATIONS

6.5.1 Introduction

This section highlights the potential effects of geological and geotechnical variability described in other chapters with specific application to foundation works. References and examples which illustrate key engineering geological issues are also provided.

The key requirements for the geological and ground models include:

- early recognition of potential geological and/or geotechnical complexity (Chapters 3, 4 and 5),
- identification of relevant variations in geology, weathering profiles and material properties which may affect foundation performance (Chapters 4 and 5),
- assessment of external factors which may affect or be affected by the foundation, such as the groundwater regime, lateral loading from slopes, and stability issues during and after construction (Sections 4.6, 6.3 and 6.4),
- identification of key areas of geotechnical uncertainty for further investigation, and
- verification and updating of the geological and ground models during construction.

The scale and detail of engineering geological inputs that may be required at the different stages of a major foundation engineering project include:

- site-specific assessments at the feasibility and preliminary design stages to identify potentially complex or adverse conditions, and
- detailed examination of pre-bored and post-bored rock cores at the construction and post-construction stages to assist in assessing the required depth and likely performance of individual piles.

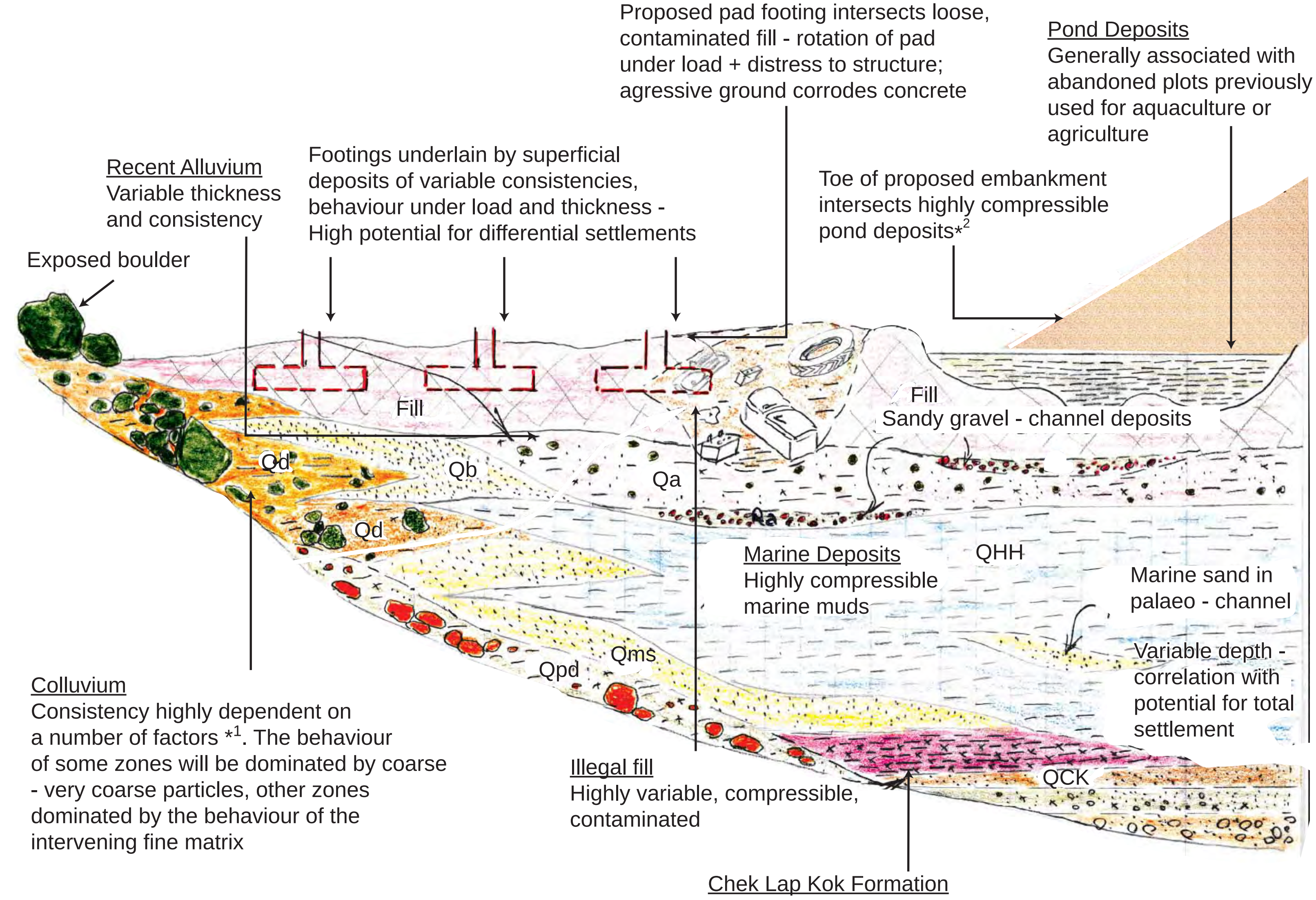
Less detail may be appropriate for small projects. Detailed guidance for the design and construction of foundations in Hong Kong is given in GEO (2006).

6.5.2 Shallow Foundations on Soil

Shallow foundations on superficial deposits and saprolite are commonly employed for pavements, light structures and utilities. The main geotechnical inputs are directed towards ensuring sufficient bearing capacity and minimizing differential settlements.

The focus of engineering geological input is therefore on uncertainties regarding the geological profile that may need to be investigated depending on the design specifications. A schematic section showing the types of feature giving rise to possible concerns is shown in Figure 6.5.1. Typical problems associated with shallow foundations include:

- Uncontrolled fill: Likely to be highly variable, possibly with a high organic content or contaminants and may contain compressible artefacts with voids.
- Pond deposits: Highly compressible, recent accumulations of fine sediment in natural and man-made depressions. High organic contents may lead



Notes
*1 source material; type of failure; distance from source
*2 pond deposits could be removed and replaced if not too extensive

Figure 6.5.1 – Shallow foundations – potential geotechnical problems caused by geological variability in superficial deposits

to much secondary consolidation if left *in situ*.
- Alluvium: This material can be expected to vary both vertically and laterally due to varying depositional environments with time.
- Holocene marine deposits: Extensive onshore areas of the northwest New Territories are underlain by this highly compressible material.

The site investigation requirements will vary according to the type and extent of the foundation structure, anticipated geological model and the effective depth of influence of the loaded area. Aggressive ground conditions and explosive gases, such as methane and hydrogen sulphide that can accumulate in, or beneath, the proposed structure, may also need to be investigated.

Examples of site investigations and settlement monitoring of road embankments constructed on soft sediments in the northern New Territories and Mainland China are contained in Chan (1987), Walsh *et al.* (1988) and Thorn *et al.* (2001). In particular, these highlight the problems associated with highly compressible, organic pond deposits and marine mud.

6.5.3 Shallow Foundations on Rock

Shallow foundations on rock (decomposition Grade III or better) are commonly employed for both low and high intensity loads where the rockhead level allows. Given the relatively high strength of the intact rocks, problems typically devolve into two broad categories:
- Bearing capacity problems associated with joint-controlled failure.
- Settlement problems associated with weathered seams or movements along joints in otherwise sound rock.

At the initial stages of a project, engineering geological inputs are usually focused on the identification of any adverse geological structures (Section 4.2) and the potential for a variable weathering profile (Section 4.3). The weathering profiles in plutonic rocks and volcanic rocks with relatively widely-spaced jointing can be highly complex as illustrated in Figures 4.4.6, 4.4.9 and 6.7.13.

A sound appreciation of the regional and local geology greatly facilitates the planning of investigations for both shallow and deep foundations where adverse geological structures and complex weathering profiles may be present (Whiteside, 1988). As noted in Section 3.2, site-specific engineering geological assessments are required to complement the limited information shown on the published geological maps.

In areas where a high density of existing ground investigation and foundation records are available, knowledge of the regional and local geological structure can be used to develop detailed rockhead models (Figure 3.2.5) to assist in assessments of the feasibility of different foundation types.

The geology and foundation engineering considerations for the shallow foundations of the Tsing Ma suspension bridge are respectively described in Sewell (1992) and Lau & Wong (1997). The variable geology associated with the tower foundations for the landing of the Tsing Ma Bridge on Ma Wan is shown in Figure 6.5.2 and consists of a series of slices of faulted tuff and complex dykes with highly fractured contact zones.

As indicated in Figure 6.5.2, vertical drillholes tended to give the impression that the steeply inclined, highly fractured zones were of substantial thickness, while in reality, the zones are relatively thin (as proven in inclined drillholes and subsequent exposures). In this case, the variable geology did not lead to complex geotechnical conditions, since all the rocks were of similarly high intact compressive strength, and the highly fractured zones were relatively thin when compared to the footing dimensions and were steeply inclined.

Approximate estimations of rock mass deformability can be obtained using rock mass classification systems in conjunction with detailed drillhole logging or engineering geological mapping of foundation excavations. For blast-disturbed masses, the RMR system is particularly well-suited to obtaining deformability parameters, due to accommodation of a joint aperture function in the system. This method was used for the assessment of suitable founding levels for airport buildings at Chek Lap Kok in blast-disturbed rock masses (Pinches *et al.*, 2000).

Engineering geological mapping is also important for large scale foundations such as dams and power stations where it has proved valuable in identifying potentially problematic ground conditions. An example of an engineering geological plan of the west dam foundation for the High Island reservoir is shown in Figure 6.5.3 (Watkins, 1979). Figure 6.5.4

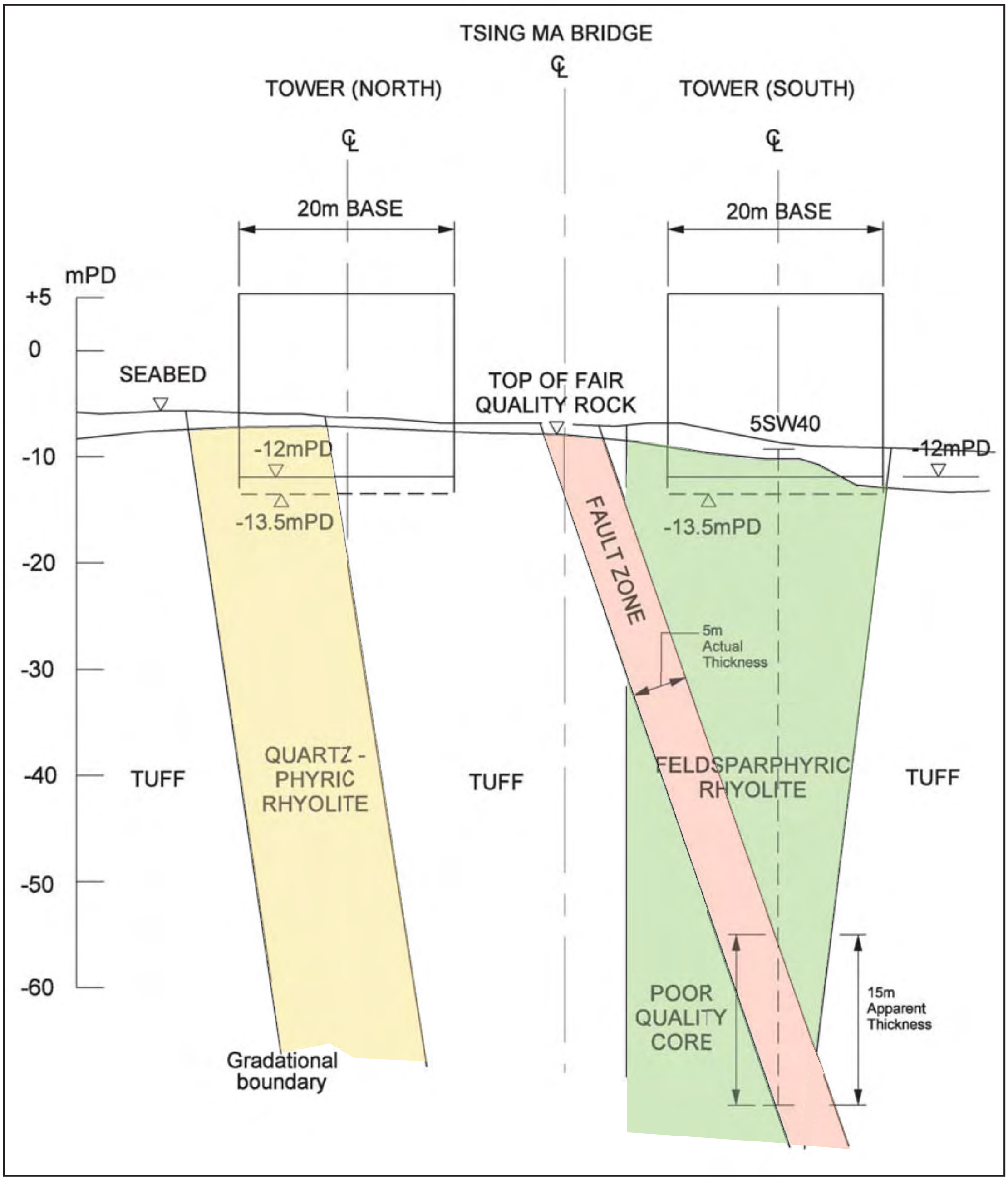


Figure 6.5.2 – Tsing Ma Bridge – geology of the Ma Wan Tower sites (Sewell, 1992)

shows further details and the grout hole layout to form a groundwater cut-off where the core of the dam crossed a faulted and weathered zone. Some of the engineering difficulties associated with the adverse geological structures encountered during construction are described in Vail *et al.* (1976).

6.5.4 Pile Foundations on Rock

End-bearing, large diameter bored piles founded on rock are among the most prevalent foundation types adopted in Hong Kong for the support of large structures and buildings.

These are usually designed on the basis of either a prescriptive approach (based on rock compressive strength, decomposition grade and core recovery) or on a rational approach which may be based on rock mass classifications using the RMR system (Hill *et al.*, 2000; Littlechild *et al.*, 2000; GEO, 2006).

Whichever method is adopted, a detailed geological model of the site needs to be developed to assess the ground conditions and the possible presence of adverse features such as corestones, coreslabs, faults, weathered seams and irregular rockhead profiles.

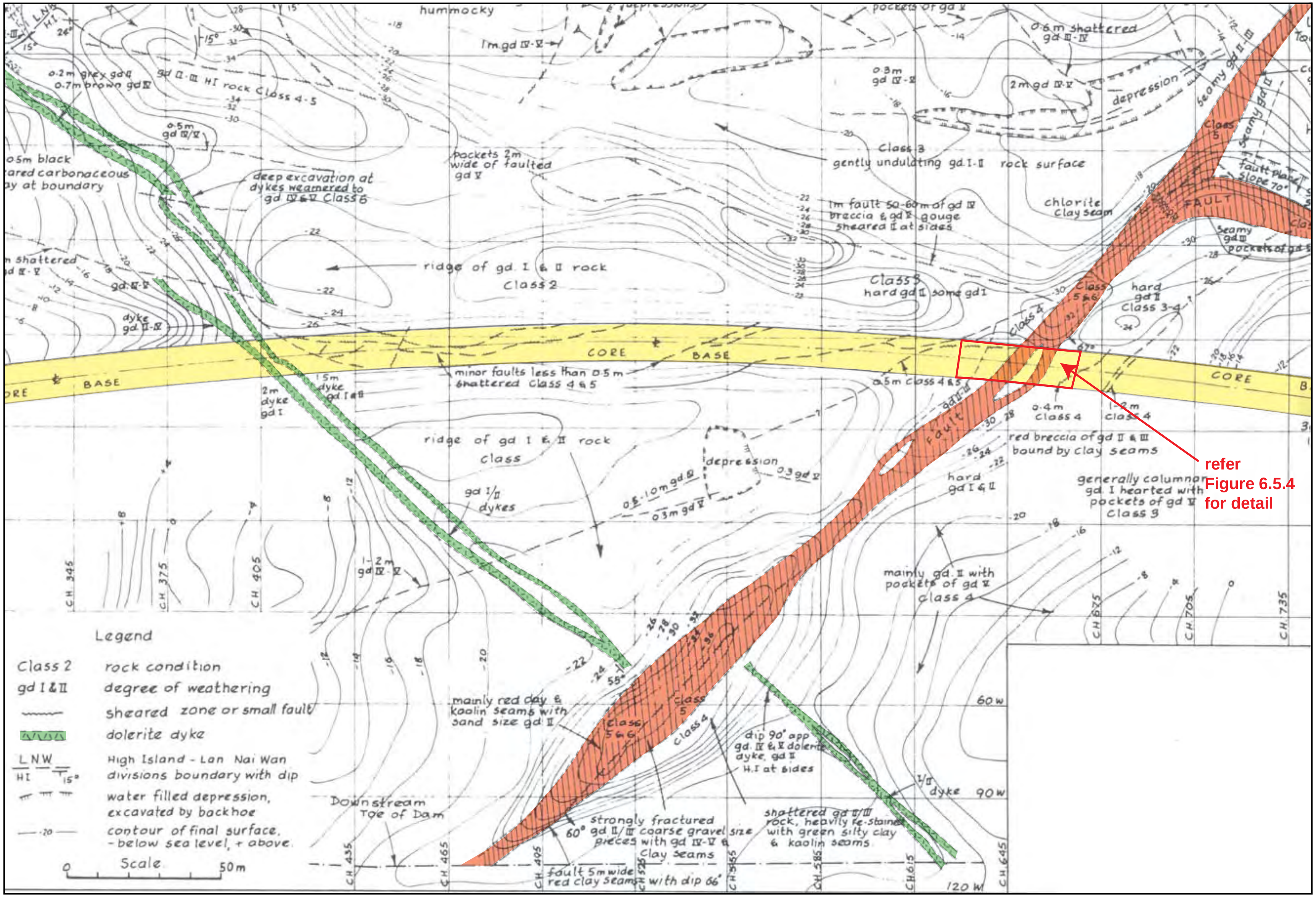


Figure 6.5.3 – Extract from an engineering geological plan for the West Dam foundation of High Island Reservoir (after Watkins, 1979)

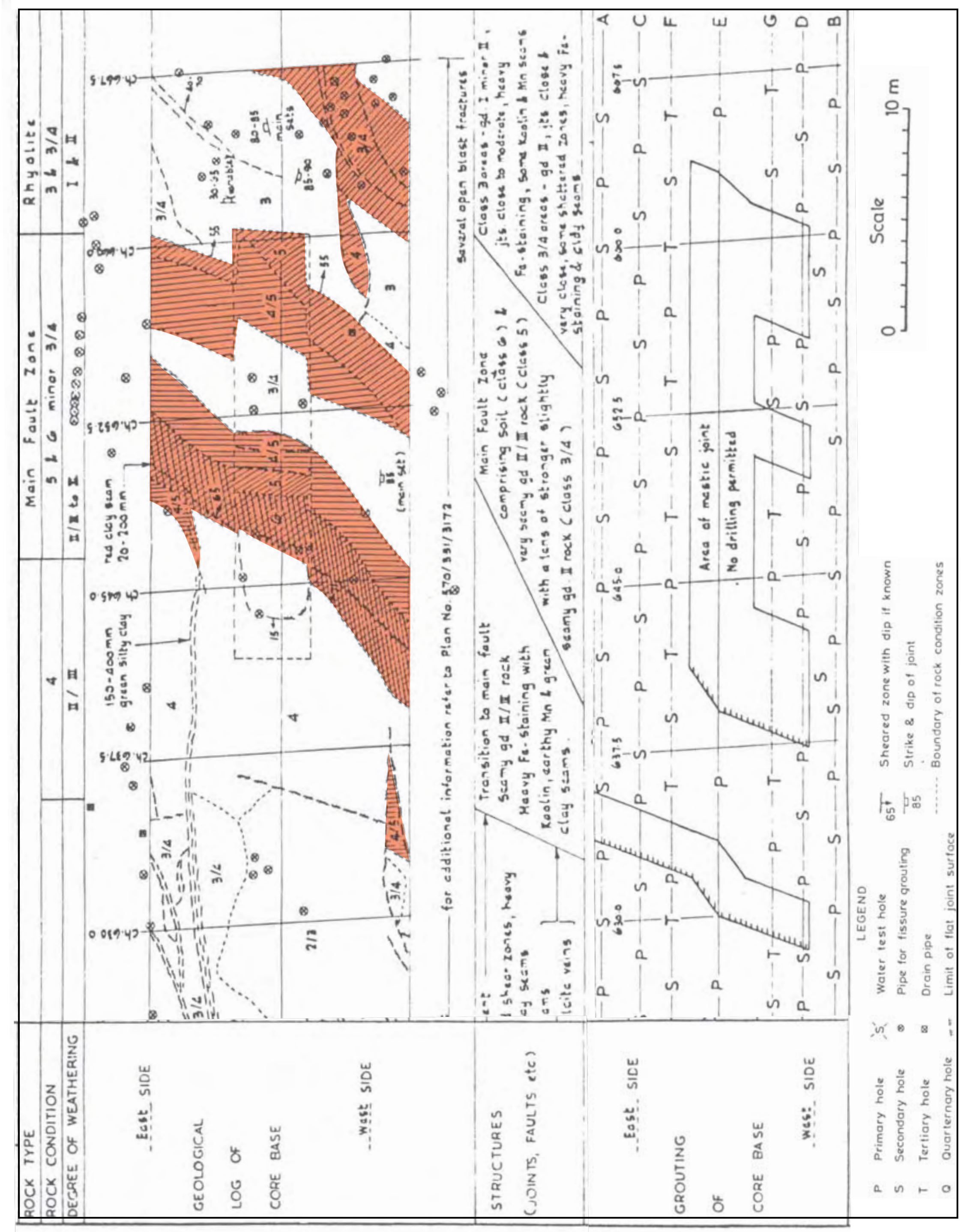


Figure 6.5.4 – Detail of the engineering geological plan and grouting layout for the groundwater cut-off beneath the core of the West Dam of the High Island Reservoir (after Watkins, 1979)

The geological model is best built up progressively using more detailed investigations as the project progresses. In areas of particularly adverse ground conditions, early recognition of excessive depth to rockhead which may preclude the viability of end-bearing piles or rock sockets can result in considerable cost savings.

Toe levels are generally defined on the basis of pre-construction drillholes at proposed pile locations. However, the nature of the rock mass can lead to significant complications where gradational profiles or weathered seams are present (Irfan & Powell, 1985).

Figures 6.5.5 and 6.5.6 illustrate some common difficulties in determining suitable founding levels for bored piles in rock. Common pile construction problems related to irregular rockhead profiles include:

- telescoping of casing through soil seams between corestones above founding level,

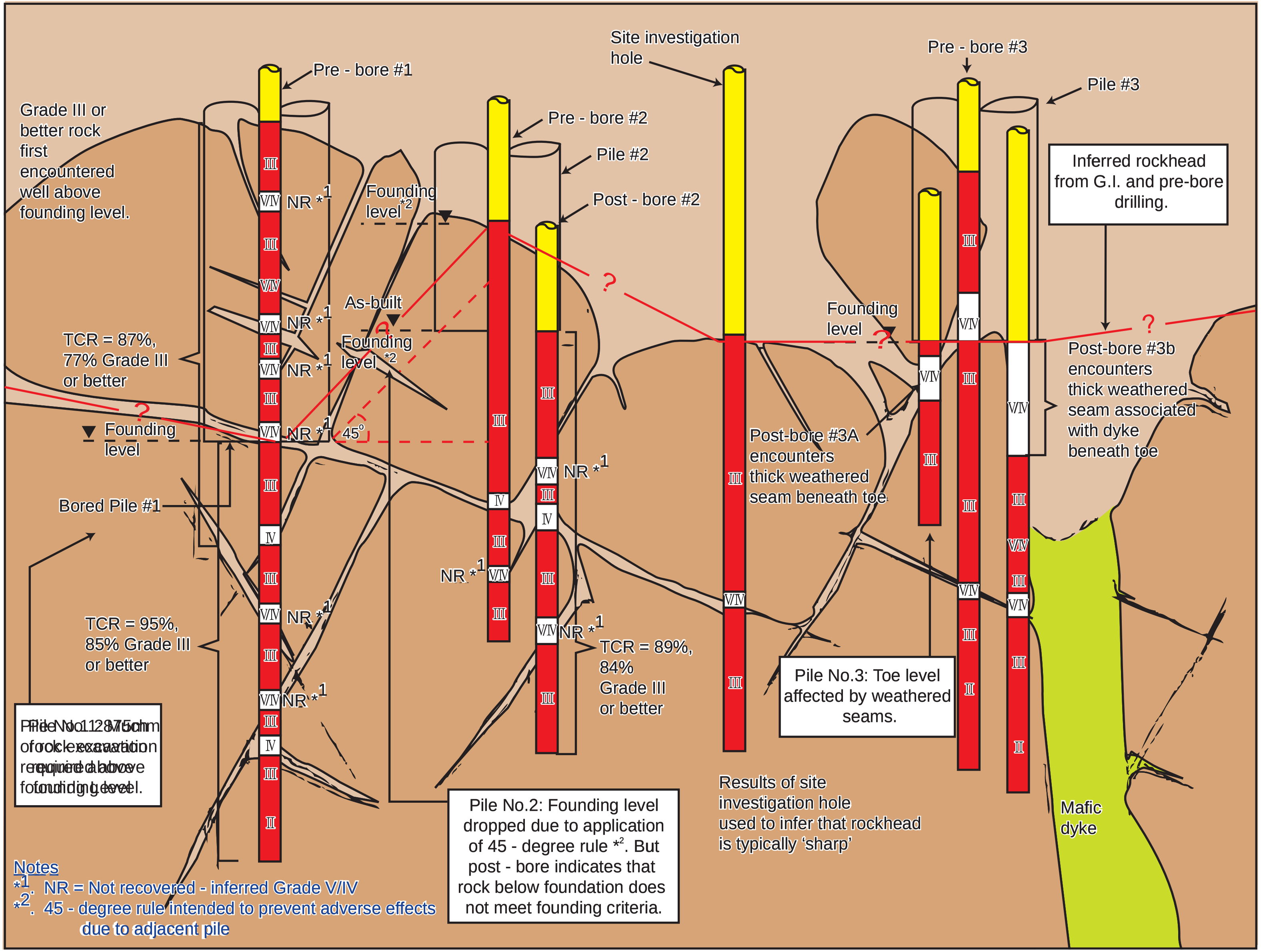


Figure 6.5.5 – Piles to rock – potential problems associated with weathered seams

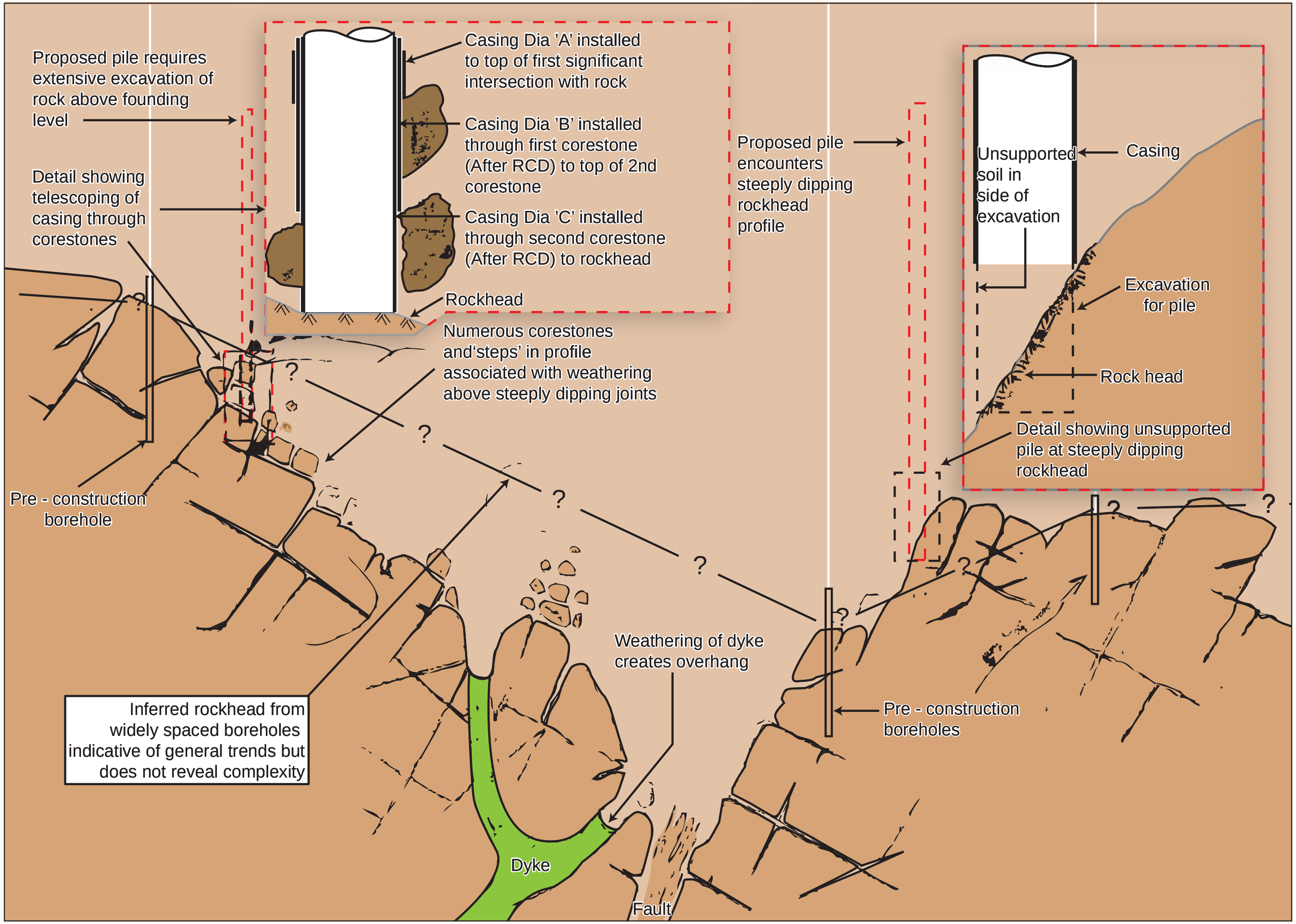


Figure 6.5.6 – Piles to rock – Potential problems associated with an irregular rockhead profile

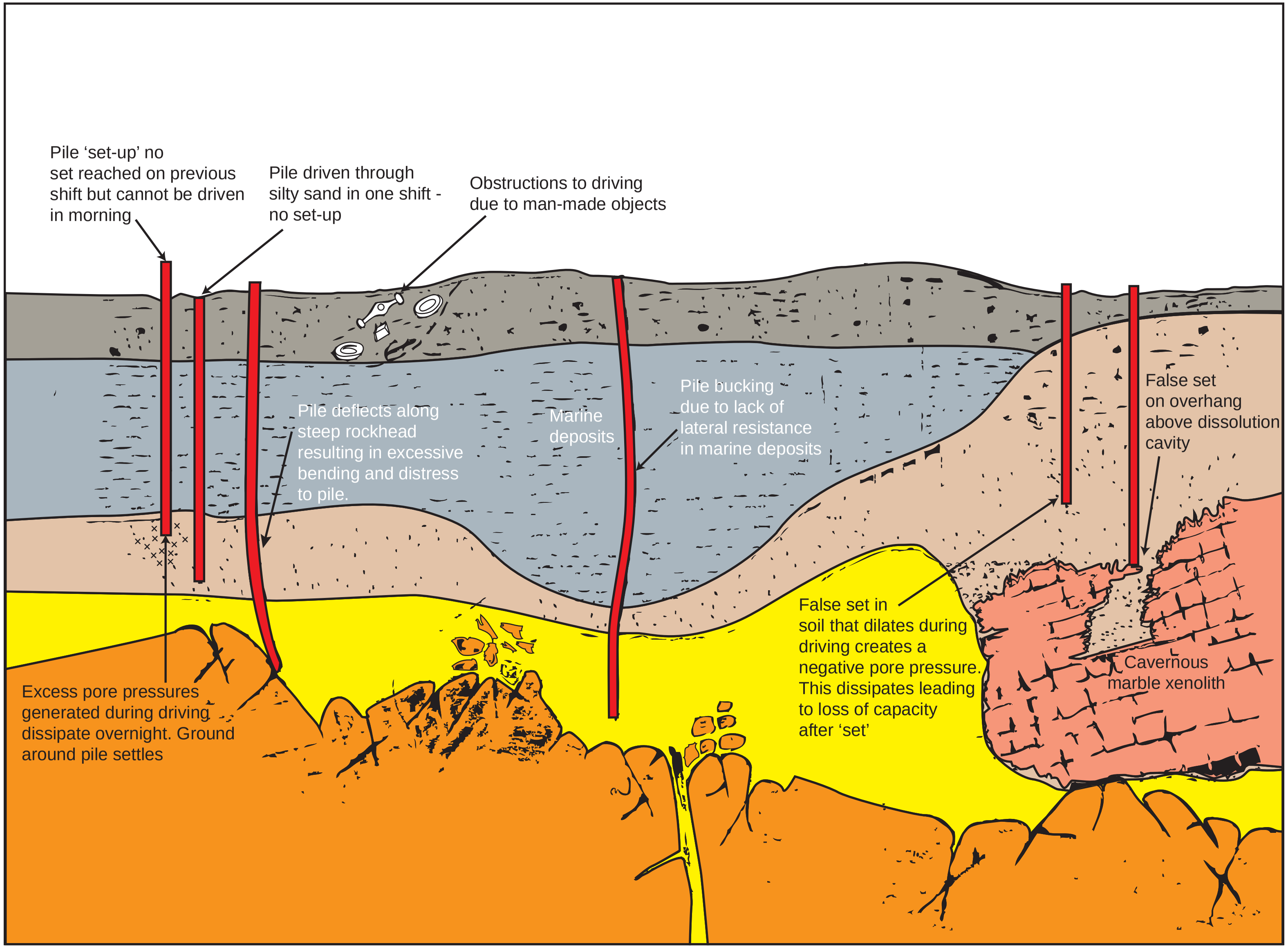


Figure 6.5.7 – Driven piles – potential problems caused by geological variability

- possible collapse of unsupported soil into the pile excavation where the casing cannot form an adequate seal hard against steeply dipping rockhead, and
- discovery of extensive weathered seams in post-construction drillholes.

6.5.5 Friction Pile Foundations in Soil

Friction piles in saprolite have been used in Hong Kong. These include driven steel H-piles and other pile types involving driving, vibrating or less commonly boring. Some of the most common problems (Figure 6.5.7) associated with engineering geological issues are:

- obstructions due to man-made objects, boulders or corestones,
- deflections due to steeply dipping rockhead levels leading to bending and distress to the pile,
- false sets due to the relaxation of high locked-in stresses during driving,
- buckling and instability due to installation through thick marine deposits,
- false sets in areas of cavernous marble, and
- false sets due to the build-up and later dissipation of pore water pressures generated during driving.

6.5.6 Foundations on Marble

The characteristics of different types of marble and marble bearing rocks, including the use of the MQD rock mass classification system to characterise the potential founding properties of marble (Chan, 1994; Chan & Pun, 1994) are described in Section 5.5.

With regard to geological and geotechnical complexity in the Scheduled and Designated Areas containing marble (Section 5.5.5), reference should be made to the requirements and guidance contained in the following references:

- BD (1993) for the Yuen Long and Ma On Shan Scheduled Areas,
- BD (2004b) and GEO (2004k) for the Designated Area of northshore Lantau,
- GEO (2005c) for supplementary technical guidelines for foundation design in areas underlain by marble and marble-bearing rocks, and
- BD (1997, 2000) for ground investigation requirements and site supervision.

In areas where the presence of marble is a possibility detailed and phased site investigations are required to establish the geological model, as interpretation of drillhole data in isolation from an appreciation of the overall site setting is fraught with potential problems (Fletcher, 2004). GEO (2006) contains guidelines on the investigation methods which are appropriate for sites underlain by marble.

The most extensive areas underlain by relatively pure marble are the Yuen Long and Ma On Shan Scheduled Areas. Examples of the wide variety of dissolution features revealed during detailed ground investigations for foundations are given in Chan (1994), Chan & Pun (1994), Chan *et al.* (1994) and Leung & Chiu (2000). These show that in general, a dense grid of drillholes is necessary to define the ground conditions in sufficient detail for the purposes of designing piled foundations. Detailed knowledge of the structural geology of the area and knowledge of the development of karst morphology will assist in the interpretation of existing information and optimise each successive phase of ground investigation.

An unusual situation was encountered during investigations for a tower block in Tung Chung underlain by cavities in marble xenoliths (Fletcher *et al.*, 2000; Wightman *et al.*, 2001). In this case, the published geology (Figure 6.5.8) consisted of granite

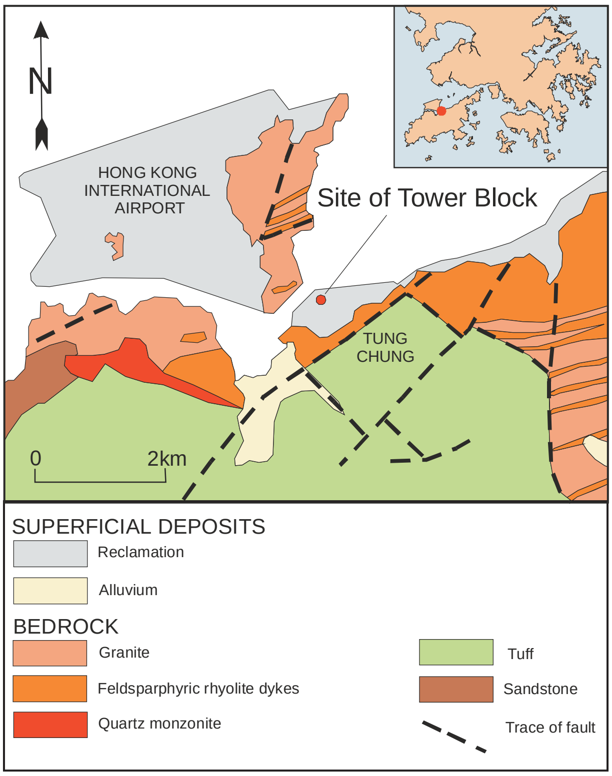


Figure 6.5.8 – Geological setting and site location plan of proposed tower block at Tung Chung (after 1:20,000-scale geological Map Sheet 9, GEO 1994)

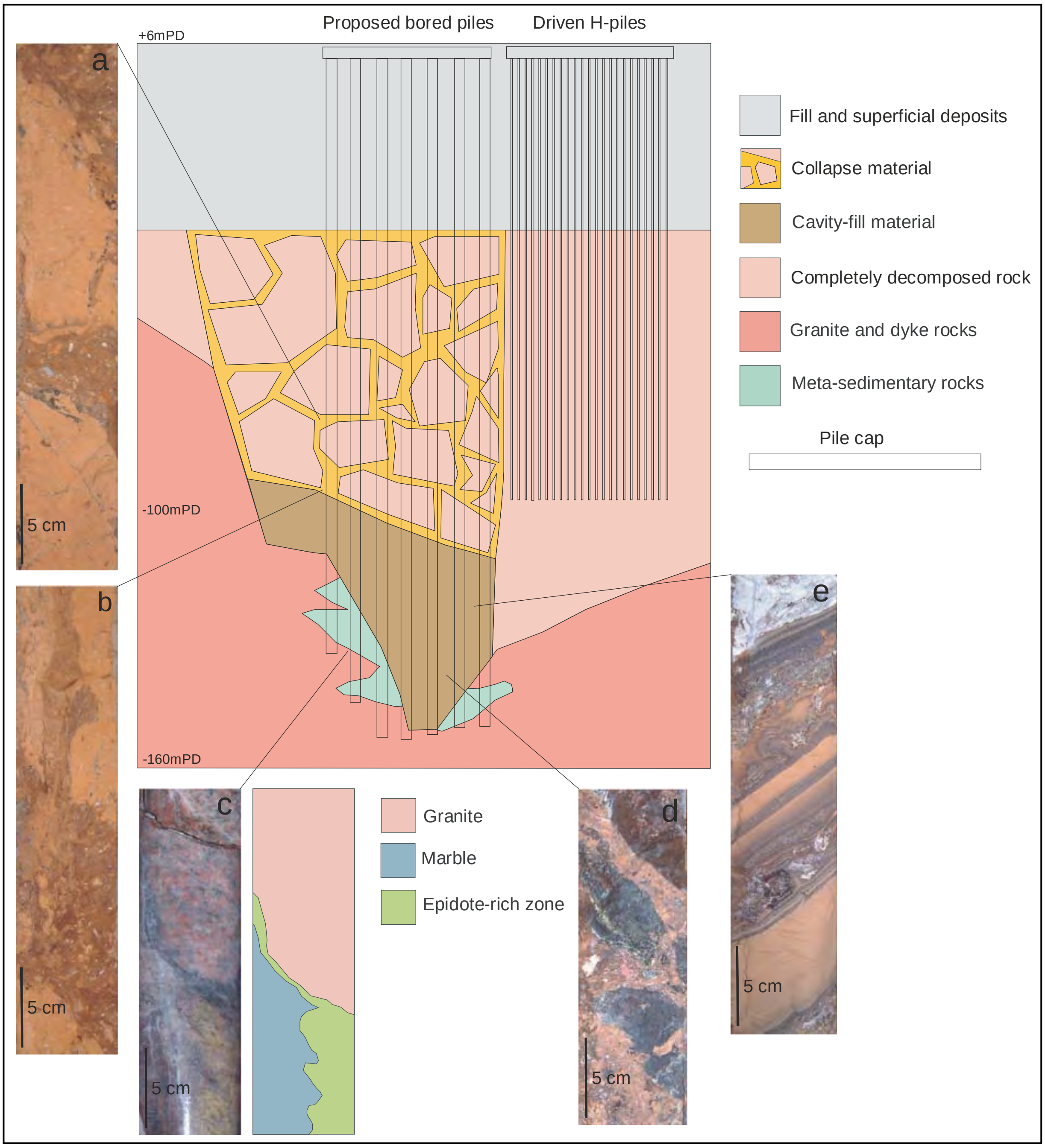


Figure 6.5.9 – Cross-section of the ground conditions beneath the proposed tower block in Tung Chung (after Fletcher et al., 2000). a. Mazier sample of sedimentary breccia between blocks of completely decomposed feldsparphyric rhyolite. b. Soil pipe in completely decomposed feldsparphyric rhyolite. The pipe extends upwards from a layer of sedimentary breccia. c. Contact between granite and marble with sketch of relationships between rock types. d. Sedimentary breccia within cavity-fill material. e. Finely laminated clay and silt in cavity-fill material

intruded by multiple feldsparphyric rhyolite dykes, which in turn are in faulted contact with a thick sequence of volcanic rocks (GEO, 1994).

During the site investigations it became apparent that the geology of the reclaimed offshore areas along the coastline is significantly different. Beneath the reclamation, the superficial deposits (up to 50 m thick) are underlain by meta-sedimentary rocks, cavity-fill sediments and other karst-related deposits (Figure 6.5.9). In places, the bedrock has been completely decomposed to depths in excess of

-150 mPD (Gillespie *et al.*, 1998; Kirk, 2000). These materials were incorrectly identified previously, due to inaccurate or inadequate descriptions of the geological materials (Kirk, 2000).

The initial ground investigations for the proposed tower block suggested that completely decomposed granite extended to about -150 mPD and that large cavity-type features, up to 12 m in height, were present at depths below -100 mPD. In order to investigate the ground more fully, a second ground investigation was conducted (Wightman *et al.*, 2001). Above -100 mPD, mazier or U76 samples were taken at 3 m intervals with Standard Penetration Tests carried out in between. Below -100 mPD continuous sampling was carried out using mazier or U76 sample tubes, with coring in rock. To enhance recovery, drilling mud comprising a mix of water, polymer and bentonite, was used. As a result sample recovery, even in cavity fill, was extremely good.

The material below -100 mPD comprises a layered sequence of laminated clay, silt, sand, gravel and sedimentary breccias as shown in Figure 6.5.9 (Fletcher *et al.*, 2000). In addition, several thick blocks of completely decomposed meta-sedimentary rock were found within the sedimentary sequence. These are overlain by completely decomposed feldsparphyric rhyolite blocks separated by thin units of dark brown sedimentary breccia, silt, clay and sand (identified as 'collapse material' in Figure 6.5.9). Similar soil types were also recovered from above -100 mPD. Apparently infilled soil pipes extend upwards from some of the sedimentary breccia layers. These materials overlie *in situ* slightly decomposed and fresh meta-sedimentary rocks, which include marble, meta-sandstone and meta-siltstone, skarn, feldsparphryic rhyolite, and hydrothermally altered granite.

Fletcher *et al.* (2000) proposed that the layered sedimentary sequence below -100 mPD accumulated in a cavity formed from the dissolution of a large marble block enclosed in granite. The former cavity was at least 30 m across and 50 m high, but was probably completely filled with sediment washed in through karstic drainage channels, and fallen blocks from the roof to the former cavity. The overlying completely decomposed feldsparphyric rhyolite blocks and surrounding sedimentary breccias above -100 mPD are considered to have formed during the progressive decomposition and partial collapse of the roof of the cavity (see Figure 5.5.7).

In the zones where cavities were indicated in the initial ground investigation, it was noted that the penetration rate increased, but there was no significant loss, or viscosity reduction, of the drilling fluid (Wightman *et al.*, 2001). This, together with a range of data from downhole geophysical surveys (sonar, electrical cylinder resistivity, cross-hole radar and gamma density), suggests that the cavities are infilled with sediments.

The detailed ground investigations showed that the form of the rockhead surface is highly variable beneath the reclamation at Tung Chung. To the west of the site it is fairly constant at around -60 mPD, but across the proposed tower block footprint the rockhead surface is steeply inclined and reaches over -150 mPD at the northeast corner of the site (Figures 6.5.9 and 6.5.10).

An understanding of the complex nature of the bedrock geology was essential to develop the geological model for the solid and superficial materials. The complexity of the ground had many engineering ramifications and the proposed tower block was not built, due mainly to the steep inclination of the rockhead surface and the presence of loose cavity-fill materials at great depth.

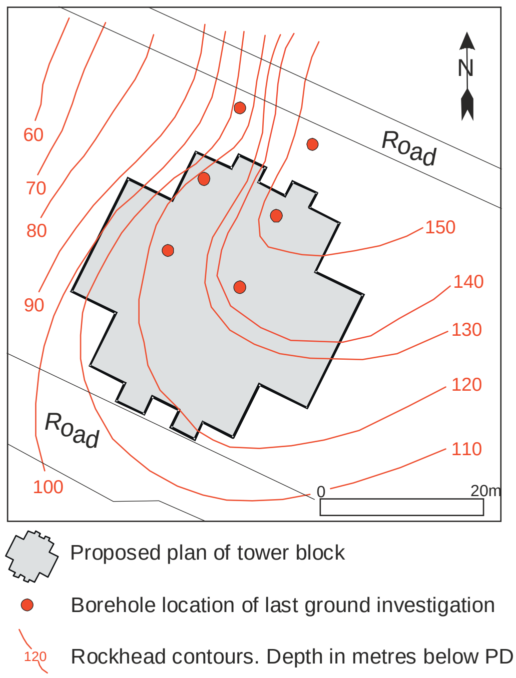


Figure 6.5.10 – Rockhead contour map of the site for the tower block and location of boreholes

6.6 DEEP EXCAVATIONS

6.6.1 Introduction

This section highlights the application of geological and ground models to deep excavations and provides references and examples which illustrate selected key engineering geological issues.

Deep excavations are commonly defined as being greater than 6 m in depth (GCO, 1990b) with very steep or vertical sides. They include supported excavations in soil and also rock excavations which may be supported or unsupported, depending on the stability of the rock mass.

Guidance for the design and construction of deep excavations in Hong Kong is given in GCO (1990b) and GEO (2004m). Requirements for submissions to the Buildings Department with respect to dewatering in basement and foundation works are contained in BD (1994).

The main engineering issues associated with deep excavations are the retention of the adjacent ground, groundwater control and prevention of damage to nearby structures, land and services.

The engineering geological issues and key requirements for geological and ground models are similar to those outlined in Sections 6.3 (Site Formation) and 6.5 (Foundations). However, an important issue associated with deep excavations is the need for detailed determination of the hydrogeological characteristics of the site and its surroundings, to facilitate the design of possible groundwater control measures to limit settlement and base heave during excavation.

Characterisation of the excavated materials for reuse or disposal can also be an important issue, particularly where the ground or groundwater may have been contaminated. Determination of the aggressivity of the ground and groundwater is also required to enable adequate durability measures to be incorporated into the design of the permanent structure.

6.6.2 Excavation and Support Types

Typical construction methods for deep excavations are described in GCO (1990b). Selection of the most appropriate method is influenced by the ground conditions, required depth and layout of the excavation, site constraints, proximity to sensitive structures and programming considerations.

The main considerations with regard to the ground conditions are the strength, compressibility and variability of the excavated and supported materials, their hydrogeological characteristics and the groundwater regime.

Excavation type is therefore often principally determined by the nature of the ground (rock or soil) and the relative position of the groundwater table. The four broad settings generally encountered are listed below, along with commonly applied support and groundwater control methods:

- Above the groundwater table – predominantly in soil: unreinforced slopes where space permits; reinforced slopes; braced, tied-back or cantilevered sheet pile or soldier pile walls.
- Above the groundwater table – predominantly in rock: steep or vertical slopes, stabilised as necessary with reinforced shotcrete, rock anchors and/or dowels, depending on the geological structure of the rock mass.
- Below the groundwater table – predominantly in soil: unreinforced slopes where space permits, but with the need for dewatering and generally some form of groundwater cut-off; braced, tied-back or cantilevered retaining walls, ranging from sheet pile walls at shallow depths, giving way to pipe-pile walls, secant pile walls or diaphragm walls as depth increases; with groundwater ingress controlled by the wall construction and supplementary grouting.
- Below the groundwater table – predominantly in rock: as in the second item above, with soldier pile walls (pre-bored H-piles or the like) where heavily fractured rock is expected; fissure grouting as necessary for groundwater exclusion.

6.6.3 Risk Management and Engineering Geological Input

Reviews of 34 cases of collapses and excessive displacements of deep excavations which occurred in Hong Kong between 1980 and 1995 are contained in Man & Yip (1992) and OAP (2002). The main causes were mostly related to non-compliance with the original design and/or poor workmanship. However, unexpected geological and hydrogeological conditions, combined with inadequate planning and precautions, were also found to be important contributory factors in some cases. An example of a collapse adjacent to a sheet-piled excavation where obstructions due to boulders prevented an adequate depth of embedment of the piles is shown in Figure 6.6.1.

The studies noted above indicate that appropriate risk management procedures are essential to the successful design and construction of deep excavations, and that the development of reliable geological and hydrogeological models is a key factor. Guidance on risk management procedures in respect of tunnel works is contained in GEO (2005b). The general principles outlined in this publication may be adapted to suit the needs of site-specific deep excavation projects.

Engineering geological input at the early stages of the project helps gain an understanding of the likely materials and hydrogeological regimes for feasibility/planning purposes. Later input during the investigation and detailed design stages helps determine the expected range of ground conditions for design of the structure and further refinement of monitoring requirements and risk mitigation or contingency measures.

Figure 6.6.1 – Collapse of road due to boulders preventing adequate embedment of sheet piles retaining adjacent excavation

6.6.4 Excavations in Rock

General

Deep excavations extending down into rock over tens of metres are not uncommon. They frequently involve extending vertical cuttings beneath the toe of strutted excavations in soil. Common problems to be overcome with respect to these cuttings include:

- lack of exposures at the surface leading to onerous or inadequate design assumptions,
- toe stability of walls supporting deep excavations in the overlying soil, and
- groundwater exclusion.

Excavations in rock may require intensive engineering geological input to help determine the likely support requirements prior to construction, and also during excavation where confirmatory mapping of the exposed rock mass needs to be carried out.

Assessment of Geological Structure

Lack of exposures at the ground surface will lead to uncertainties regarding the structural stability of the unsupported rock mass. Most of the information regarding rock mass discontinuities at the site of the proposed excavation is often extracted from drillhole records or from the mapping of exposures at some distance from the site.

Whilst drillhole discontinuity surveys (impression packer or acoustic televiewer) provide valuable data with respect to orientation, no definitive information regarding persistence can be recovered. Extrapolation of orientation, nature and persistence data from exposures is often useful, but caution is required. The size of the exposure is often limited relative to the persistence of the joints (particularly subvertical joints) and the depth of the proposed excavation. As a result, there is considerable dependence on the development of reliable geological models.

Lack of definitive data at the design stage can lead to the adoption of conservative ground models, such as the assumption of large wedges day-lighting in the toe of the excavation, as described by Morton *et al.* (1984) regarding the original excavations for North Point Station. Where non-critical excavations can be made in the rock mass, such as adits driven from access tunnels, engineering geological mapping and interpretation of the exposed discontinuities can enable the final excavation support requirements to be optimised. Matson *et al.* (1986) describe the use of this method during bottom-up construction works for the deep basement of the MTRC Causeway Bay east concourse.

A combination of rock exposure and drillhole discontinuity data was used to provide preliminary assessments of rock reinforcement requirements for the 45 m deep excavation for the North Point MTRC Plant Building, constructed as part of the Quarry Bay congestion relief works in 1999-2002 (Figure 6.6.2).

Although the jointing pattern was generally favourable, the potential size and frequency of localised wedges was assessed from the available data using statistical methods developed by Pahl (1981) and Mauldon (1992, 1995). Engineering geological mapping during excavation was conducted to identify the need for stabilisation of local wedges. In general, only moderate stabilisation works were required and, with the exception of isolated occurrences of very persistent joints (Figure 6.6.3), the actual range of joint persistence (Figure 6.6.4) was similar to that predicted at the detailed design stage.

Support of Retaining Walls Founded Near the Edge of Rock Excavations

For deep excavations in soil and rock, the wall retaining the soil portion may subject the rock mass to high vertical and horizontal forces. Stabilisation methods include: provision of a rock pillar between the toe of the wall and the rock cut, installation of shear keys, tie-back by rock anchors, bracing via a layer of struts, or carrying all or part of the toe of the wall to the base of the excavation in rock.

With the exception of the latter option, detailed engineering geological mapping of the rock mass at the base of the wall is required to assess the stability of the loaded rock mass and the need for additional reinforcement.

Groundwater Exclusion

Excessive groundwater inflows may lead to settlement problems in the overlying soil adjacent to the site. Prediction and control of in flows in rock are often problematic from technical and contractual points of view, particularly since it is dif ficult to implement successful post-excavation fissure grouting. As in the case of rock tunnelling (Section 6.7), detailed engineering geological assessments help identify potentially adverse hydrogeological controls such as dykes, faults and deeply weathered zones, and rock masses adjacent to faults which may contain more open joints. Where such features have been identified, more intense investigations of the rock mass permeability should be considered to better assess the potential hazards.

6.6.5 Excavations in Soil

General

For excavations in soil, where the primary retaining structures and dewatering elements are installed prior

Figure 6.6.2 – Initial excavations for the MTRC North Point plant building

Figure 6.6.3 – MTRC North Point plant building during excavation showing limited support and occasional very persistent joints

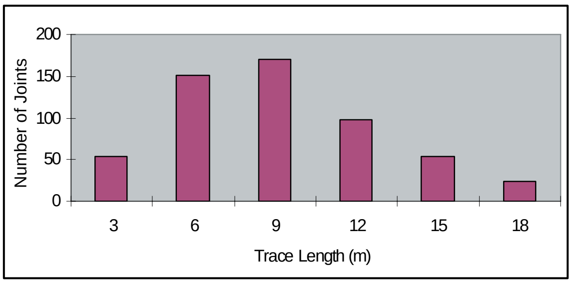


Figure 6.6.4 – Trace length distributions of joints (≥3 m) measured in the MTRC North Point plant building and station excavations

to exposure of the ground at depth, a considerable level of confidence in the robustness of the scheme is required. In addition to determining the basic materials and hydrogeology of the site and its surroundings, key engineering geological inputs include:

- identification of variations in soil types and properties to assist in relating spatially variable soil test results to the ground and design models,

- identification of variations in permeability that may affect groundwater drawdown or base stability during construction,
- assessment of the variability of the ground with regard to obstructions, corestone-bearing profiles and irregular rockhead,
- planning and interpretation of pumping tests, with particular attention to possible hydrogeological factors which may give rise to variable drawdown responses, and
- interpretation of unusual ground or groundwater responses arising from unforeseen ground conditions during construction.

Case histories which illustrate some of the difficulties and resulting settlements during construction of deep excavations in soft ground are contained in Thorley, (1985, 1986a, 1986b) and Buttling (1990).

Strength and Deformability Estimates

The strength and deformability of soils may be measured or indirectly estimated from laboratory and *in situ* tests, including large-scale pumping tests. An example of a pumping test and comparison of predicted and back-analysed ground movements to establish deformability and permeability parameters for design of the Dragon Centre in Shamshuipo is described in Lui *et al.* (1995).

Comparison of back-analysed Young's modulus parameters with other *in situ* tests such as SPT 'N' values has enabled broad correlations to be established for preliminary design. However, such correlations are site-specific, and significant variability can be expected due to differences in each site's geological history. A typical problem that can arise concerns averaging of the 'N' values that may in fact form separate populations when their spatial distribution is considered. In saprolite, this may be due to horizontal and/or vertical variations in original grain size, hydrothermal alteration, weathering intensity and leaching (see Section 4.4.3). Superficial deposits can also exhibit large lateral and vertical variations in grading and engineering properties, due to changes in depositional environments with time.

A geological model which is further developed in the light of soil test results can assist in constraining the spatial applicability of test data, leading to improvements in the design model used for analysis and when interpreting ground movements during pumping tests.

Permeability Estimates

Dewatering is a clear priority for the successful performance of open excavations beneath the water table. Excavations to be carried out with no cut-off wall or curtain are particularly vulnerable to variation in permeability values, which are difficult to establish accurately from the results of *in situ* or laboratory tests. Some of the more commonly encountered problems are:

- poor estimates of permeability due to the scale effects associated with *in situ* testing,
- wide variations in permeability associated with non-uniform ground treatment coverage,
- high void ratios and very high permeability associated with pre-existing marine structures (e.g. stone revetments, rubble mounds, armour stones) buried within reclamation fill, and
- high permeability zones associated with highly fractured zones or open discontinuities in rock masses.

The problems associated with estimation of permeability are not solely confined to open excavations. They also have a direct impact on the determination of cut-off toe levels and dewatering provisions for flows beneath cut-off walls.

Groundwater Cut-off at Rockhead

Commonly, the most problematic zone for groundwater cut-off for deep excavations is at rockhead level. Often, the difficulties are associated with the following features:

- steeply dipping rockhead can result in incomplete penetration of the cofferdam wall (Figure 6.6.5),
- gradational rockhead due to abundant corestones leading to hanging cofferdam walls, with the possibility of wash-out of soil around the corestone due to high hydraulic gradients (Figure 6.6.5), and
- high permeability zones close to rockhead subjected to high hydraulic gradients: these features may contribute to the formation of a confined aquifer beneath less permeable saprolite.

The nature of the rockhead profile becomes crucial where the toes of diaphragm wall panels are to be constructed in rock, or where significant embedment into corestone-bearing saprolite is required. Diaphragm wall plant generally cannot excavate Grade II or better rock, and production rates in areas where much rock is to be removed are low.

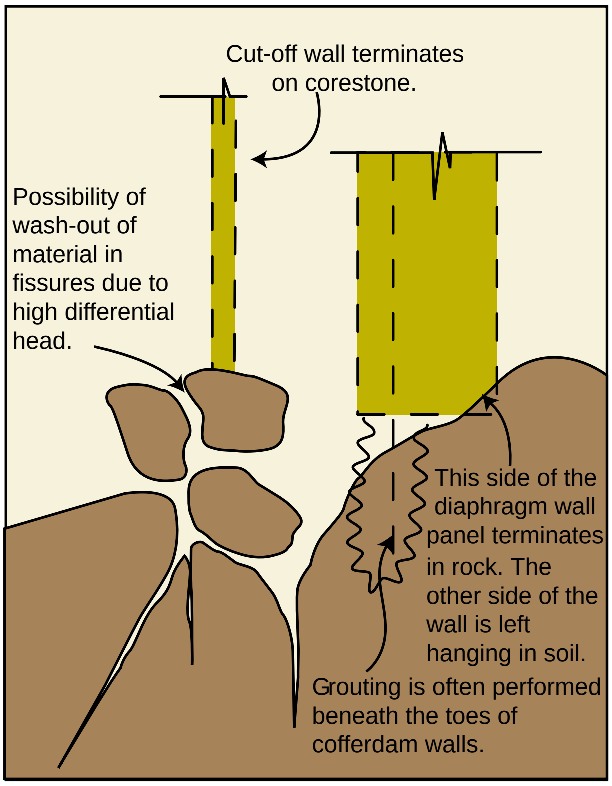


Figure 6.6.5 – Typical groundwater cut-off difficulties associated with irregular rockhead

Confined Aquifers
Typical groundwater models are based on the assumption that the rockhead acts as a relatively impermeable base for the main water table, with the majority of the groundwater volume residing within the more porous and more permeable saprolitic and superficial soils above rockhead. Studies by Jiao *et al.* (2001, 2003) suggest that not all piezometric and permeability data conform to the 'typical' model. These studies indicate that a more permeable zone at rockhead may act effectively as a confined aquifer, with elevated pore pressures in response to recharge upslope (Section 4.6). Permeability data at Wanchai MTR Station (Davies, 1987), and from a number of other case histories (Twist & Tonge, 1979; Cowland & Thorley, 1985) also suggested the existence of a highly permeable zone in weathered granite profiles near rockhead. Such confined aquifers can lead to increased groundwater inflow and/or base heave as the excavation nears the confined aquifer, unless adequate groundwater cut-offs and pressure relief provisions are made.

A similar situation involving a locally confined aquifer commonly arises with deep excavations carried out on undredged reclamations (Figure 6.6.6). The fine marine deposits of the Hang Hau Formation (see Section 5.8) at the base of the cofferdam act as an impermeable cap, which may have insufficient strength and dead weight to resist uplift forces generated by groundwater pressures at the base of the formation. In such cases, either dewatering beneath the marine deposits or pressure relief holes through the formation, are required. An example is described by Morton & Tsui (1982) regarding the China Resources Building in Wanchai, where dewatering wells were extended through the marine deposits into the underlying completely decomposed granite.

Recharge Wells
In addition to the provision of groundwater cut-offs, the use of recharge wells can be an effective means to limit drawdown of the water table and consolidation settlements beyond excavation footprints (Norcliffe *et al.*, 2002). At the KCRC West Rail Tsuen Wan Station, recharge wells were used to limit drawdown within a confined aquifer consisting of alluvium and completely decomposed granite close to rockhead.

Restriction of Hillside Groundwater Flow
Blockage of the existing groundwater flow paths by cofferdams for deep excavations has been recognized as a potential problem on hillside sites since the time of the catastrophic failure on Po Shan Road in 1972 (Cooper, 1992). A schematic section which illustrates this effect is shown in Figure 6.6.7.

6.7 TUNNELS AND CAVERNS

6.7.1 Introduction

Overview
Tunnels and caverns have been constructed in Hong Kong for many years for the purposes of mining, civil and military defence, storage space, water supply and drainage, sewage treatment, and utility and transport infrastructure. The lack of suitable land for these purposes, combined with the need to service outlying centres of population, industry and commerce, has led to the growth of tunnel and cavern construction in Hong Kong.

Underground construction is capital intensive, with the overall cost, programme and risk of adverse consequences being heavily dependent on the ability to characterise and manage the ground conditions adequately. This is particularly true of tunnels driven by tunnel boring machines (TBMs) where

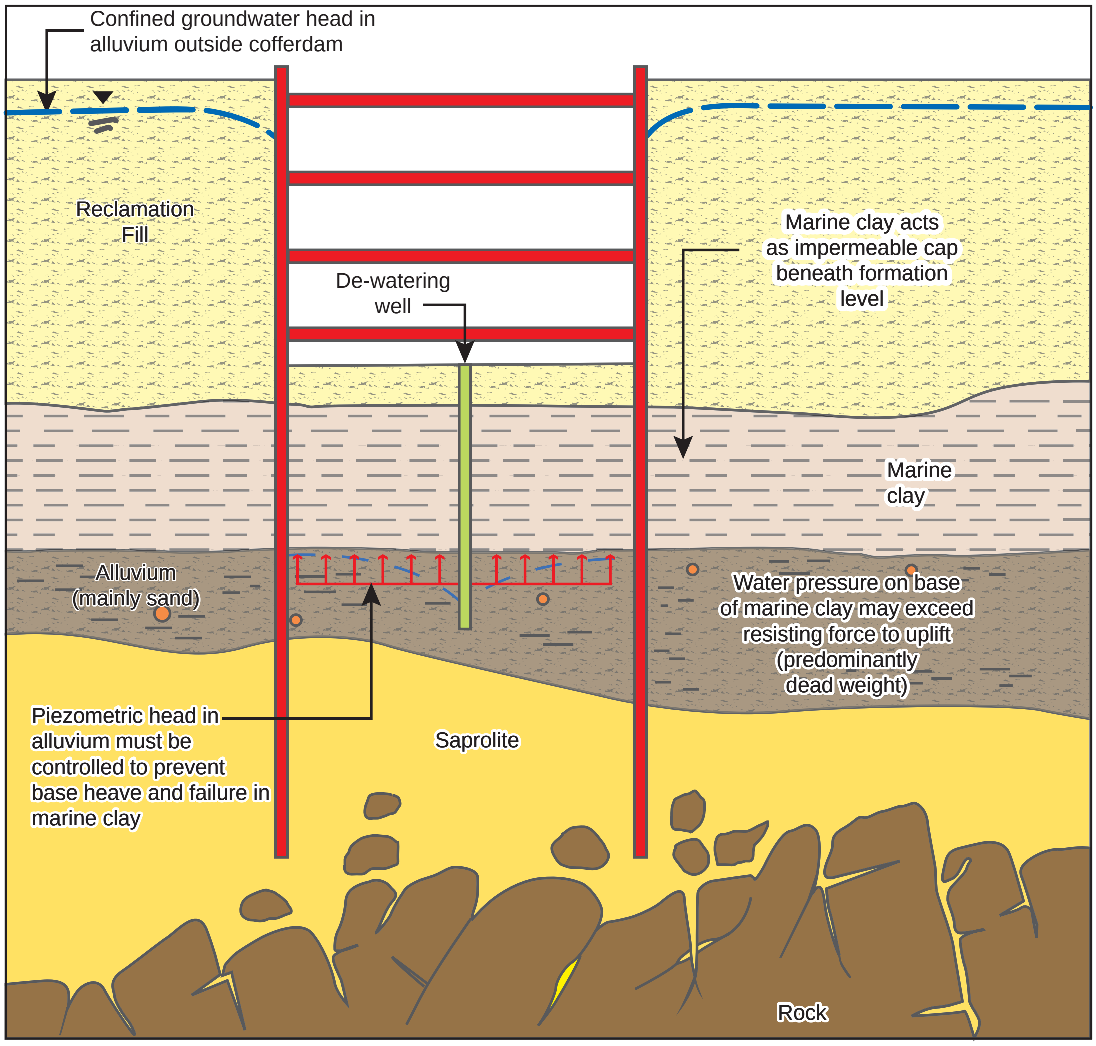


Figure 6.6.6 – Potential base heave in excavation due to confinement of groundwater in alluvium by the overlying marine clay

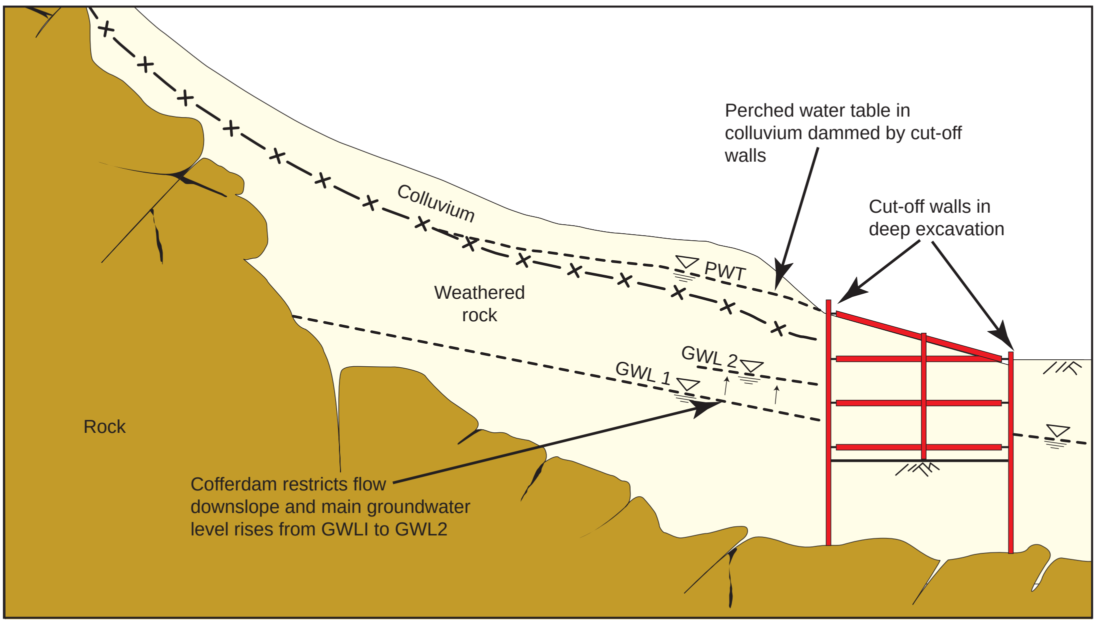


Figure 6.6.7 – Groundwater damming on hillsides due to deep excavations

knowledge of the anticipated operating conditions is required to determine machine suitability well in advance of commencement of the tunnelling works. If unforeseen ground conditions are subsequently encountered which are close to or beyond the limits of the TBM, the limitations on available response strategies imposed by the presence of the TBM in the tunnel may exacerbate the consequences.

Ideally, unforeseen ground conditions should not be encountered during construction or post-construction. However, there are inherent uncertainties in the ground conditions, regardless of the nature and extent of the site investigations. Serious failures or other adverse consequences can occur, especially where the risks are not recognised or are inadequately managed, e.g. GEO (2005a,b) and Pang *et al.* (2006).

For most tunnelling and cavern projects, a number of horizontal and vertical alignments, excavation methods and lining types will need to be considered. These will be influenced by the potential ground conditions (Section 6.7.2) and the perceived levels of geological uncertainty and geotechnical risk. Timely recognition of the ground conditions and the determination of appropriate measures to deal with them are central to the degree of success of all tunnelling projects.

Engineering Geological Issues

Errors in estimating the percentage of tunnel requiring heavy support or the extent of groundwater control measures can result in large differences between anticipated and actual costs and construction programmes. The risk of adverse consequences such as tunnel collapses or excessive surface settlement are also of concern.

Although engineering geological input is essential for all types of tunnelling, the input can vary depending on the stage of the project, the ground conditions and the tunnelling methods employed. Intensive engineering geological input is generally required for:

- rock tunnelling where empirical methods of estimating the engineering properties of the ground are used, and
- mixed face tunnelling conditions where the risk of ground loss due to tunnel collapse and contrasts in excavatibility are major considerations.

Engineering geological input for TBM-driven tunnels in soft ground tends to be more limited, due to:

- the key engineering properties being assessed using conventional geotechnical testing,
- groundwater ingress and ground loss being controlled by the TBM earth or slurry pressure balance method and the primary lining being constructed within the tailskin of the machine, and
- lack of opportunity to observe the ground conditions at the face.

This section concentrates on the engineering geological input required to develop geological and ground models for the following purposes:

- Assessment of geological structures and identification of potentially adverse geological conditions such as weak or water-bearing features related to faults, dykes and deep zones of weathering.
- Assessment of the relevant characteristics of the ground, which may include the use of rock mass classifications to aid selection of excavation methods, temporary support types and permanent lining types.
- Assessment of mixed ground interfaces associated with variable rockhead profiles and corestone-bearing profiles.
- Assessment of the impacts on the local and regional hydrogeology prior to, during and after construction.
- Cavern construction.

Other important engineering geological input includes verification and re-interpretation of the ground conditions during excavation, particularly where an observational method is employed.

GEO (2005a,b) and Pang *et al.* (2006) contain guidelines and references for the planning of site investigations and the management of geotechnical risks related to tunnelling works. Examples of risk assessments for feasibility studies of tunnels and caverns in connection with the Harbour Area Treatment Scheme (HATS) are contained in CDM (2004).

Radon

Radon is a naturally occurring radioactive gas. It is produced in the decay series of both uranium and thorium (Ball *et al.*, 1991). GEO (1992) notes that:

"Radon can be a radiation hazard if concentrations of the gas and its decay products exceed safety limits. In caverns, the concentration of radon can be kept low by ventilation systems, both during construction and operation. Nevertheless, radiation levels should be monitored, particularly after blasting. Until local standards for radon concentrations are established, the concentration should not be allowed to exceed 200 Bq/m^3 as long as there are persons present."

Li & Chan (2004) give a review of statutory radon control in Hong Kong and other countries and report on a field study of radon levels in a tunnel under construction in Hong Kong. They found that radon concentrations were related to geology, groundwater ingress and degree of ventilation. A maximum concentration of over 30,000 Bq/m^3 was recorded during the field study.

6.7.2 Methods of Excavation and Types of Lining

The principal types of tunnels are cut-and-cover (C&C), immersed tube (IMT) and bored tunnel (BT), the latter including machine-excavated and drill-and-blast driven tunnels.

From a geotechnical perspective, the selection of appropriate methods of excavation is influenced by the ground conditions and the need to limit groundwater inflows, ground movements, deformations and vibrations. Typical bored tunnelling techniques for soft ground, mixed face and hard rock conditions are listed in Table 6.7.1.

The type of permanent lining may be dictated by the functional requirements of the tunnel, but also may be influenced by the ground conditions and in particular,

<table>
<tr><th>Soft Ground</th><th colspan="2">Mixed Soil and Rock</th><th>Hard Rock</th></tr>
<tr><td>TBM – earth pressure balance (EPBM) in silts and clays, slurry pressure balance (SPBM) in water-bearing sands and gravels</td><td>Mechanical excavation/hand mining if front of shield under compressed air may be necessary to remove obstructions in mixed-face conditions</td><td>Ground improvement may be necessary for soft or unstable ground</td><td>TBM – open face where groundwater drawdown is not particularly problematic or where groundwater inflow can be cost-effectively controlled by grouting</td></tr>
<tr><td colspan="2" rowspan="3">Mechanical excavation / hand mining with steel arch/shotcrete linings, grouting and forepoling.
Multiple headings may need to be used depending on tunnel dimensions, ground conditions and amount of pre-reinforcement or ground treatment used.</td><td>Mechanical excavation/hand mining under compressed air may be necessary to remove obstructions in mixed-face conditions</td><td>Multi-mode closed/open face EPBM/SPBM where groundwater drawdown problems cannot be cost-effectively alleviated by other means. Also where sections of rock and sections of soil are expected along the same alignment</td></tr>
<tr><td rowspan="3">Ground improvement and multiple headings may be necessary for soft or unstable ground</td><td>Drill & blast for non-circular sections and very hard rock which is difficult to excavate by TBMs or roadheaders</td></tr>
<tr><td>Non-blasting methods in blast vibration-sensitive zones, e.g. hydraulic breakers, expanding chemicals, propellant breakers, rock sawing, etc.</td></tr>
<tr><td colspan="2">Mechanical excavation/hand mining using support shield with grouting and/or compressed air (relatively short tunnel lengths)</td><td>Road-header for medium-sized spans in highly fractured or weak to moderately strong rocks</td></tr>
<tr><td colspan="4">Pipejacking (small diameter)
Back-reamed directional drilling (smallest diameter)</td></tr>
</table>

Notes:

1. Accurate assessment of the ground conditions is of paramount importance to enable selection of the most appropriate tunnelling methods.
2. In hard rock, longer tunnels of small to moderate span tend to favour the use of TBMs.
3. In hard rock or mixed ground, shorter tunnels, large spans or non-circular sections tend to favour drill & blast or mechanical excavation methods.
4. Effective groundwater exclusion for closed-face TBMs is limited by the capacity of the seals in the shield and the segmental linings to withstand the water pressure.
5. Ground treatment methods depend on the ground conditions, tunnelling methods, allowable ground displacement/deformation and risk.
6. Cement or chemical grouts are commonly used, but ground freezing may be appropriate in special circumstances.
7. Many additional factors and specialized methods not listed in this table may need to be considered.

Table 6.7.1 – Summary of typical bored tunnelling methods for soft ground, mixed face and hard rock conditions

hydrogeological and settlement considerations.

Drained linings have the largest impact on hydrogeology and grouting may be required to control inflows in the temporary or permanent condition.

Undrained linings have less impact on hydrogeology, but where the tunnel is constructed in a drained condition, temporary groundwater drawdown may occur. Consequently, it may be necessary to control groundwater inflow by grouting to prevent excessive settlement or drainage of surface water.

6.7.3 Adverse Geological Structures

General

This section provides examples of the occurrence and typical characteristics of adverse geological structures based on experience gained during tunnelling works for the HATS Stage 1 sewage tunnels (CDM, 2004) and the Route 8 Sha Tin Heights highway tunnel. These projects underline the potential for high groundwater inflows in submarine rock tunnels (HATS Stage 1) when compared with the relatively dry conditions typically encountered in tunnels driven above sea level beneath mountainous terrain (Sha Tin Heights).

As noted in Section 3.2, the geology and geological structure shown on the published geological maps of Hong Kong are based on interpretations of limited data and are constrained in detail due to the map scale. These limitations make it essential that adequate engineering geological knowledge and skills are used to develop realistic geological models at an appropriate scale for the proposed works by:

- interpreting the published maps and memoirs to obtain an overall appreciation of the general geology and structure,
- based on that geological knowledge consider the potential engineering implications of the geology on the project,
- collation, interpretation and integration of existing data to refine the geological and ground models (Section 3.2),
- carrying out field mapping and targeted ground investigations to minimise remaining geological and geotechnical uncertainties (Sections 3.3 and 3.4), and
- continuous verification of the geological and ground models based on data obtained during construction.

In areas where tunnelling works have been carried out, as-built construction records may provide valuable information to refine the understanding of the geological structure of the area of interest.

HATS Stage 1 Tunnels

Occurrence of Adverse Geological Structures

A partial layout plan of the HATS Stage 1 tunnels is shown in Figure 6.7.1, This also shows the location of adverse geological structures encountered in the tunnels (based on examination of summaries of as-built records held in GIU) and the inferred faults and photolineaments shown on the published 1:20,000- and 1:100,000-scale geological maps (GCO, 1986b; Sewell *et al.*, 2000). The apparent difference in the location and number of adverse geological structures are primarily due to limitations of the published maps as noted above.

Faults

Faults encountered can be classified into three main categories (CDM, 2004):

(a) Zones of brittle deformation with calcite and quartz infill but limited clay infill: these were characterised by moderately decomposed (Grade III) rock with RQD generally below 40%. Joints were commonly planar and smooth or slickensided but little clay infill was observed. A total length of 870 m (about 4.5%) of the tunnels fell in this category. Where the tunnels were driven under land in this category, only 13% of the tunnel length encountered water inflows >50 litres per minute from probe holes. Where the tunnels were driven beneath the sea in this category, some 74% of the tunnel length encountered inflows >50 litres per minute from probe holes, and 16% encountered inflows >300 litres per minute from probe holes.

(b) Zones of brittle deformation with significant clay infill: these were characterised by moderately decomposed (Grade III) rock with RQD generally below 40%. Joints were commonly planar/smooth or slickensided with appreciable clay infill usually kaolin. A total length of 489 m (about 2.4%) of the tunnels fell in this category. Some 83% of the tunnel length driven under the sea in this category encountered inflows >50 litres per minute from probe holes, with 16% yielding >300 litres per minute from probe holes.

(c) Zones of fault gouge: these were characterised by completely decomposed (Grade V) rock. RQD

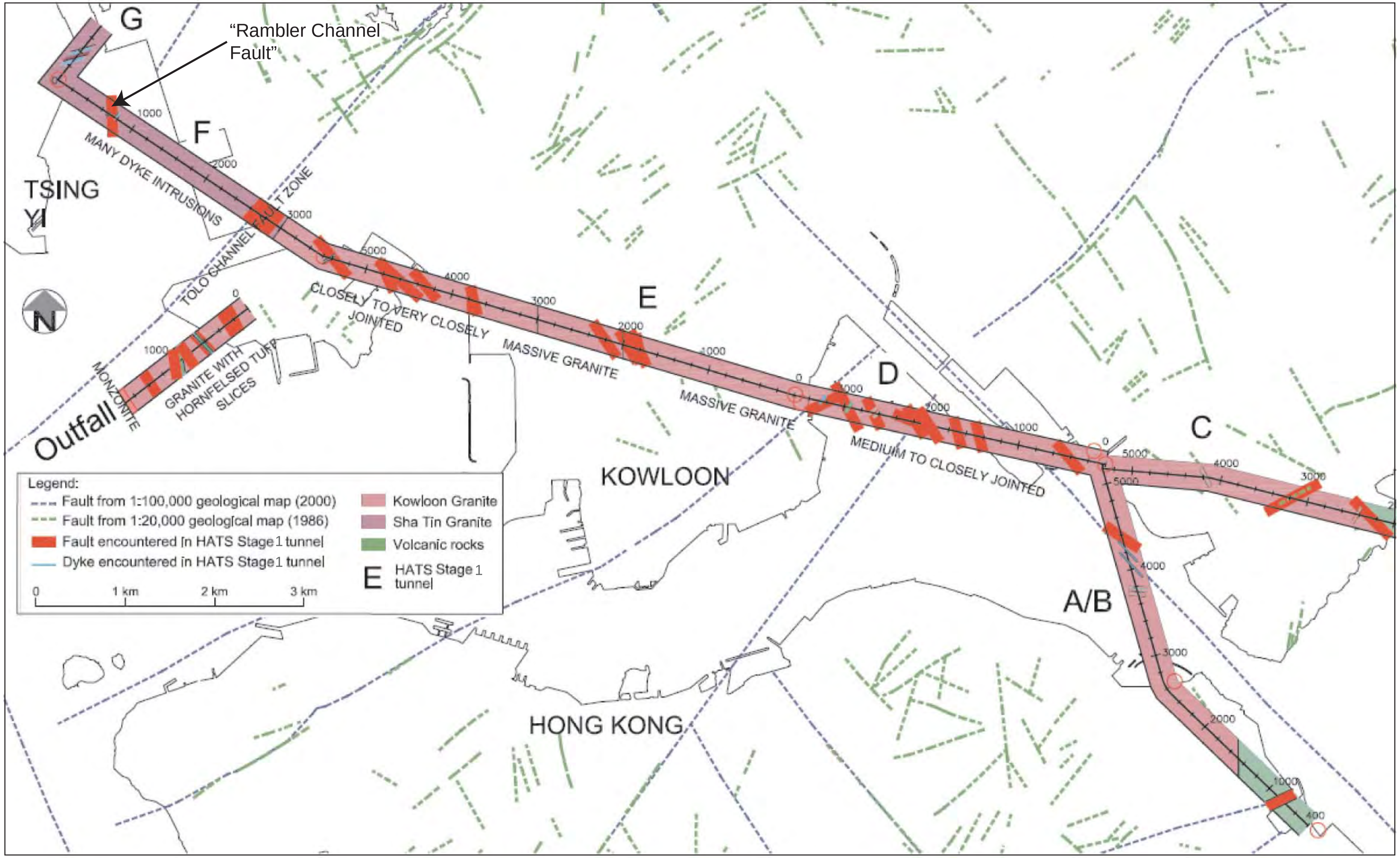
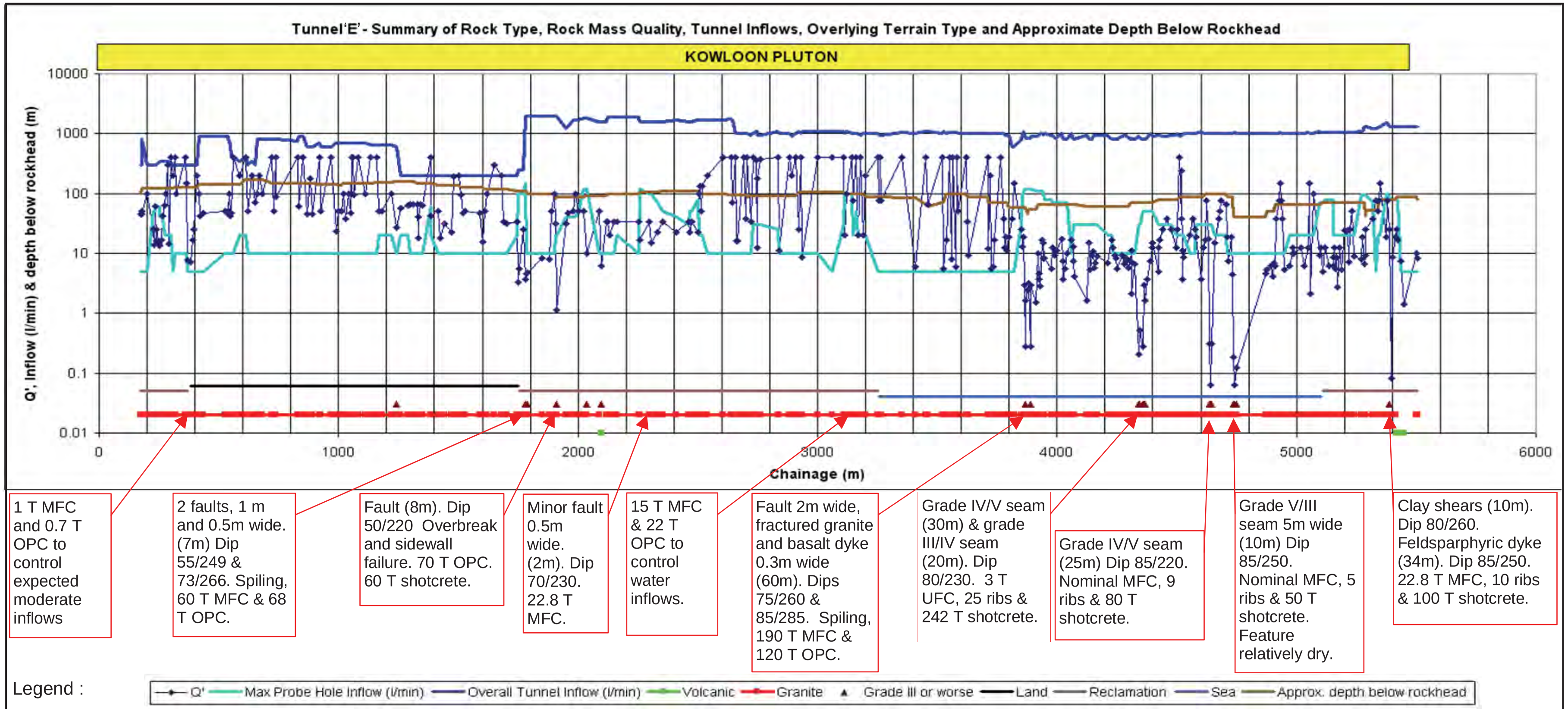


Figure 6.7.1 – Schematic plan of adverse geological structures encountered in HATS Stage 1 tunnels West of Tseung Kwan O (from as-built records) and faults and photolineaments from the 1:20,000-scale (GCO, 1986b) and 1:100,000-scale (Sewell et al., 2000) published geological maps



Notes:
1. Information from checked final record drawings and final reports for HATS Stage 1 held in the Geotechnical Information Unit (GIU) of the Civil Engineering Library.
2. Thickness of geological features in brackets is as-encountered along tunnel chainage.
3. True thickness of geological features is given without brackets where known.
4. OPC indicates ordinary Portland cement grout.
5. MFC indicates microfine cement grout.
6. UFC indicates ultrafine cement grout.
7. T = tonnes.

Figure 6.7.2 – Summary of ground conditions and ground treatment works in HATS Stage 1 Tunnel 'E'

> was generally below 10%. Soil-like conditions were commonly characterised by a firm to stiff clay with sheared lenses of rock. A total length of 106 m (about 0.65%) of the tunnels fell in this category. Only 24% of the tunnel length driven in these zones encountered inflows >50 litres per minute from probe holes, with only 4% >300 litres per minute from probe holes.

Within about 200 m of major faults, joints were commonly more open, resulting in higher groundwater inflows. Peak inflows of >300 litres per minute were occasionally recorded for probe holes where discrete, open joints were intersected in plutonic rocks. In volcanic rocks, peak inflows of >500 litres per minute were recorded, particularly where joints were initially associated with thick kaolin infills which were subsequently washed out by groundwater flow.

Dykes
The dyke rocks encountered included mafic rock, monzonite, aplite, pegmatite and rhyolite.

Mafic dykes were usually less than 2 m thick but were occasionally up to 5 m thick. Sheared zones comprising firm to stiff clay resulting from hydrothermal alteration of the rock were sometimes encountered that softened under the influence of water. These caused some minor collapses and generally required strengthening ahead of the face. The dykes intruded into plutonic rocks often had sharp margins with low water inflows. High groundwater inflows associated with the margins of dykes intruded into tuff were common, resulting in probe hole inflows of 200 to 300 litres per minute.

With the exception of aplite, which was not particularly problematic, the other dyke rocks, particularly the pegmatites and rhyolites, were generally very blocky with fractured margins, commonly resulting in probe hole inflows of 200 to 250 litres per minute.

Zones of Hydrothermal Alteration and Deep Weathering
Hydrothermal alteration and weathering along subvertical joints in plutonic rocks resulted in zones of highly to completely decomposed rock between 0.5 m and 6.0 m thick, which were generally unstable under high water pressure. These led to some minor collapses and generally required strengthening ahead of the face. Although not directly attributed to faulting, the presence of such zones at depths of about 100 m below rockhead gave rise to tunnelling difficulties similar to those associated with decomposed fault zones.

Tunnel 'E'
Tunnel 'E' was driven through the centre of the Kowloon Pluton, where tunnelling conditions overall were generally better than in the other HATS Stage I tunnels. However, a number of faults, dykes and weathered seams were encountered that gave rise to isolated difficulties, requiring pre-grouting to control groundwater inflows and occasional heavy support to maintain stability of the face and sidewalls.

Figure 6.7.2 provides a summary of the adverse geological structures, rock mass quality, tunnel inflows, overlying terrain type, depth below rockhead, ground treatment and support measures carried out to control groundwater inflows and maintain stability.

Tunnel 'F'
Tunnel 'F' was driven largely through the Sha Tin granite which is intruded by many predominantly rhyolitic dykes of the East Lantau dyke swarm. These intrusions, and the presence of numerous faults, including the 20 m wide "Rambler Channel Fault" (Section 4.2) and a 280 m wide zone affected by the Tolo Channel Fault, made tunnelling conditions much worse than in the other HATS Stage 1 tunnels.

The generally adverse conditions resulted in local collapses, high rates of groundwater ingress and major delays. For example, it took a total of 10 months to tunnel through the zone affected by the Tolo Channel Fault. The total time required for drilling and grouting represented about 75% of the total tunnelling time. Table 6.7.2 shows average tunnelling rates and days lost to bad ground and control of excessive inflows of groundwater for Tunnel 'F' compared to other Stage I tunnels that were excavated and supported using similar methods.

Route 8 Sha Tin Heights Tunnel
The location and nature of the Tolo Channel Fault zone was also a major engineering geological consideration when planning the alignment and ground investigations for the Route 8 Sha Tin Heights Tunnel.

A plan of the tunnel is shown in Figure 6.7.3, along with the locations of inferred faults and

Item	Tunnel F - Open TBM Drive with no segmental temporary support	Other Open TBM Drives with no segmental temporary support
Total length (m)	3098	9704
Peak tunnelling rates (m/day)	30	30
Days lost due to control of excessive groundwater	300	0
Days lost due to bad ground	238.6	14.7
Metres/day overall	2.9	12.2

Table 6.7.2 – Comparison of Tunnel 'F' with other similarly excavated tunnel drives for HATS Stage 1 (after CDM, 2004)

photolineaments associated with the Tolo Channel Fault zone, obtained from the published 1:20,000 and 1:100,000 geological maps (GCO, 1986a; Sewell *et al.*, 2000). Based on this information, the main Tolo Channel fault was expected either to pass through the tunnel alignment or to lie to the southeast of the alignment.

Subsequent detailed API suggested that the main NE-trending fault zone would probably lie to the SE of the tunnel, with subsidiary, mostly NW-trending faults striking across the tunnel alignment (Figure 6.7.3). The locations and orientations of crushed or weak, weathered zones with clay in fill identified from the

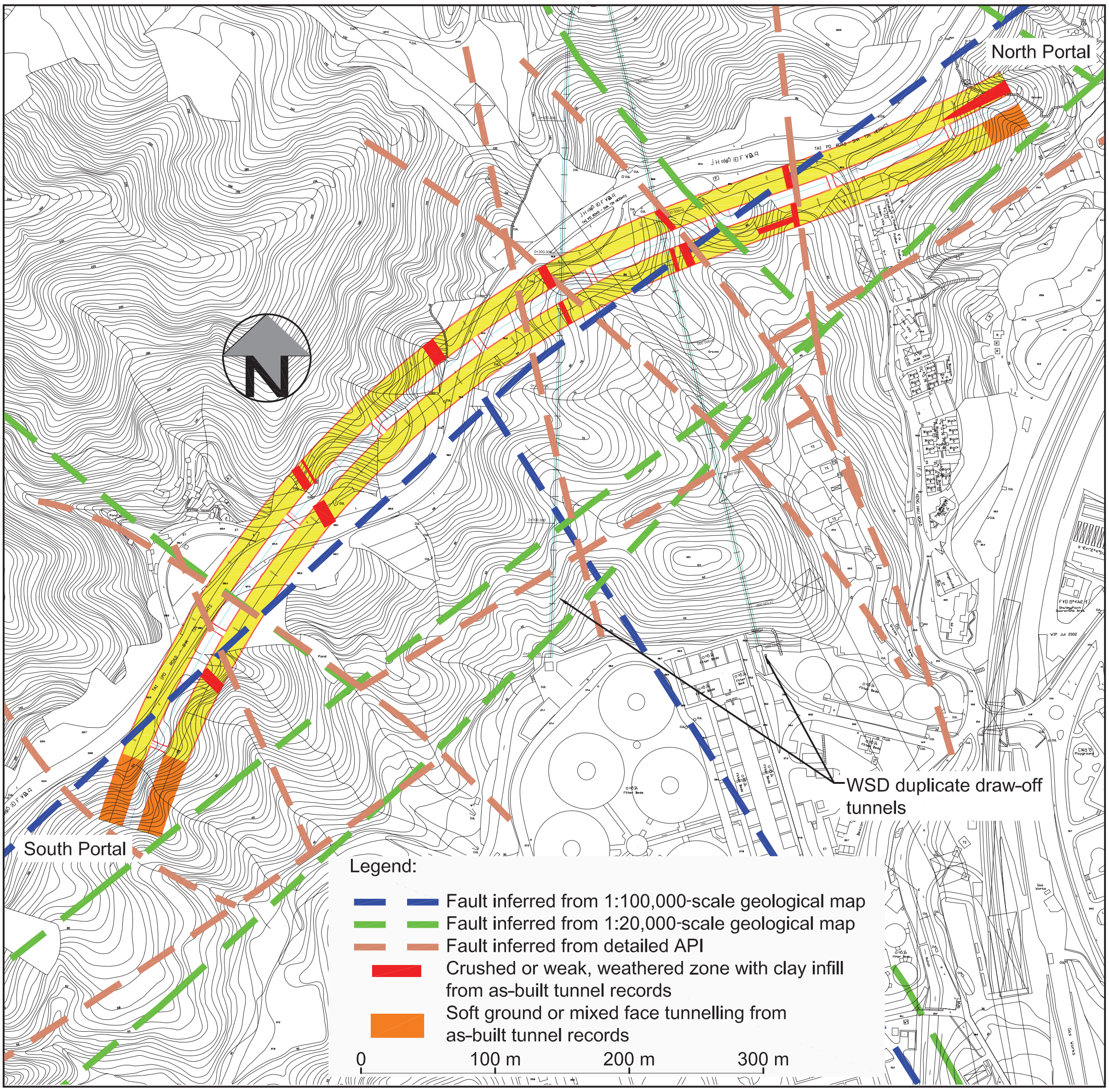


Figure 6.7.3 – Layout plan of geological structures encountered in the Sha Tin Heights tunnel and faults and photolineaments inferred from detailed API and the 1:20,000-scale (GCO, 1986a) and 1:100,000-scale (Sewell et al., 2000) published geological maps

as-built tunnel records are also shown in Figure 6.7.3. This indicates that the faulting pattern inferred from the detailed API provided a more realistic depiction of the distribution and orientation of weak zones actually encountered during construction.

The subsequent ground investigations targeted the potentially adverse features, and two cored holes about 500 m in length were drilled along the alignment from the north and south portal areas using directional drilling techniques to better understand the ground conditions (Section 6.7.4).

In contrast to the HATS Stage 1 tunnels, only minor seepage was associated with the weak zones encountered in the Sha Tin Heights tunnel, probably due to the tunnel being situated in rock at relatively shallow depth, well above sea level.

6.7.4 Assessment of Rock Mass Quality

General

At the pre-construction stage, reliable assessments of potentially adverse geological structures and rock mass quality are required to provide a realistic basis for estimating tunnelling feasibility, support requirements, groundwater ingress, costs and construction programmes. Technical guidelines on investigation methods for tunnelling works are given in GEO (2005a).

Re-assessments of engineering geology and rock mass quality are also required during construction to validate the design and to determine temporary and permanent support requirements, normally using an observational method (e.g. Peck, 1969; Muir Wood, 1990; Powderham, 1994; Nicholson *et al.* 1999; GEO, 2005b) in conjunction with a rock mass classification system.

Such observational methods, incorporating an intensive ground response monitoring programme with reference to the observed and as predicted ground conditions, have been used during the construction of large span caverns (see the Tai Koo Shing Cavern example in Section 6.7.7), tunnels in weak and/or potentially over-stressed rock masses (see the Tai Lam Tunnel Sham Tseng Fault example below) and tunnels in completely to highly decomposed rocks (Endicott *et al.*, 2000). In all these cases, detailed engineering geological mapping during excavation was required to relate the observed performance of the excavation to the observed ground conditions. Such mapping enables forward planning of excavation and support strategies based on a comparison of the existing conditions with the expected conditions ahead of the face (characterised from engineering geological assessments based on site investigation and probe-hole data).

Rock Mass Classification Systems

Rock mass classification systems are commonly used to characterise the ground conditions for rock tunnelling works in Hong Kong. An outline of these systems and key references on their use are contained in Section 3.5.4. Typical uses include the assessment of systematic support requirements, rates of tunnelling and prediction of zones of ingress of groundwater.

One of the main advantages of such classifications is that the use of only a few parameters renders them potentially applicable to a wide range of tunnelling situations.

However, as noted in Section 3.5, whichever system is used to characterise the ground conditions, considerable knowledge and experience is required to translate the output into support strategies or engineering parameters that are appropriate for site specific conditions. Furthermore, other factors which may affect the engineering performance of the rock mass, such as differences in relative joint orientation, discontinuity persistence, jointing frequency in different directions, rock types and geological structural regimes, also need to be considered.

Consistency in rock mass classification assessments is important for the purposes of establishing useful databases and for providing reliable estimates of ground conditions that are commensurate with the assumptions made in the detailed design. Although detailed references and examples on the use of rock mass classification systems are readily available (Section 3.5), improvements in consistency of tunnel logging on site are sometimes necessary. This can be facilitated by the use of a site specific manual which provides clear definitions of how the design assumptions relate to the rock mass classification system and how each parameter is assessed.

Ground Investigation Strategies and Assessment Methods

Shallow Tunnels with Relatively Easy Access for Investigation

For relatively shallow (<200 m) land-based tunnels or underground excavations with compact layouts, the cost of direct investigation works is relatively modest, and economical investigations can be designed to provide a reasonable degree of cofidence with regard to the ground conditions.

Investigations typically include the use of relatively closely-spaced vertical and inclined drillholes and sub-horizontal holes carried out using directional drilling techniques. In the case of caverns or sections of tunnel with relatively easy access from the surface, pilot tunnels can be constructed. These may comprise top or bottom headings carried out using an observational approach to provide information on the ground conditions and ground responses for the purposes of re fining the design for the fully excavated span (e.g. the Tai Koo Shing Cavern in Section 6.7.7).

Taking the Route 8 Sha Tin Heights Tunnel as an example, a relatively lar ge number of vertical and inclined drillholes were carried out to investigate potentially adverse geological structures (Section 6.7.3). These holes also provided information on the general rock mass conditions along the alignment, which lies at relatively shallow depth.

The relatively short tunnel (about 800 m) also made it practicable to carry out directional drilling from the north and south portal areas to provide a near-continuous record of the rock mass along the length of the tunnel. However, it was recognised that the small diameter, horizontal cored holes do not realistically reflect the rock mass conditions which would be assessed in the 18 m span tunnels. Thus simply using the results of the directionally drilled holes could lead to overestimates or underestimates of overall rock mass quality.

It was also recognised that the steeply sloping hillside would result in large differences in tunnel depth across the span of the twin tunnels (Figure 6.7.3) which might give rise to potential variations in the rock mass conditions. All drillhole cores were therefore characterised and the data were then processed to provide estimates of the percentage length of each Q-system temporary support class in broad zones based on depth below rockhead. These estimates were then applied to each tunnel alignment, with allowance made for downgrading rock mass quality in the vicinity of known faults and weathered zones in addition to using specific drillhole information.

A comparison of the pre-construction estimates and as-built records of the percentages of Q-value ranges is shown in Table 6.7.3. Depending on the span of the tunnel and the approaches adopted for temporary support design, a Q-value of less than about 0.3 generaly indicates heavy systematic support, while a Q-value greater than 4.0 indicates that either no or only light systematic support would be required.

Total Length of Bored Tunnel (1,609 m)			
Q-value range	< 0.3	0.3 – 4.0	> 4.0
Pre-construction estimate	14.7%	60.6%	24.7%
As-built records	7.5%	43%	49.5%

Table 6.7.3 – Sha Tin Heights Tunnel Q-value ranges estimated at the pre-construction stage and from as-built records

The comparison indicates that the pre-construction estimates were conservative. This is probably due to three main factors:

- over-estimation of the extent of in fluence of the major geological structures,
- higher than expected rockhead in the vicinity of the portals, and
- over-application of high SRF values in the assessment of Q-values from the drillhole logs to allow for possible loose rock mass conditions at shallow depths. However, such occurrences were rarely encountered during excavation.

Although the pre-construction estimates erred on the conservative side, it should be noted that it is very difficult to accurately de fine rockhead unless very closely spaced drillholes are carried outAlso, the risk of encountering loose rock mass conditions increases with the shallowness of the tunnel alignment.

Comparison of the as-built Q-value ranges with those shown in Table 6.7.4 for the KCRC West Rail Tai Lam Tunnel (up to 500 m in depth) indicates that the tunnelling conditions were much worse than those generally encountered at depth in the plutonic and volcanic rocks west of the Sha Tin granite, e.g.

the percentage of tunnel length where the Q-values were greater than 4.0 was about 50% for the Sha Tin Heights Tunnel, as opposed to about 85% for the Tai Lam Tunnel. This may also be due to the close proximity of the Tolo Channel Fault zone to the Sha Tin Heights Tunnel.

Tunnels at Depth with Difficult Access for Investigation

For tunnels situated at great depth (>200 m) beneath mountainous terrain or situated beneath the sea bed, the cost of direct investigation works is relatively high. The main ground investigations commonly comprise vertical and inclined drillholes at widely spaced intervals (commonly about 500 m) combined with relatively cheap geophysical investigations.

In such cases, the risk of encountering unforeseen, adverse ground conditions along the lengths of tunnel where no direct information is available may be high. This is particularly the case in areas which may be affected by adverse geological structures (Section 6.7.3), relatively highly fractured and variable non-plutonic rocks, or where little previous experience and data are available (e.g HATS Stage 1 tunnels).

One example of a relatively costly investigation method for a deep tunnel is the use of an almost full length pilot tunnel for the Aberdeen Tunnel (Twist and Tonge, 1979). This was carried out due to concerns over lack of previous experience of deep tunnelling works in Hong Kong, combined with uncertainties regarding the contact zones between granite, deeply weathered monzonite and volcanic rocks near the south portal.

The pilot tunnel provided high quality information for pricing purposes. Inability to drive the pilot tunnel from the south portal highlighted the adverse ground and groundwater conditions. This led to adoption of a remeasurement contract based on a schedule of rates for equitable reimbursement of tunnelling costs in this difficult section of the tunnel.

The KCRC DB350 Tai Lam Tunnel provides an example of the assessment of overall tunnelling conditions for a very long and deep tunnel in rock based on conventional site investigation methods. A location plan of the 5.5 km long Tai Lam Tunnel is shown in Figure 6.7.4. Longitudinal sections which illustrate the difference between the anticipated and

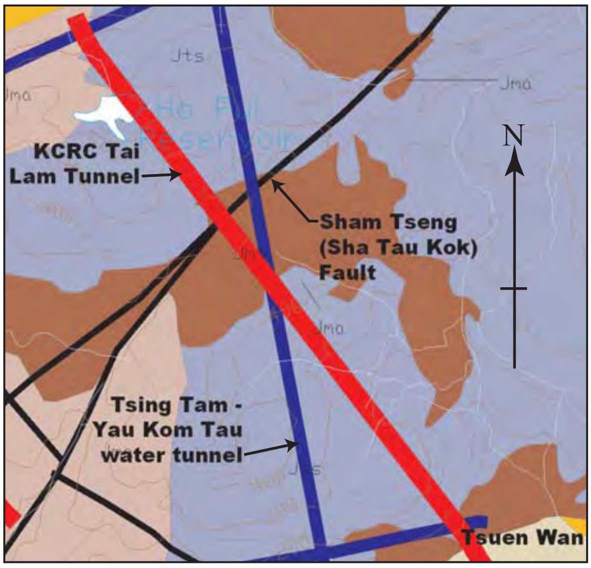


Figure 6.7.4 – DB350 Tai Lam Tunnel location plan

actual geology for the northern section of the tunnel are shown in Figure 6.7.5.

Information at the tender stage primarily consisted of:

- data from drillholes carried out at intervals between 500 m and 1000 m (closer spacing near portals),
- a continuous as-built log of the Tsing Tam – Yau Kom Tau water tunnel (see Figure 6.7.6), and
- the pre-tender Geotechnical Basis of Design Report (GBDR) which contained preliminary estimates of Q-values marked on a longitudinal geological section of the proposed tunnel alignment.

During the tender stage for the design-and-build contract, additional API and structural geological assessments were carried out using similar methods to those outlined for the Sha Tin Heights tunnel in Sections 6.7.3 and 6.7.4. During this process, some of the more minor faults were eliminated from individual consideration and reassessment of the Sham Tseng Fault zone reduced the estimate of its width at tunnel level to about half the width shown in the pre-tender reports (see the upper section in Figure 6.7.5).

A re-assessment of the likely percentages of Q-value ranges was also carried out. This was based on project specific drillhole data, combined with a large database derived from previous experience of similar geological conditions during construction of other tunnels in Hong Kong. The methodology used to assess the percentage of Q-value ranges along the tunnel alignment was essentially the same as that previously described for the Sha Tin Heights tunnel.

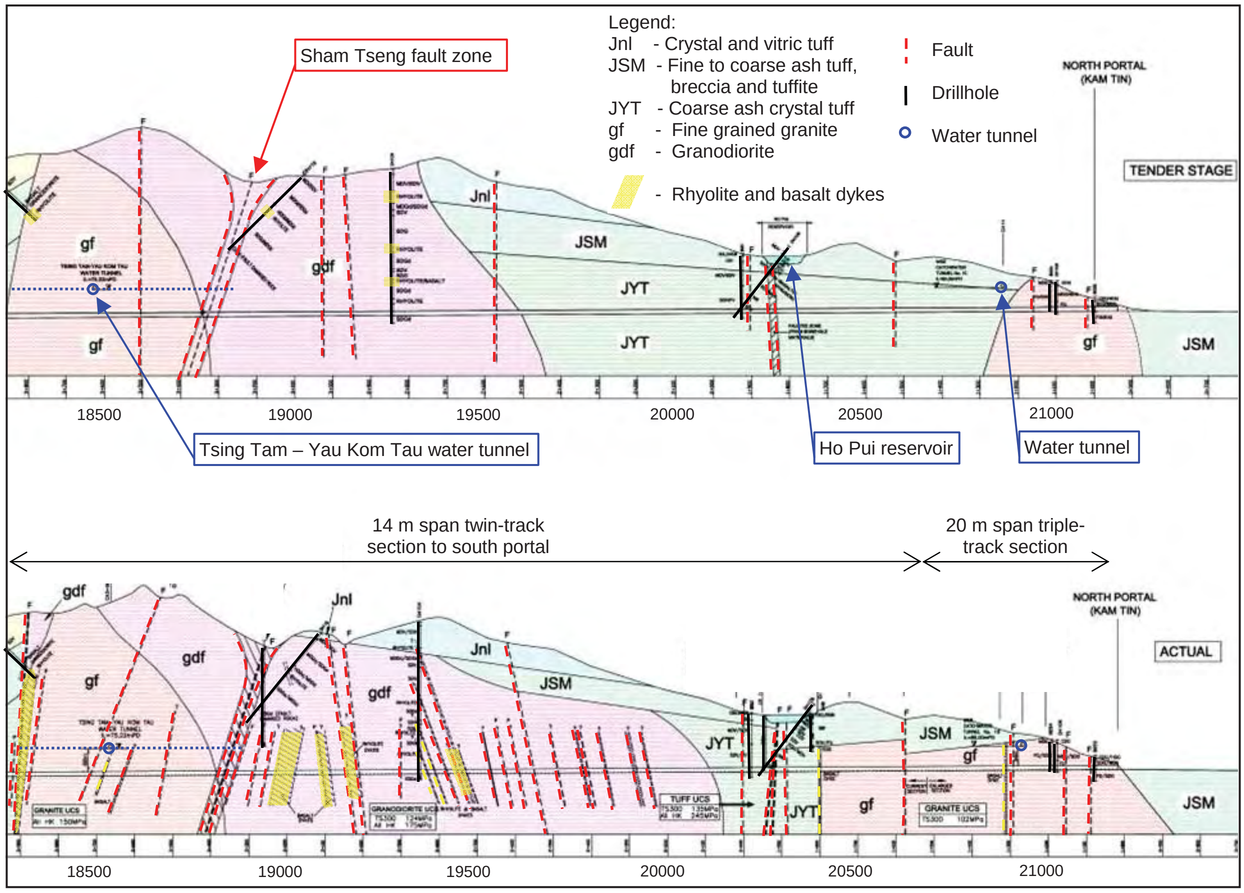


Figure 6.7.5 – West Rail Tai Lam Tunnel (northern section) – comparison of the tender design and as-built geological profiles (based on records provided by Nishimatsu-Dragages Joint Venture)

For the northern section of the tunnel, it was found that coarse ash crystal tuff was less prevalent, and rhyolitic and mafic dykes and moderate faulting were more common than expected at the tender stage. These differences were primarily due to the difficulty of accurately extrapolating intrusive contacts to a depth of between 200 m and 500 m below ground level in a structurally complex area, cross-cut with NW- and NE-trending faults.

Comparisons of the percentage of ranges of Q-values between the pre-tender, tender and construction stages are shown in Table 6.7.4. For the 14 m and 20 m span tunnels, a Q-value less than 0.4 generally indicates the need for heavy systematic temporary support, while a Q-value greater than 4.0 generally indicates that either no support or only light systematic temporary support would be required. Table 6.7.4 shows differences of less than 5% between the tender and as-built estimates for the overall tunnel length.

For the southern section, the rock mass quality in the predominantly volcanic rock was noticeably better than that expected at the tender stage. For the northern section, the rock mass quality in the granitic rocks, intruded by mafic and rhyolitic dykes, was marginally worse than that expected at the tender stage. The length of tunnel that was affected by the Sham Tseng Fault zone proved to be shorter than was expected.

While this example illustrates the difficulty in extrapolating intrusive geological contacts at depth over long distances and accounting for local differences in rock mass quality, it also demonstrates the value of engineering geological input when preparing competitive design-and-build tender bids.

The use of directional drilling methods and further developments to recover core over long distances may result in the method being used more frequently for investigation of long tunnels with difficult access. Cores have been successfully recovered over a length of about 800 m (GEO, 2005a), and open hole probing has been carried out over a distance of about 1200 m for the Route 8 Eagle's Nest tunnel.

Total Length of Bored Tunnel (5,500 m)			
Q-value range	**< 0.4**	**0.4 – 4.0**	**> 4.0**
Pre-Tender	13.7%	18.4%	67.9%
Tender	2.5%	19.8%	77.7%
As-built	0.9%	16.3%	82.8%
Southern Section (2,580 m)			
Q-value range	**< 0.4**	**0.4 – 4.0**	**> 4.0**
Pre-Tender	10.7%	20.3%	69.0%
Tender	1.7%	24.5%	73.8%
As-built	0.3%	9.9%	89.8%
Northern Section (2,920 m)			
Q-value range	**< 0.4**	**0.4 – 4.0**	**> 4.0**
Pre-Tender	16.2%	16.8%	67%
Tender	3.2%	15.6%	81.2%
As-built	1.3%	21.9%	76.8%

Table 6.7.4 – West Rail Tai Lam Tunnel – comparison of pre-tender, tender and as-built estimates of rock mass Q-values

KCRC DB350 Tai Lam Tunnel – Sham Tseng Fault

This case study describes how geological, ground and design models were developed using an observational method where a major fault zone was excavated at a depth of about 400 m in a 14 m-span tunnel (see Figures 6.7.4 and 6.7.5). The example illustrates the translation of engineering geological data available at the tender stage into a series of design models and compares the as-built conditions with the range of conditions assumed for design.

Pre-tender Information

Information about the ground included site investigation data and records of construction of nearby tunnels. The log of the water tunnel in the vicinity of the Sham Tseng Fault zone is shown in Figure 6.7.6. This indicated that the zone was composed of several faults, with the largest being about 45 m thick. The zone was classified as major and described as '*laminar, brecciated, crushed, fractured granodiorite, weak, moderately and highly weathered'* with a secondary schistosity. The overbreak for this zone was recorded as '*moderate to large (400 – 800 mm)*', and near its southern edge, high water inflows and overbreak of '*2 - 5m+*' were experienced.

The log indicated that very adverse tunnelling conditions might be encountered when passing through the fault zone, where the much wider railway tunnel would probably need to be driven using

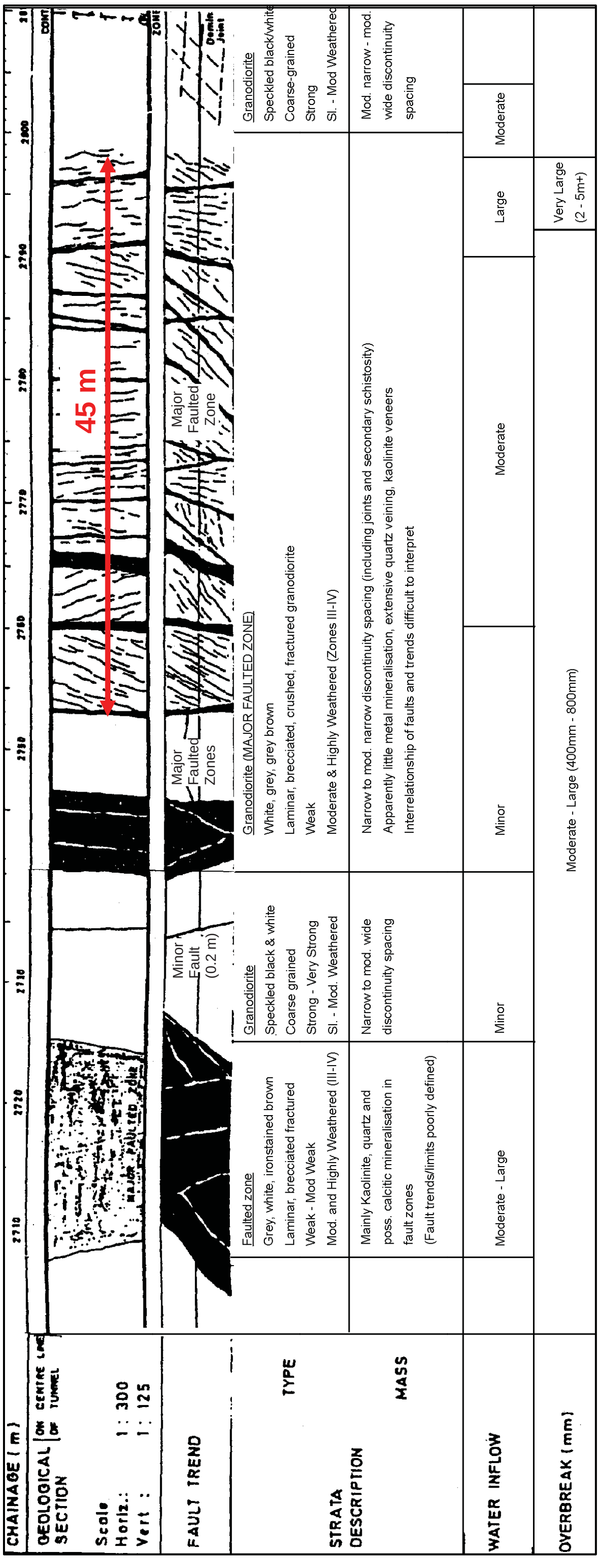


Figure 6.7.6 – Water tunnel log

incremental excavation and support techniques.

Factors taken into account during the tender design and initial risk assessment included:

- The geological memoir (Langford *et al.*, 1989) suggests that the fault is affected by both brittle and ductile movements, and the 1:20,000-scale geological map shows sub-parallel zones of metamorphism.

- The metamorphic zones reflect the development of foliation fabrics which are often silicified by hydrothermal fluids.
- Such fault zones when encountered at a depth of 400 m within the core of a mountain are unlikely to be significantly affected by weathering.
- It was considered likely that the fault would be composed of zones of foliated, slightly to moderately decomposed rock (reflecting ductile deformation) containing narrow zones of fault gouge and breccia (reflecting brittle deformation).

In the light of these factors, it was considered that the 'black' zones shown on the water tunnel log corresponded to altered fault gouge and breccia, and that the 45 m thick zone with thin wavy black lines shown on the log may represent slightly to moderately decomposed foliated rock containing very thin zones of weaker material. This interpretation implied that the descriptions shown on the water tunnel log had been made by treating the two different types of material as one, and that the material strength description '*weak*' may have been influenced by the foliation of the rock mass.

Detailed Design

In order to confirm the ground conditions, an additional 450 m-long inclined drillhole to intersect the fault zone was considered. However, factors which detracted from this option included:

- The drillhole might have to pass through the 60 m exclusion zone around the water tunnel.
- There was uncertainty regarding the dip of the fault and consequently the location of its intersection

45 m THICK GRANODIORITE FAULT ZONE		**Q-SYSTEM Interpretation**				**Notes**	
WSD TUNNEL DESCRIPTION (3.5m dia. tunnel)	**Param.**	**Worst**	**Poor**	**Typical**	**Best**	**Poor**	**Best**
Decomposition Grade "Moderate and Highly Weathered (Zones III-IV)" It is not known whether Grade IV is confined to seams or not - although the "and" in the weathering description tends to favour Grade III material with highly weathered seams.	Grade =	III-IV	III-IV	Close to Grade III	III	Grade III-IV With Grade IV Seams	Grade III With Grade IV Seams
Strength "Weak" - but fracturing and Grade IV seams have probably influenced description. **Note: This assumption is very critical!**	Intact Unconf. Comp. Strength (MPa) =	8.00	8.00	12.00	16.00	Weak to Moderately Weak	Moderately Weak to Moderately Strong
Discontinuity Spacing "Laminar, brecciated, crushed, fractured. Narrow to moderately narrow discontinuity spacing (including secondary schistosity)".	RQD =	10.00	10.00	10.00	10.00	RQD = 10	RQD = 10
Joint Sets / Structure Depends on how rock mass description is viewed. Crushing & brecciation may be confined to seams and the seams and secondary schistosity which will strike directly across tunnel may blank out all other sets. Therefore may be few effective sets with orientation of shears most favourable for stability.	Jn =	15.00	15.00	10.50	6.00	Brecciation, crushing and fracturing assumed to dominate "sugar cube"	Brecciation, crushing and fracturing assumed to be confined to seams normal to to tunnel "Two sets plus random"
Joint Roughness No joint roughness description available. However, assumed to be "no rock wall contact when sheared" to "slickensided undulating".	Jr =	1.50	1.50	1.25	1.00	"rock wall cont. before 10cm shear" "slickensided undulating"	"no rock wall contact when sheared"
Joint Infilling Description limited to "extensive quartz veining, kaolinite veneers". Range interpreted to vary between "zones or bands of disinegrated or crushed rock and clay" and "softening clay infillings".	Ja =	8.00	6.00	5.00	4.00	"Softening clay mineral fillings" but 0.75* Ja to allow for favourable orientation of shears	"Bands of crushed rock and clay" but 0.5* Ja to allow for favourable orientation of shears
(RQD*Jr)/(Jn*Ja)	**Q' =**	**0.13**	**0.17**	**0.24**	**0.42**		
"Geological Strength Index"	**GSI =**	**25**	**28**	**31**	**36**	Calculated from Q'	Calculated from Q'
Intact mi Value Discontinuity spacing indicates material is better than "Phyllite" (mi = 10). For Granodiorite, the minimum mi values for "normal" Grade III-IV and Grade III may be 22 and 25 respectively. For worst Grade III-IV a value of 16 may be appropriate.	**m_i =**	**16.0**	**22.0**	**23.5**	**25.0**	Grade III-IV	50% Grade III-IV to 100% Grade III
Poisson's Ratio based on laboratory testing of weathered intact HK rock specimens.	ν =	**0.35**	**0.35**	**0.33**	**0.32**	-	-

Table 6.7.5 – Geomechanics interpretation of the water tunnel log

with the tunnel. The geological memoir (Langford *et al.*, 1989) indicates that the Sha Tau Kok Fault dips steeply to the NW, while the GBDR geological section indicates a SE dip direction.

- The recovery of the fault zone material at a depth of 400 m could not be guaranteed, as indicated by the failure of a previous drillhole to recover any core.

The design and construction strategy adopted was to develop the initial geological and ground models based on interpretations of the existing information from the water tunnel log, assuming a range of conditions varying from a 'worst-case' interpretation to one based largely on engineering geological knowledge and the pictorial depiction of the fault zone on the log. The actual ground conditions would then be confirmed by horizontal core drilling ahead of the face as the tunnel approached the zone, and the new information used to refine the ground and design models (i.e. an observational method).

In view of the time required for design, a series of design models for a range of ground conditions were worked out in advance. This allowed sufficient time for the final design to be detailed when the actual conditions became known and for the contractor to have appropriate equipment and construction materials ready on site.

The degree of uncertainty of the actual ground conditions was reflected in the range of possible interpretations of the water tunnel log descriptions and the initial range of ground model parameters derived using the Q-system, Geological Strength Index (GSI) and the resulting Hoek-Brown parameters (see Table 6.7.5).

After further adjustments to add a 'Best-Plus' category for the fault zone, assuming a higher compressive strength of 22 MPa for the fault zone material, and the addition of a 'Type 1 Ground' assuming moderately to slightly decomposed rock with thin, discrete faults, the design model parameters for sensitivity analysis were finalised using the principles outlined in Sections 3.4 and 3.5 (see Table 6.7.6).

Simple 'characteristic line' sensitivity analyses were initially conducted for both the water tunnel and the KCRC tunnel. As can be seen from Section D of Table 6.7.6, the estimated total deformations lie within a plausible range when compared to the overbreak estimates shown on the water tunnel log. However, applying the same parameters to the much larger span KCRC tunnel gives very large deformations indicative of very challenging tunnelling conditions (see Section E of Table 6.7.6).

Figure 6.7.7 shows the comparative range of difficulty in tunnelling that might be expected for the range of

	Range for Main Fault Zone (Type 3 Ground)				Northern Generally Faulted Mass (Type 1 Ground)	General Granite Rock Mass
	Worst	Typical	Best	Best-Plus		
A - Modelling Parameters						
Deformation Modulus (Mpa) =	682	1166	1799	2111	4143	30400
Poisson's Ratio =	0.35	0.33	0.32	0.32	0.30	0.25
Intact U.C.S. (MPa) =	8	12	16	22	50.00	105.00
m (peak) =	1.53	2.01	2.55	2.55	3.25	10
s (peak)=	0.00025	0.00047	0.00083	0.00083	0.0014	0.033
Angle of dilation =	0	0.5	1	1.5	2	4
m (residual) =	1.46	1.85	2.26	2.21	2.46	3.4
s (residual) =	0	0.00037	0.00056	0.00053	0.0006	0.001
B - Rock Mass Strength						
Mass U.C.S. (MPa) =	0.567	0.984	1.494	2.054	5.33	25.95
Peak Shear Strength (MPa) =	0.7	1.2	1.74	2.4	5.94	19.7
Residual shear Strength (MPa) =	0.54	1.14	1.64	2.24	5.36	13.9
C - Strength : Insitu stress Relationships						
Mass U.C.S. / Insitu Stress =	0.06	0.10	0.12	0.16	0.42	2.03
Ns = Insitu Stress / Peak Shear Strength =	14.63	8.53	7.36	5.33	2.15	0.65
D - Estimated Behaviour of WSD Tunnel from characteristic line calculations (330m depth)						
Estimated Mass Behaviour of Face (WSD Tunnel) =	Unstable	Unstable	Marginally Unstable	Stable	Elastic	Elastic
Estimated Unsupported Total Deformation (WSD Tunnel) =	200 mm	75 mm	60 mm	43 mm	11 mm	Negligible
E - Estimated Behaviour of KCRC Tunnel from characteristic line calculations (400m depth)						
Estimated Mass Behaviour of Face (KCRC Tunnel) =	Unstable	Unstable	Unstable	Marginally Stable	Elastic	Elastic
Estimated Unsupported Total Deformation (KCRC Tunnel) =	1.3 m	450 mm	330 mm	260 mm	67 mm	<5 mm

Table 6.7.6 – Initial design parameter range and sensitivity analysis results

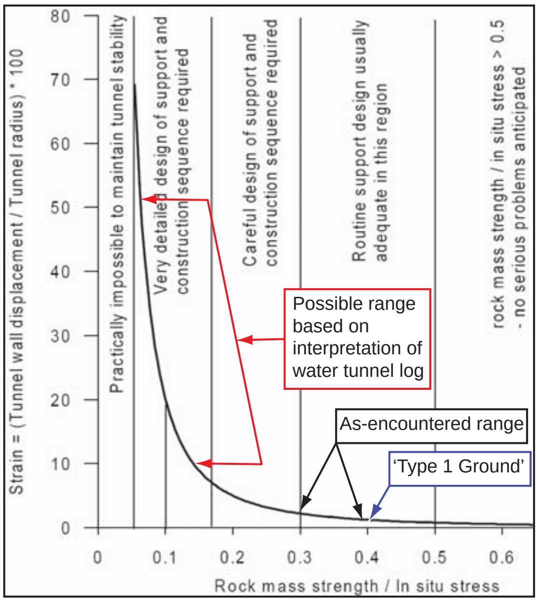


Figure 6.7.7 – Graphical representation of relative tunnelling difficulty

initial design parameters adopted (see strength/*in situ* stress ratios in Section C of Table 6.7.6).

The main challenges were:
- Supporting the tunnel when a large amount of stress (virgin stress = 10.4 MPa) may be transferred to the temporary supports if they are installed too close to the face.
- Ensuring a stable face – usually requiring support close to the face (counter to above).

Support strategies considered were:
- Full face excavation and reinforcement of the face with dowels and shotcrete to control stability while allowing stress relaxation to take place before the temporary support takes-up the remaining load (see Figures 6.7.8 and 6.7.9).
- The use of small headings to control face stability (see Figure 6.7.10).

The reinforcement and support strategies were modelled using finite element analyses to obtain an appreciation of the suitability of each method over the full range of assumed ground conditions. The most successful method indicated was the use of full face excavation with face reinforcement and forepoling. This method also provided maximum flexibility for the contractor.

Construction

Horizontal coring was carried out as the fault zone was approached. Conditions were much better than indicated by the water tunnel log. Probing ahead established that the fault zone was composed of discrete, narrow faults separated by zones of relatively competent rock. The largest individual fault was only 8 m thick, so a 3-D 'arching'effect was expected with the loads being transferred to the more competent rock on either side of the faults. Consequently, forepoles,

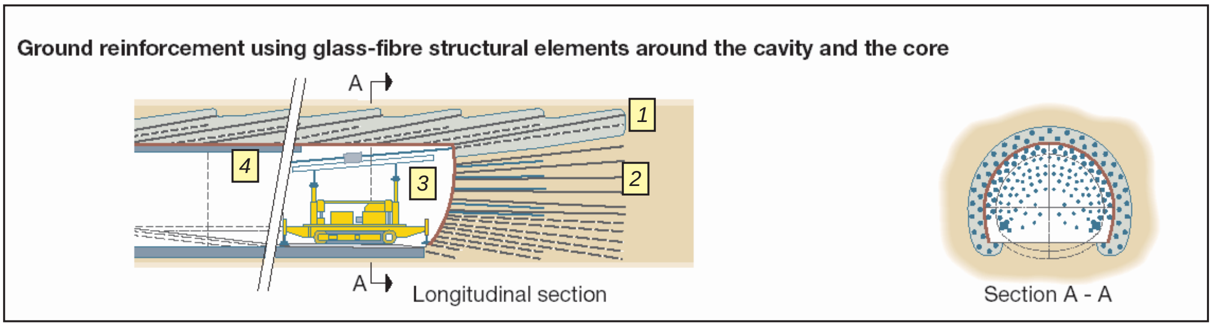


Figure 6.7.8 – Full face excavation, face reinforcement and forepoling (after Lunardi, 2000)

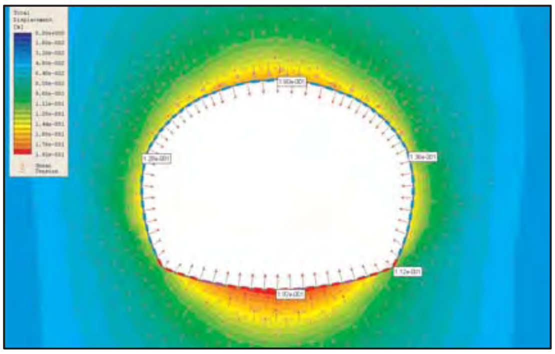


Figure 6.7.9 – Full face excavation

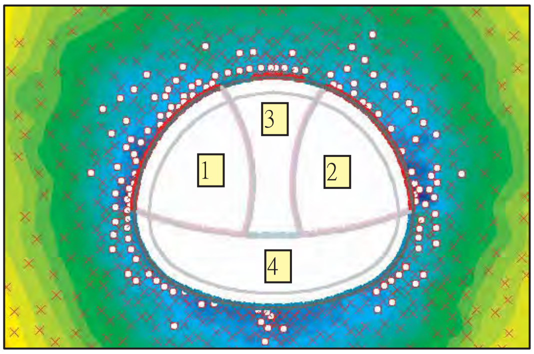


Figure 6.7.10 – Multiple headings

Figure 6.7.11 – Full face excavation, face reinforcement and forepoling during construction (photograph by Nishimatsu-Dragages Joint Venture)

face reinforcement and concrete-lagged steel arches were only required to span across the major zones of weakness (see Figure 6.7.11).

Initial water inflows through probe-holes of up to 350 litres per minute were reduced to less than 20 litres per minute by grouting in advance of the tunnel face.

The contractor's as-built summary log for the Sham Tseng Fault encountered in the KCRC Tai Lam Tunnel is shown in Figure 6.7.12. The key features of this log which enhance its usefulness in providing a realistic depiction of the rock mass conditions are:

- weak zones are clearly identified in the tunnel wall and crown sketch,
- separate descriptions are provided for the rock mass and the weak zones,
- the material strength descriptions are not influenced by discontinuity spacing,
- the water inflow rates from the probe-holes are quantified, and
- the records of the temporary support and progress rates provide a good indication of relative tunnelling difficulty.

The essential difference between the water tunnel log and the contractor's Tai Lam Tunnel log is that the former depicts a very thick, brecciated and crushed, weak, moderately and highly decomposed zone (potentially thick plastic zone around tunnel), while the latter depicts moderately strong to strong rock, containing discrete, relatively narrow faults composed of crushed rock and breccia and a narrow schistose zone (very limited zone of plastic behaviour). The conditions as-encountered are very similar to the more optimistic conditions initially assumed from a general engineering geological knowledge of brittle-ductile fault zones at depth. The differences in implications with regard to relative tunnelling difficulty are considerable, as can be seen in Figure 6.7.7 where the actual ground conditions revealed in the KCRC Tai Lam Tunnel lie between the most favourable interpretation of the water tunnel log and the 'Type 1 Ground'.

Risk Management

The risks were reduced by a cautious approach being adopted by both the contractor and designer, who worked together over a period of about one year to develop the excavation and support strategy for the Sham Tseng Fault for the full range of possible ground conditions.

The risks were further minimised by probing and core-drilling ahead of the face, over-sizing of the excavation to accommodate more deformation than expected and thickening of the lining if necessary, and the establishment of a comprehensive monitoring system with well-defined alert levels and action plans.

6.7.5 Mixed Ground Conditions

The risk of settlement, tunnel collapse, and/or

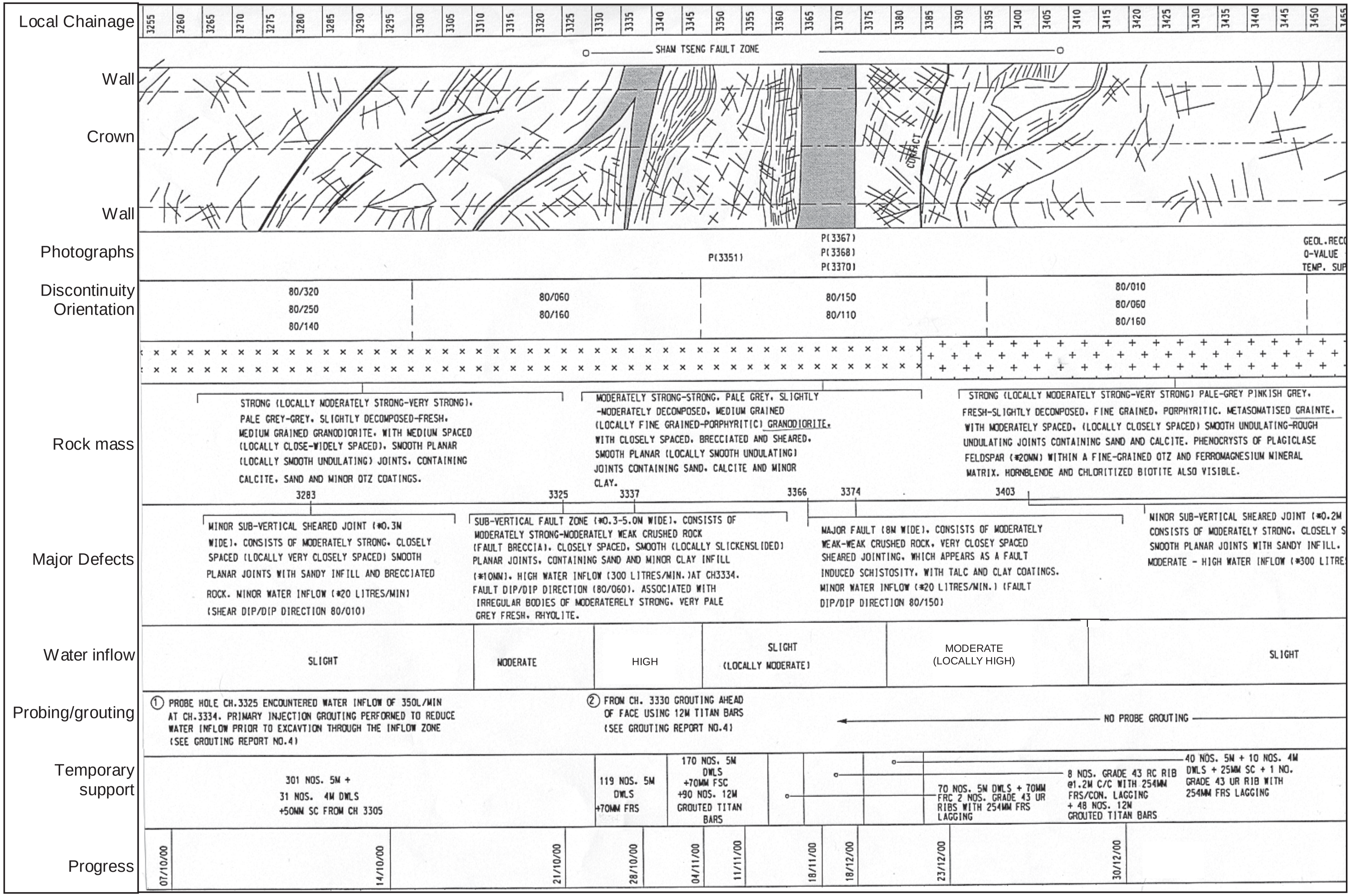


Figure 6.7.12 – Part of an as-built summary log for the Sham Tseng fault zone (provided by Nishimatsu-Dragages Joint Venture)

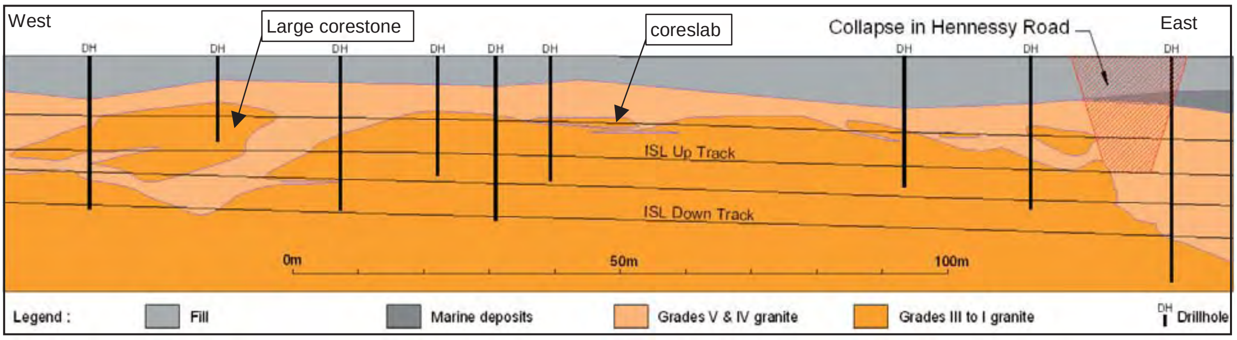


Figure 6.7.13 – Section along the MTRC Island Line to the east of Admiralty Station showing irregular rockhead, corestones, coreslabs and the location of the 1981 Hennessy Road collapse at a steep soil/rock interface (based on as-built records from MTRC)

construction delays is particularly high when tunnelling in mixed face conditions, especially where its occurrence has not been accommodated in the overall risk management and tunnel construction strategies.

Accurate prediction of rockhead using vertical drillholes can be problematic, particularly in urban areas where the location of existing infrastructure such as roads and other structures and utilities may limit the locations where drilling can be carried out.

Figure 6.7.13 shows the irregular rockhead profile encountered along a section of the MTRC Island Line to the east of Admiralty Station. The rockhead profile is influenced by weathering along both sub-vertical and sub-horizontal features, resulting in lage corestones, coreslabs and steep rockhead interfaces which could not be accurately defined by drillholes carried out at between 10 m and 60 m spacing.

Broad variations in levels of rockhead may be predicted by contouring of rockhead based on a combination of drillhole data and knowledge of the local sub-vertical faulting and jointing pattern (see Figure 3.2.5) but zones of weathering along shallow dipping features such as stress-relief joints cannot be realistically predicted by this method.

Lessons learnt from collapses at steep rockhead interfaces such as that shown in Figure 6.7.13 have led to comprehensive probing, grouting and pre-reinforcement strategies being required for tunnelling works in Hong Kong. Where open-face tunnelling techniques are employed, probe holes using percussive drilling or coring can be carried out to confirm the general ground conditions ahead of the face. Figure 6.7.14 shows an example of the 3-D construction of a soil to rock interface based on the logging of cuttings from drillholes carried out for pre-reinforcement and grouting. Such methods can be particularly useful in defining complex interfaces to facilitate forward-planning of tunnel support and ground treatment requirements.

In the case of open-face bored tunnelling works, the main considerations, in addition to the anticipation

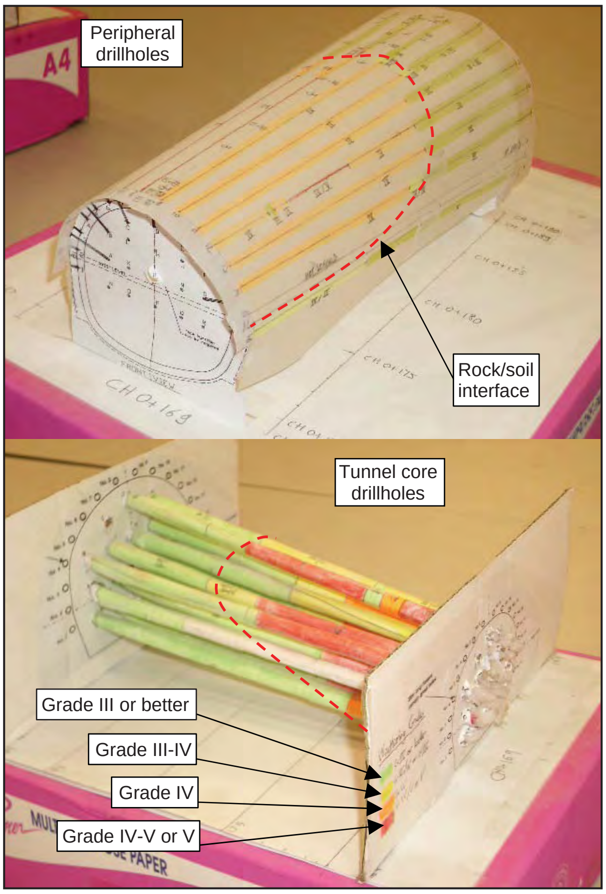


Figure 6.7.14 – 3-D models of a rock/soil interface based on logging of cuttings from drillholes for pre-reinforcement and grouting (models by Gammon Construction Ltd)

and detection of mixed face ground conditions, are limiting groundwater ingress which may lead to excessive settlement, and limiting loss of ground and face instability which may lead to collapse of the tunnel and/or development of crown holes at the ground surface. Where groundwater inflows are of major concern, compressed air, ground freezing or grouting, possibly in combination with groundwater recharge wells, may be used to maintain groundwater levels.

Selection of the most practicable method of ground treatment is heavily dependent on a thorough understanding of the hydrogeology and soil grading, particularly where grouting is to be carried out. Experience of grouting in decomposed granite, colluvium and alluvium during construction of the MTRC Island Line is given in Howat & Cater (1983), Willis & Shirlaw (1984), Bruce & Shirlaw (1985) and Shirlaw, (1987). Accounts of ground settlement during soft ground tunnelling for the MTRC Island Line are given in Cater & Shirlaw (1985) and Thorley (1985, 1986a,b). These accounts indicate that mixed ground interfaces often resulted in higher settlements than when tunnelling in relatively uniform conditions.

For tunnels in non-urban areas and marine crossings, geophysical methods to help determine rockhead may be useful. However, where transitional weathering profiles result in a large number of corestones lying above the PW90/100 weathering surface, it may be difficult to distinguish corestones from rockhead. The results of geophysical surveys should always be adequately calibrated and confirmed by direct drillhole investigations.

Corestones and rockhead interfaces can be problematic where they are encountered along immersed tube tunnels (IMT) and closed-face TBMs such as earth pressure balance and slurry shield machines. Figure 6.7.15 shows the geology, possible tunnelling options and ground investigation layout for a hypothetical marine crossing. In this example, both the immersed tube tunnel and the predominantly soft ground bored tunnel (BT) option might encounter a zone of saprolite with corestones defined by geophysical surveys as seismic 'rockhead' which would need direct drillhole investigation to better define the ground conditions.

In the case of the IMT option, the breaking out and removal of large corestones may result in additional costs, delays and inconvenience to shipping. For example, large corestones up to 200 m^3 in volume were encountered during the IMT excavations for the first MTRC cross-harbour route (Kennedy, 1980). These were removed over a period of about five months using underwater blasting which consumed over 8,000 kg of explosives.

In the case of the closed-face, soft ground bored tunnel option, the presence of large corestones and rockhead interfaces might result in long delays and

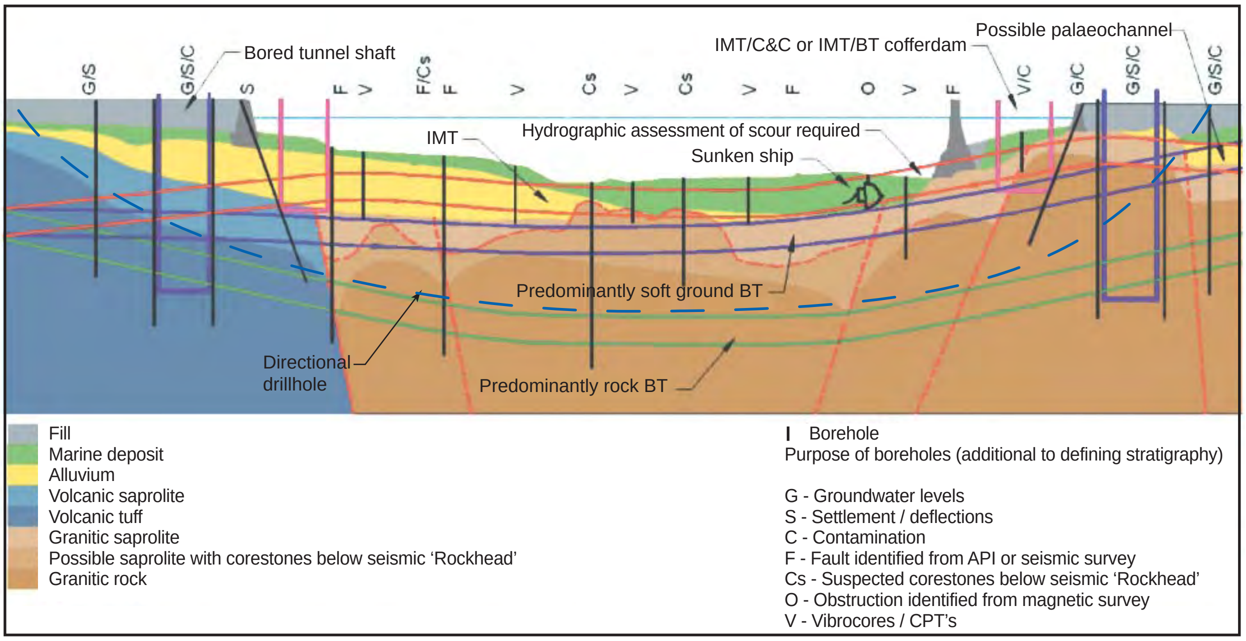


Figure 6.7.15 – Typical layout of investigations for a marine tunnel crossing

increased risks, caused by boulders being pushed ahead of the cutters and ground loss, sinkholes or blow-outs resulting from over-mining of the softer material. Shirlaw (2002) notes that many cases of major ground loss have occurred during closed-face tunnelling in mixed face conditions. As the necessary ground treatment works are most effectively carried out from the surface prior to the tunnelling works being carried out, engineering geological input is required to anticipate and investigate the occurrence of problematic ground conditions prior to construction.

6.7.6 Hydrogeology

General

Ground settlement due to groundwater inflows in underground excavations is a significant risk which can be difficult to quantify prior to construction. This is due to the uncertainties regarding the nature of local and regional hydrogeological regimes. These include:

- the overall storage capacity and permeability of the rock mass, weathered zones and superficial deposits,
- the presence and connectivity of highly conductive discontinuity networks and relatively permeable zones, e.g. palaeochannels,
- the presence and effectiveness of natural and artificial aquitards, and
- mechanisms and rates of recharge.

CDM (2004) summarises the lessons learnt with regard to groundwater drawdown and associated ground settlement during construction of the HATS Stage 1 tunnels. One of the most important lessons is the need for detailed hydrogeological studies to be carried out at an early stage of a project in order to establish the potential extent of the area which could be adversely affected by groundwater drawdown and hence the area in which further investigations and monitoring need to be made.

Prediction of Groundwater Inflows in Rock Tunnels

A number of empirical methods exist for the prediction of groundwater inflows in rock tunnels (e.g. McFeat-Smith *et al.*, 1998; Norwegian Tunnelling Society, 2002). All of these methods are dependent on characterisation of the rock mass in terms of fracture frequency, aperture, persistence and inter-connectivity.

However, rock mass classifications are limited in the number of parameters used to define the classification systems and may be unreliable predictors of groundwater inflows except in so far as they are likely to give lower rock mass ratings where geological structures occur that may give rise to high groundwater inflows.

Experience from the HATS Stage 1 tunnels suggests that most of the groundwater inflows were related to geological structures such as faults and dykes (Section 6.7.3). Identification and assessment of ground affected by these features using engineering geological principles and practice can assist in the estimation of potential groundwater inflows, especially where direct ground investigation data are limited.

The following two examples illustrate the influence of geology on groundwater drawdown and settlement during tunnelling works.

Construction of a Tunnel in Central District

Cowland & Thorley (1985) describe an unusual pattern of groundwater drawdown and settlement that occurred during construction of a tunnel in Central. Figure 6.7.16 shows a plan and section of the geological model and groundwater drawdown pattern. It is notable that the drawdown pattern is offset from the tunnel axis and appears to be associated with a depression in rockhead.

The maximum drawdown recorded in the overlying fill and marine deposits was about 2 m, while the maximum drawdown recorded in the underlying completely to highly decomposed granite was 25 m, located approximately 100 m to the west of the tunnel axis. The settlement pattern closely followed the drawdown pattern, with total settlement in the vicinity of the Star Ferry Car Park being of the order of 75 mm (see Figure 6.7.17). This caused superficial structural damage to buildings and pavements.

The hydrogeological controls can be summarised as follows:

- The fine-grained marine deposits provided an aquitard preventing recharge by the sea into the underlying weathered granite.
- The diaphragm walls of Central MTR Station provided a significant barrier to groundwater recharge from the hillside.
- The relatively modest water inflows into the tunnel

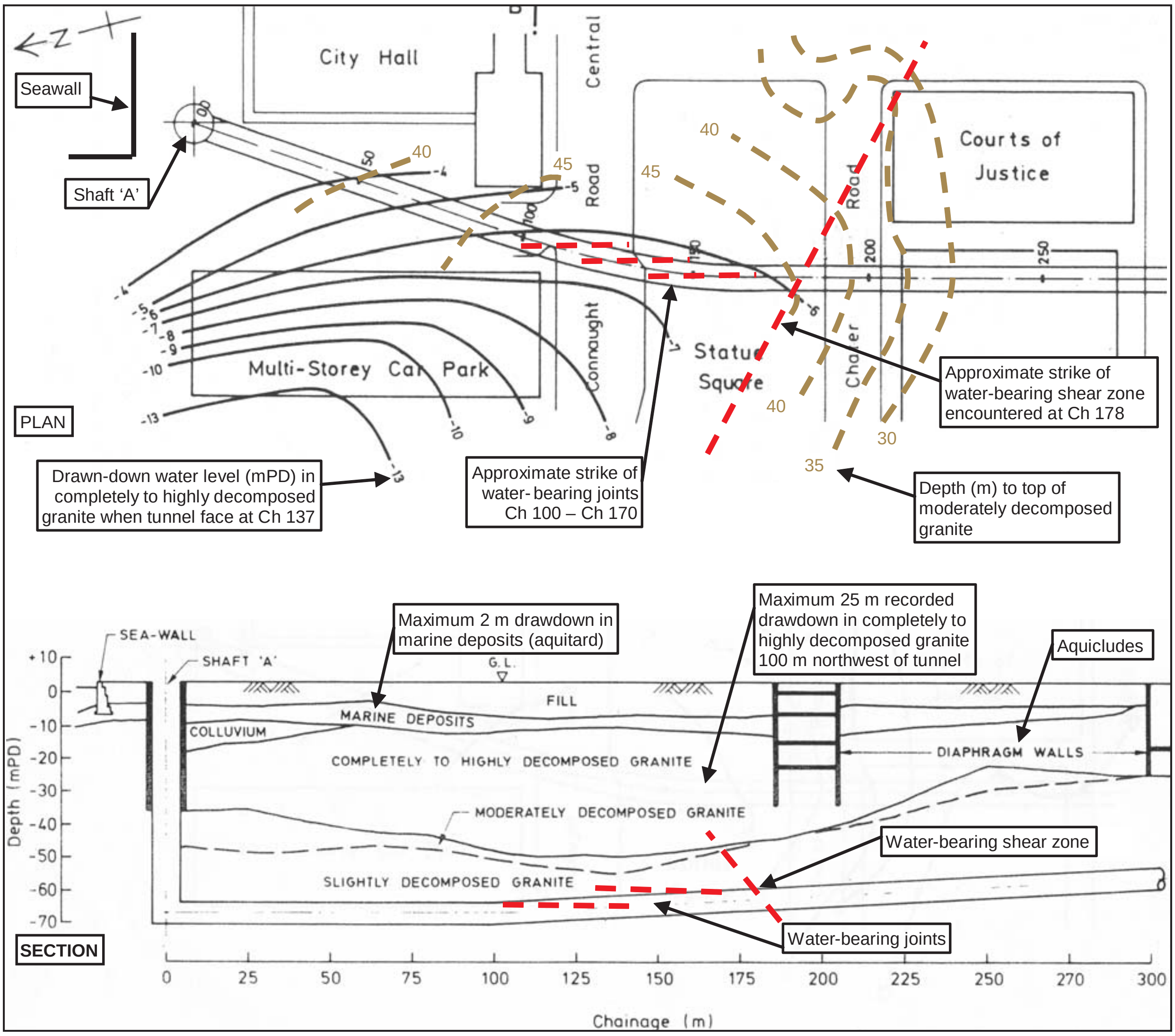


Figure 6.7.16 – Plan and section showing geological model and groundwater drawdown during rock tunnelling works (after Cowland & Thorley, 1985)

effectively drained the available aquifer in the completely decomposed granite.

- The rapid response of the drawdown to localised inflows suggests the existence of good hydraulic connections between some of the joints in the rock mass and the completely decomposed granite.
- The offset of the drawdown trough indicates that preferential drainage paths exist in the weathered granite which were only intersected by the tunnel in a few specific locations.

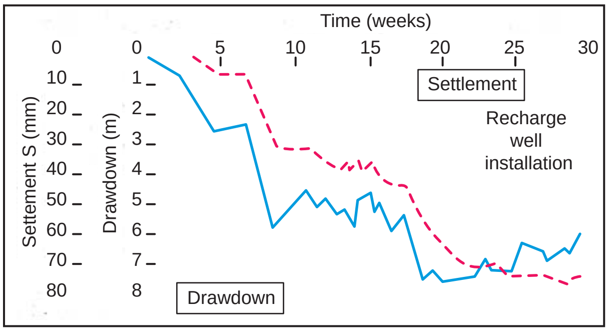


Figure 6.7.17 – Groundwater drawdown in decomposed granite and settlement recorded adjacent to Star Ferry Car Park during rock tunnelling works (after Cowland & Thorley, 1985)

Construction of HATS Stage 1 Tunnel 'C'

Figure 6.7.18 shows the hydrogeological setting of Tseung Kwan O Bay and HATS Stage 1 Tunnel 'C'. During construction of the first 1,700 m of tunnel, large water in flows were encountered through the generally highly fractured tuf fs, despite signi ficant pre-grouting and back-grouting ef forts to control water ingress.

Figure 6.7.19 shows the location of faults encountered

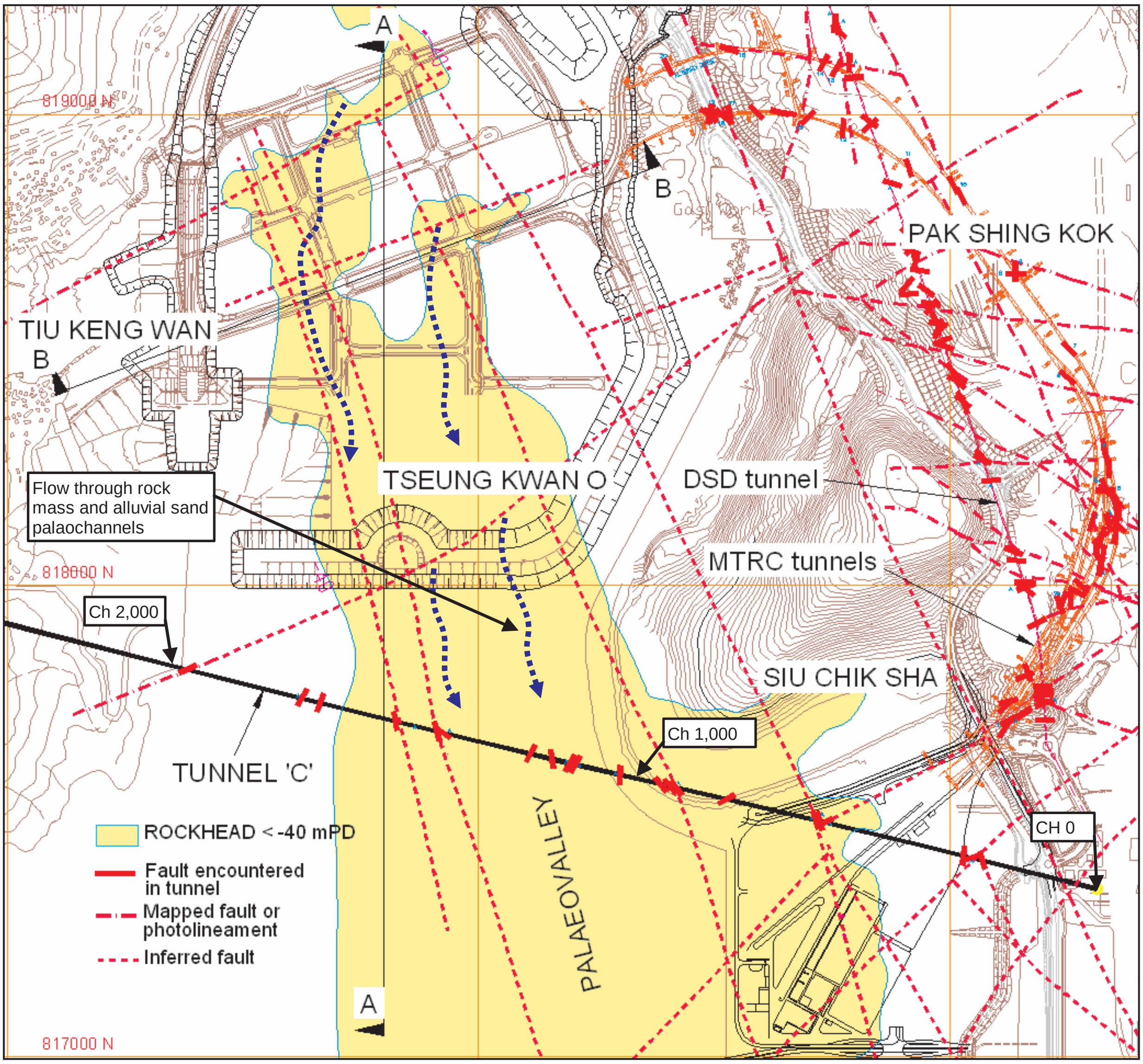


Figure 6.7.18 – Hydrogeological setting of Tseung Kwan O Bay and HATS (Stage 1) Tunnel 'C' showing palaeovalley, faults encountered in tunnels, photolineaments and inferred faults

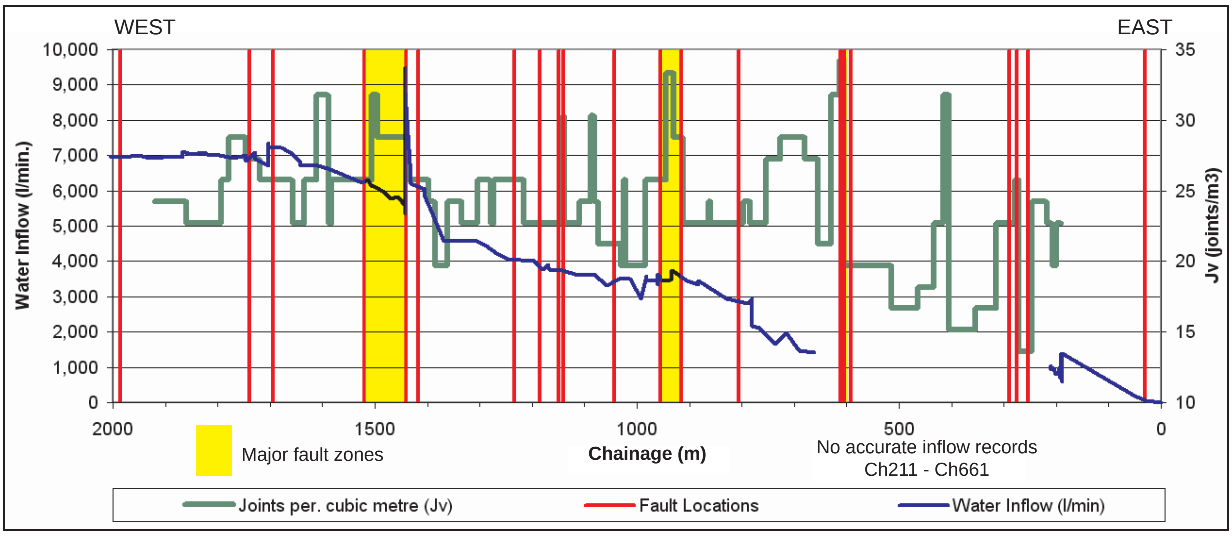


Figure 6.7.19 – Tunnel 'C' volumetric joint frequency (Jv), fault locations and water inflow rate vs. chainage across Tseung Kwan O Bay

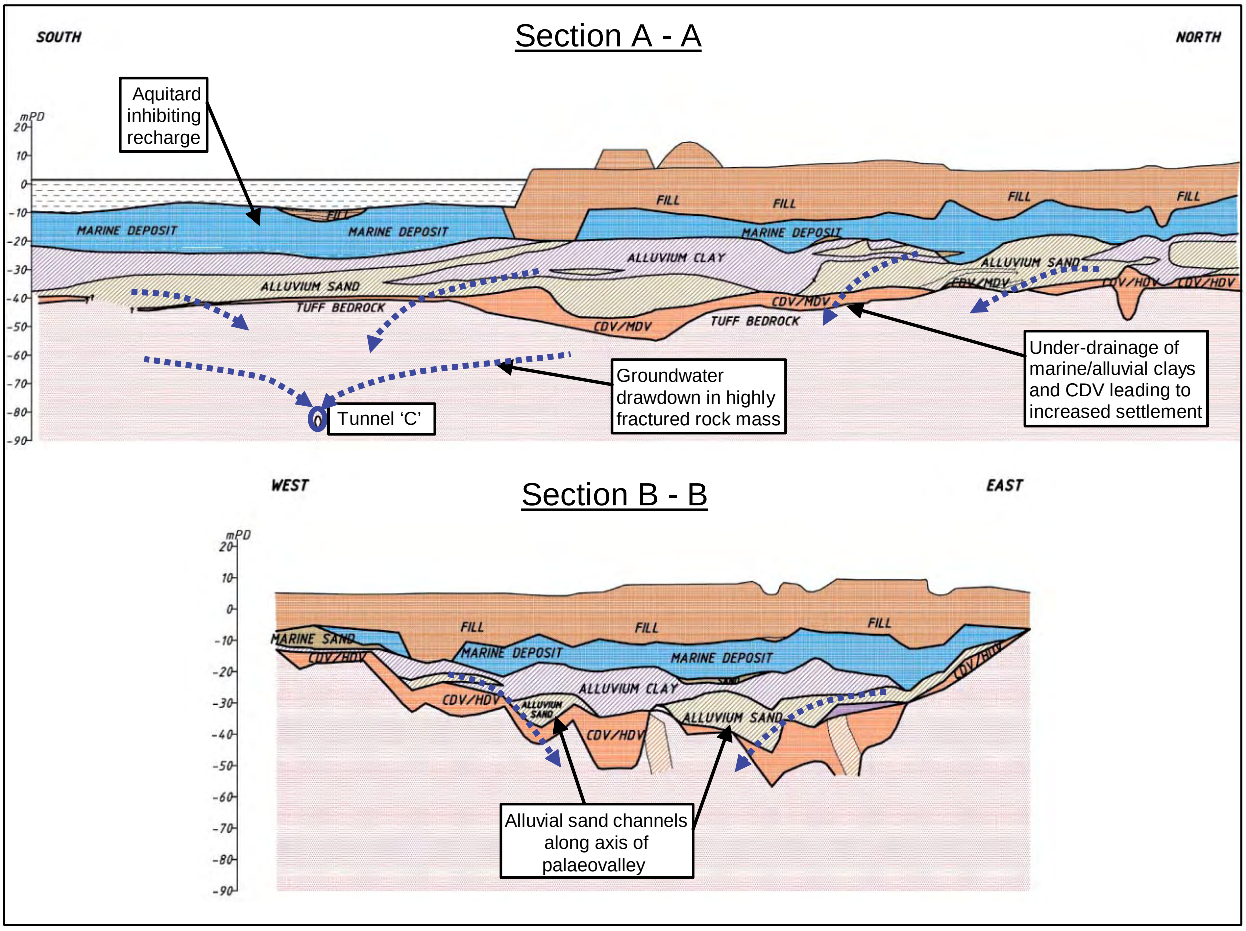


Figure 6.7.20 – Geological sections showing inferred mechanism of reclamation under-drainage during tunnelling works (see Figure 6.7.18 for location of sections)

in Tunnel 'C', the overall water in flow rate into the tunnel and the volumetric joint frequency versus chainage. Although some peaks in the water in flow rate appeared to be related to faulting and joint intensity, the average water in flow rate steadily increased to about 7,000 litres per minute by the time the tunnel had been progressed to chainage 1,700. The groundwater level was consequently drawn down, and large, unexpected settlements occurred at distances up to 2 km from the tunnel.

The geological model and inferred groundwater drawdown mechanisms are shown in Figure 6.7.20. Tseung Kwan O Bay is a palaeovalley which is probably related to weathering along a system of NNW-trending faults within the generally closely-jointed tuff rock mass. Alluvial sand/cobble infilled channels run along the axis of the palaeovalley and are overlain by clayey alluvium and/or marine deposits upon which the reclamation has been placed. The water inflows into Tunnel 'C' and prevention of effective recharge by the marine deposits led to under-drainage of the marine deposits beneath the reclamation via pathways through the rock mass, saprolite and alluvial sand channels (Figure 6.7.20). The experience with regard to groundwater drawdown during the HATS Stage 1 tunnelling contracts (CDM, 2004), can be summarised as follows:

- Where aquitards are continuous over large distances and inflows into a deep tunnel are not sufficiently controlled, the drawdown of groundwater in a confined aquifer where recharge comes from a distance, can extend one to two kilometres from the tunnel.
- Deeper tunnels affect a wider area.
- Large and sustained groundwater inflows into a tunnel in rock are commonly controlled by zones of inter-connected and dilated discontinuities related to adverse geological structures. Groundwater inflows from such zones may deplete aquifers at some distance from the tunnel. Any zones closer to the tunnel that have less well developed connections to the main geological structures may be less affected in the short-term. Over time these zones may also become depleted as the groundwater system reaches steady state.

6.7.7 Caverns

General

Technical studies for caverns in Hong Kong commenced in the 1980s (Neste OY Engineering, 1982a,b; OAP, 1990). These were followed by a number of cavern project studies (OAP, 1991a,b), and the publication of Geoguide 4: Guide to Cavern Engineering (GEO, 1992). Between 1992 and 2000, cavern area studies for North Lantau, Hong Kong West, Hong Kong East and Kowloon were undertaken by GEO (Choy, 1992; Roberts, 1993, 1994; Roberts & Kirk, 2000). These incorporate many of the key findings of the previous studies and classify the suitability of land for rock cavern development, based on consideration of surface developments, underground facilities, topographical constraints and engineering geological conditions (Table 6.7.7).

Chan & Ng (2006) review the past work carried out on cavern related activities and discuss the prospects for promoting the use of under ground space for public and private facilities. They note that the successful construction of a number of caverns, such as the Tai Koo MTR concourse, Mount Davis refuse transfer station, Stanley sewage treatment plant and Kau Shat Wan explosives depot, have demonstrated that caverns can provide cost-effective engineering solutions, particularly for environmental and security-sensitive facilities.

CAVERN LAND CLASS	SUITABILITY	DESCRIPTION
IA	High	Land generally above 60 mPD and at least 50 m from major underground installations or geological faults.
IB	High	Land generally below 60 mPD and at least 50 m from major underground installations or geological faults.
IIA	Medium	Land within 50 m of geological faults. Higher excavation and support costs associated with more difficult ground conditions may occur. May require more detailed site investigation than Land Class I.
IIB	Medium	Land within 50 m of major underground installations. Higher construction costs expected due to constraints on excavation and construction techniques. May require more detailed site investigation than Land Class I.
III	Low	Low suitability due to the presence of existing or proposed surface development which may cause complicated land ownership and resumption issues and may require more difficult cavern engineering techniques.

Table 6.7.7 – Cavern suitability classification (Roberts & Kirk, 2000)

Large Span Tunnels and Caverns in Hong Kong

Examples of large span tunnels and caverns in Hong Kong are shown in Figure 6.7.21 and Table 6.7.8 along with key references.

Until the 1990s rock tunnels and caverns were built primarily for transportation, utilities and water supply and were rarely more than about 12 m in width. Two exceptions are the Quarry Bay and Tai Koo Shing MTR stations, where caverns of 15 m to 24 m span were constructed in the early to mid-1980s.

In the 1990s construction of several three-lane road tunnels, wide railway crossover caverns and TBM launching chambers with spans of 15 m to 30 m were carried out. In addition, three specialised caverns for sewage treatment, refuse handling and explosives storage, were constructed at Stanley, Mount Davis (Island West Waste Transfer Station) and Kau Shat Wan respectively (see Figure 6.7.21). The performance of these caverns has generally been satisfactory, except for minor loosening in the Stanley cavern requiring remedial shotcreting and seasonal groundwater in flows in the Mount Davis cavern, where drip canopies in the lower sidewalls needed to be installed to deflect the water.

Tai Koo Shing MTR Station

An isometric view of the station and adjoining tunnels is shown in Figure 6.7.22. The main cavern, completed in 1985, is 251 m in length, 16 m high and 24.2 m wide. Shortly after construction of the reinforced concrete lining, the overburden was reduced from a maximum of 80 m to about 1 1 m by site formation works for the Kornhill development (Sharp *et al.*, 1986).

Owing to programming constraints, relatively little direct investigation of the cavern area was carried out before excavation commenced. Investigations included surface outcrop mapping, shallow drillholes to prove rockhead, and two inclined drillholes which intersected the cavern area (Figure 6.7.23). A 'flexible' design approach was adopted whereby the cavern would be excavated in stages and rock bolt and shotcrete support installed, based on ground conditions observed and results of performance monitoring.

Figure 6.7.21 – Examples of large span tunnels and caverns in Hong Kong

Name and Purpose	Maximum Span	Rock Type	Rock Condition (Q-system)	Design Method	Comments
Quarry Bay MTR Station - overrun tunnels (later modified to Quarry Bay Congestion Relief Works diversion caverns)	18 m	Granite	Extremely Good	Q-system, arch-building with bolts and discrete element analysis	Horizontal roof originally 15 m span (Matson, 1989). Later widened to 18 m supported with permanent bolts and shotcrete.
Route 3 Tai Lam Tunnels – Portals	18 m	Granite (South Portal) Completely to highly decomposed granite (North Portal)	Poor (South portal) Exceptionally poor (North Portal)	Q-system and block analysis (South Portal) Finite elements (North Portal)	South Portal – two 16 m span road tunnels and one 18 m span flared ventilation tunnel with 5 m rock cover. Separation < 8 m. Extensive pre-support to prevent dilation. North Portal - NATM centre diaphragm wall method of excavation. Pipe-roof pre-support. Iterative integration of monitoring and analyses enabled optimisation of construction sequence. (Endicott *et al*., 2000)
KCRC West Rail DB350 – triple-track tunnel and portal	20 m	Granite with minor faults and rhyolite and basaltic dykes generally striking normal to tunnel axis	Fair to good	Q-system and block analysis	500 m long tunnel section with 6 m rock cover at portal requiring pre-support and wedge stabilisation.
Taikoo Shing MTR Station	24 m	Granite with faults generally striking normal to axis	Good to Poor	Q-system, block analysis and boundary elements	Permanent concrete arch roof. Excavation for Kornhill site formation reduced cover from about 80 m to a minimum of 11 m. (Sharp *et al*., 1986)
Pak Shing Kok MTR Tunnels - Crossover Cavern	25 m	Tuff	Fair	Q-system, block analysis and finite elements	Arch roof with varying cross-section supported by permanent bolts and shotcrete lining.
KCRC West Rail DB320 - Portals	27 m	Granite	Fair	3-D finite elements and 2-D discrete elements	Two, 13 m and 9 m span flat arch tunnels with 10 m separation, skewed to portal face with 5-7 m rock cover. Extensive pre-support to prevent dilation. (Bandis *et al*., 2000)
Quarry Bay MTR Congestion Relief Works - temporary TBM launching cavern	30 m	Granite	Good	Q-system and block analysis	Flat arch cavern with 4 portals and one 70 m deep shaft with temporary bolt and shotcrete support.
Kau Shat Wan Explosives Depot	13 m	Granite and feldsparphyric rhyolite	Poor to fair	Q-system and block analysis	Ten caverns, 6.8 m high with 12 m separation. Permanent bolts and shotcrete support.
Stanley Cavern Sewage Treatment Works	17 m	Granite with shear zone and weak basalt dyke	Fair to good	Q-system and block analysis	Two main treatment caverns with separation of about 25 m and one intersecting service cavern. Extensive unsupported sections later required shotcreting to prevent small rock falls. (Oswell *et al*., 1993)
Island West Waste Transfer Station	28 m	Coarse ash tuff	Good	Q-system, block analysis and finite elements	Permanent bolts and shotcrete support. Seasonal water inflows. (Littlechild *et al*., 1997)

Table 6.7.8 – Summary of selected examples of large span tunnels and caverns in Hong Kong

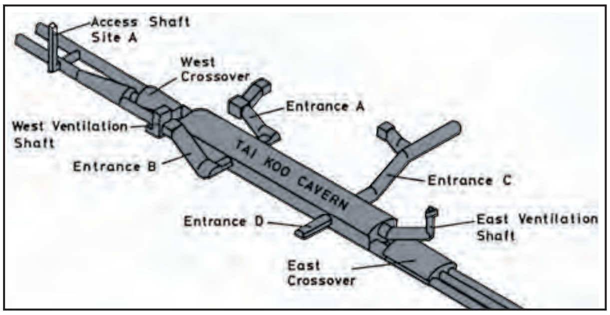


Figure 6.7.22 – Isometric drawing of Tai Koo MTR Cavern (Sharp et al., 1986)

During the excavation, engineering geological mapping of the cavern identified five structural domains and two main faults, F1 and F3 (Figure 6.7.23) as follows:

- Zone I: Extreme western end of cavern in moderately decomposed granite; slightly decomposed to fresh granite elsewhere; persistent low-angle sheeting joints in arch.
- Zone II: Slightly decomposed to fresh granite; no sheeting joints.
- Zone III: Moderately to highly decomposed granite; closely-jointed with slickensiding and chloritisation; Fault zone F1 - 5 to 8 m in width.
- Zone IV: Slightly to moderately decomposed granite, sheeting joints in arch; Fault zone F3 forms

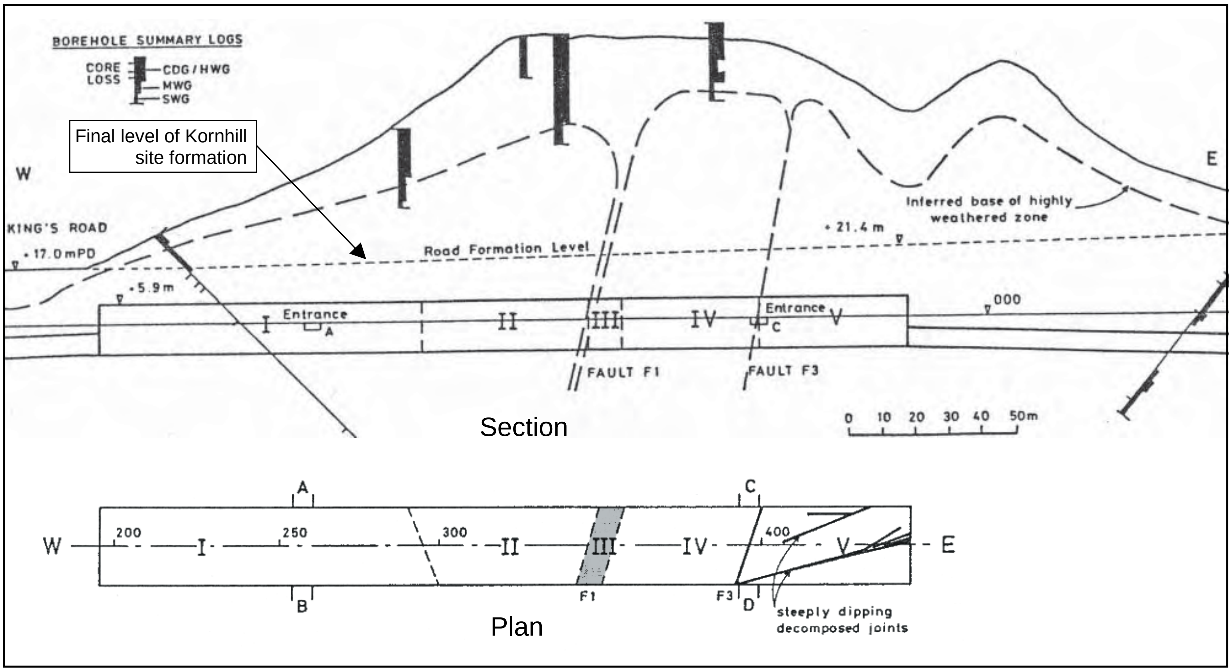


Figure 6.7.23 – Longitudinal section and plan of Tai Koo MTR cavern showing zonation (I to V) based on rock mass characteristics (Sharp et al., 1986)

the eastern boundary and is similar in character to F1 but much narrower in width.

- Zone V: Slightly decomposed granite; major, sub-vertical, highly to completely decomposed seams up to 1 m wide striking sub-parallel to the cavern axis with associated completely decomposed zones up to about 200 mm thick.

Two sub-vertical joint sets striking sub-parallel to the cavern axis and sub-parallel to Faults F1 and F3 were present throughout the cavern.

Initial support requirements were based on the engineering geological mapping. Typical cross-sections, excavation stages and rock reinforcement layouts are shown in Figure 6.7.24. Zone III required the heaviest support with bolts being installed on a 1.0 m grid. In Zones IV and V, the bolt spacing was increased to 1.5 m, but additional 7 m long bolts were installed across discrete weathered joint planes to provide increased support. A 100 mm thick, mesh reinforced shotcrete cover was applied to prevent loosening and spalling at the extreme western end of Zone I and throughout Zones III, IV and V.

An observational approach was adopted. Performance of the initial support, which had been based on the engineering geological mapping, was checked by observation of the periphery and monitoring of extensometers as the cavern was progressively enlarged. The final concrete lining design was carried out after excavation, when complete engineering geological mapping and ground response data were available.

The performance of the final concrete lining was closely monitored during excavation for the Kornhill development using extensometers, embedded strain gauges, pressure cells and convegence arrays. Strains were found to be minimal, and stress redistributions well within design limits.

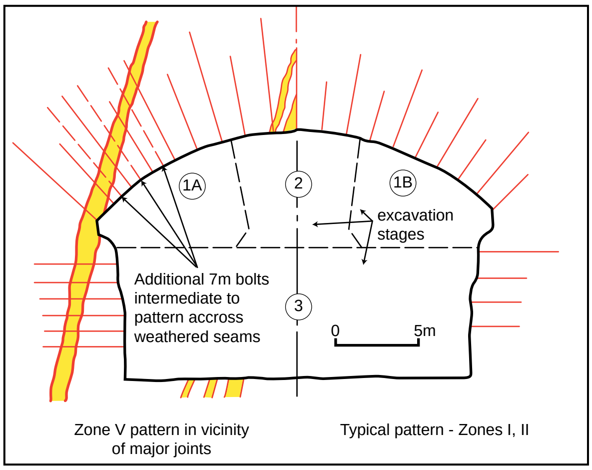


Figure 6.7.24 – Typical cavern sections, excavation stages and rock reinforcement (after Sharp et al., 1986)

This case study provides an example of the use of an observational approach to overcome problematic conditions where initial engineering geological data were not adequate to confirm the actual ground conditions in advance of excavation. Although this situation is similar to other projects involving long rock tunnels under sparsely-developed areas, the engineering geology needs to be defined in much more detail at the feasibility, investigation and detailed design stages when tunnels or caverns are planned beneath existing developments and the consequences of non-performance during construction are much more severe (see GEO, 2005a,b and Pang *et al.*, 2006).

6.8 MARINE WORKS AND RECLAMATION

6.8.1 Introduction

Marine works described in this section comprise :

- reclamation, seawalls and breakwaters,
- beach replenishment,
- mud dredging for navigation channels and anchorages, and
- marine disposal of dredged mud, including contaminated mud.

Foundations for marine structures, such as bridge piers, are discussed in Section 6.5, and construction of immersed tube tunnels is included in Section 6.7.

An important change that has affected reclamation over the years is that works have tended to be located progressively further from the original shoreline. This has meant that foundation conditions have typically changed from granular sediments overlying competent deposits in the near-shore area to an increasing thickness of fine grained and more compressible deposits further from shore. As a result, marine works today require comprehensive programmes of investigation to determine suitable founding conditions.

Cross-reference should be made to Section 5.8 which describes engineering geological aspects of the superficial deposits in the offshore area but some key issues relating to marine works are identified below:

- Apart from intertidal coastal areas, there is no opportunity to examine and map exposures of marine sediments. Therefore, geological and ground models have to be generated solely from data such as seismic profiles, drillholes and cone penetration tests (CPTs) aided by an understanding of the offshore geology.
- Whether the compressible marine deposits are removed or not, their base needs to be established during the ground investigation, either to determine the dredge level for removal or to accurately quantify their thickness and distribution throughout the site. Because the marine deposits were laid down on a surface which had been gullied by watercourses (see Section 5.8 and Figure 6.8.1), their thickness is greater where they have infilled these gullies. Furthermore, because the organic content of the infilling Holocene deposits is relatively high and anaerobic decomposition may form gas bubbles in the sediment, seismic survey is often ineffective – the so-called "gas blanking" effect (Premchitt *et al.*, 1990b). Although drillholes and CPTs can still be used in these areas, the inability to use seismic profiling to establish

Figure 6.8.1 – Seismic boomer profile showing consolidation drapes of Sham Wat Formation sediment laid down on the gullied surface of the Chek Lap Kok Formation (Fyfe et al, 2000)

the three-dimensional geometry of such channels has posed a significant problem in planning major dredging contracts where the design requirement is to remove all soft material prior to filling. Careful investigation is therefore needed to both delineate and characterise these buried drainage lines and their infilled sediments.
- The extent, and particularly the rates, of consolidation of the Pleistocene alluvial sediments can be difficult to predict. This is due to the nature of their deposition by braided streams bringing sporadic, storm-related granular sediment loads as well as the predominantly fine sediments from the hilly hinterland. Consequently, the alluvial deposits are discontinuous and variable both laterally and vertically. Sand lenses, even discontinuous ones, can promote drainage of porewater during reclamation loading, but modelling their behaviour during ground investigation and subsequent design is difficult.
- Reclamation, particularly in areas directly adjacent to steep natural terrain, has the potential to affect the groundwater regime of the adjoining hinterland by restricting the seaward flow of groundwater (see Figure 6.3.6).

6.8.2 Reclamations, Seawalls and Breakwaters

Reclamation Methods

There are two main methods of forming reclamations – the drained method and the dredged method. In the drained method the soft Holocene marine deposits are left in place whereas in the dredged method the Holocene marine deposits are removed and taken to a disposal site. Another method previously used, is to use the fill placement itself to displace the soft deposits (the so-called displacement method). However, problems with mud waves and the risk of trapping volumes of soft mud beneath the reclamation, with attendant problems of localized long-term differential settlement, mean this method is not now used. The formation of reclamations on top of the soft material has resulted in major failures when imposed loads have exceeded the materials strength (Powell *et al.*, 1984; Leung & Tse, 1990).

Drained Method

The presence of soft Holocene marine mud requires particular attention to be paid to the site geology when designing marine works. Section 5.8 discusses the geotechnical properties of these soft deposits. When the mud is loaded by a reclamation, the increase in stresses will cause deformation which could lead to foundation failure and unpredictable differential settlements if the deformation becomes excessive. Although vertical drains encourage faster porewater dissipation (e.g. Fraser *et al.*, 1990), initial placing of fill on the sea bed is undertaken in layers in order to avoid the creation of mud waves. Kwong (1997) provides a summary of projects that have used the drained reclamation method in Hong Kong from the 1950s to 1995. Detailed knowledge of the geotechnical properties of the marine mud from previous documented work is useful. The Hang Hau Formation has fairly consistent geotechnical properties throughout Hong Kong so the results of previous investigations are relevant, e.g. summary by Yeung & So (2001). However, this knowledge should be supplemented by careful, project specific ground investigation. Ground improvement methods for marine muds are discussed in Ho & Chan (1994).

Endicott (2001) discusses a large number of detailed Hong Kong case histories and indicates that use of wick drains in combination with temporary surcharge loading can effectively provide 100% primary consolidation and significantly reduce secondary consolidation. He also highlights that when sandy lenses or layers are present within the Holocene mud, as has been inferred at reclamation sites at Sai Wan Ho and Castle Peak Bay, Tuen Mun, primary settlement is achieved far more quickly. It is therefore important during the ground investigation to log all lengths of drillhole core and examine all CPT records to determine if any such features are present so as to improve the settlement rate predictions. The use of *in situ* field permeability tests can provide a quantitative gauge of the possible effect of sand layers and lenses without having to fully delineate their physical extent.

The practice of surcharging to reduce long-term secondary consolidation depends on the fact that the secondary consolidation reduces rapidly as the over-consolidation ratio (OCR) increases. Surcharging leaves the soft marine muds with an OCR >1. While the Holocene marine mud is essentially normally consolidated (OCR = 1), the underlying Chek Lap Kok Formation, and to a lesser extent the Sham Wat Formation, have an OCR >1 presumably related to emergence during periods of relatively low sea level, perhaps with associated erosion of material, and perhaps in combination with ageing effects in the sediment. As a result, the settlement characteristics of such layers can be expected to be significantly different from the overlying Holocene mud. For example, a seismic boomer profile showing consolidation drapes of Sham Wat Formation sediment laid down on the gullied, surface of the Chek Lap Kok Formation is shown in Figure 6.8.1.

Dredged Method
In this method, all the highly compressible sediment is removed by dredging and replaced with fill thus significantly reducing total settlement, permitting faster construction of the reclamation, and rendering long-term settlement more predictable. Construction of Hong Kong's Container Terminals 6, 7, 8 and 9 and Hong Kong International Airport provide examples of such considerations.

Different criteria have been used to delineate and specify the dredge limit, depending on the reclamation design requirements, and there will normally be an interaction between the ground investigation works and the design to ensure that appropriate investigation data is obtained to permit dredge levels to be determined in advance (see Section 6.8.3 below). A major disadvantage of the dredged method is that potentially large volumes of dredged mud require disposal, which can be particularly undesirable if the mud is contaminated. Also, a correspondingly large additional volume of fill material is needed. If mud has to be dredged, chemical and biological testing would be required at the ground investigation stage in order to determine the appropriate disposal arrangements (ETWB, 2002).

Partially Dredged Method
A hybrid of the above two methods is to remove the very compressible materials from critical areas only. Although the design and control of dredging and filling are more complicated and differential settlement between dredged and drained areas a potential problem, the reduced dredging and filling volumes can make this a cost-effective solution.

Seawalls and Breakwaters
Design of foundations for seawalls and breakwaters has been relatively unchanged for many years in Hong Kong. In particular, the practice of removing the soft Holocene deposits and founding on the underlying generally firm to stiff alluvial deposits of the Chek Lap Kok Formation continues to be the normal practice. Engineering geological considerations relating to mapping out the design dredge surface are similar to those discussed above in respect of reclamations.

Fill Material
An early part of reclamation planning involves determining the likely source of fill material to form the reclamation. The type of fill material to be used, and its method of placement, will have a bearing on the reclamation design, construction and programme, and may also impose different considerations in the ground investigation and formulation of the geological and design models. General requirements of the different types of fill material for marine works are given in CED (2002a) and summarized in CED (2002b). Reclamation fill could be sand, decomposed rock (normally granite), as-blasted or processed rock and public fill. Assessment of the suitability of natural materials as fill, including dredging-related aspects, is discussed in Section 6.10. Public fill comprises the inert portion of construction and demolition materials (rock, concrete, asphalt, rubble, bricks, stones and soil) and its beneficial use is a major concern.

Seismicity and Reclamations
It is normal practice in Hong Kong reclamation design, including seawall design, to assume that seismic forces are minor in relation to the combined effects of other imposed loads, e.g. see the Port Works Design Manual : Part 1 (CED, 2002b). In the 1990s, the widespread use of dredged sand as reclamation fill prompted the GEO to review the sensitivity of hydraulic fill to the level of seismic loading which might be experienced in Hong Kong. The overall conclusion of the study was that, given the relatively low seismicity in Hong Kong, the use of hydraulic fill does not pose significant problems (Shen & Lee, 1995). A second phase of the study examined five reclamations where sand fill was used and did not identify any problem with the dynamic stability of the sites (Shen *et al.*, 1997).

Coastal Erosion and Instability
During the Holocene marine transgression coastal erosion provided a huge input of sediment into the marine system. Since c.5000 BP, sea level has stopped rising but the pattern of coastal erosion continues though much reduced and can be significant in the stability and safety of reclamations near the original coastal break in slope (Fyfe *et al.*, 2000).

Environmental Aspects
Reclamations are designated projects under the Environmental Impact Assessment Ordinance and as such, they require an Environmental Impact Assessment (EIA). An understanding of the natural physical conditions and processes is required in order to assess environmental impacts. An important, and often contentious issue during the planning and construction stages is that of sediment release during mud dredging and during fill placement, particularly placement of sand fill in areas of the reclamation not yet enclosed by seawalls. These issues are covered in Section 6.10.

An EIA for a reclamation project is likely to require a study of the sea bed which will be buried by the reclamation in order to establish the nature of the littoral and sub-littoral ecosystem. By way of mitigation, consideration can be given to the new habitat which will be created by the rock armour layers on sloping seawalls.

The requirement for a Marine Archaeological Study as part of an EIA often results in collection of detailed side-scan sonar data which can sometimes also provide useful information about sea bed sediment type.

6.8.3 Case Study – Hong Kong International Airport Reclamation

General
The engineering geological aspects described below are pertinent to almost all reclamations in Hong Kong. In particular, the channelised pre-Holocene surface is a ubiquitous feature around the coast, and related difficulties with investigation, design and construction are common (see also James (1994) for the Penny's Bay reclamation).

The airport is constructed on a 12.48 km^2 artificial platform encompassing the former islands of Chek Lap Kok and Lam Chau. Some 69 million cubic metres of Holocene material were removed from the reclamation site. The levelling of the two islands produced 108 million cubic metres of mixed soil and rock which, together with 76 million cubic metres of dredged sand, nearly 7 million cubic metres from levelling the Brothers Islands and over 7 million cubic metres surplus material from another contract, was used to form the reclamation. The design, construction and performance of the airport island have been documented in Plant *et al.* (1998).

A thorough understanding of the engineering geology of the area was central to the reclamation design process and this is the focus of this brief account of the project. The engineering geological aspects of the sand borrow areas and related dredging considerations are discussed in Section 6.10, and certain aspects of the mud disposal are discussed in later sections.

Planning, Investigation and Design
A decade of detailed planning and investigation preceded the construction of the airport, with the largest investigative effort being focused on characterising the compressible sediments at the reclamation site. In addition to ground investigations, a 100 m square trial embankment was constructed to study the effectiveness of different types and spacing of vertical drains in accelerating consolidation of the soft Holocene mud (Premchitt *et al.*, 1990a). A variety of design approaches for the airport were considered before adoption of the final design that involved removal of the soft Holocene deposits. An intensive programme of ground investigation and analysis concentrated on delineating the dredge level needed to remove the soft mud and on estimating the rate and extent of settlement associated with the pre-Holocene compressible materials which would remain beneath the reclamation.

Ground investigations included seismic surveys, vibrocores, drillholes with field vane tests and SPTs, piezo-cone CPTs with dissipation tests, and extensive laboratory testing. As more investigation data became available, both from the airport site and from elsewhere in Hong Kong, so the geological model and the design model progressively developed. Plant *et al.* (1998) record this development of the models. The reclamation design was based on the final geological model, produced in 1993, which was derived from a detailed seismo-stratigraphic interpretation of a 100 m grid seismic reflection survey (50 m spacing near shore), over 200 drillholes and over 400 piezo-cone penetration tests. Figure 6.8.2 illustrates, schematically, the relationship between the stratigraphic units at the reclamation site, and Figure 6.8.3 illustrates in an idealised cross-section the location of the dessicated palaeosol crust of the Chek Lap Kok deposits which was to form the founding layer for the reclamation. Figure 6.8.4 shows an idealised CPT profile of the sea bed before the airport reclamation.

The airport project generated an enormous volume

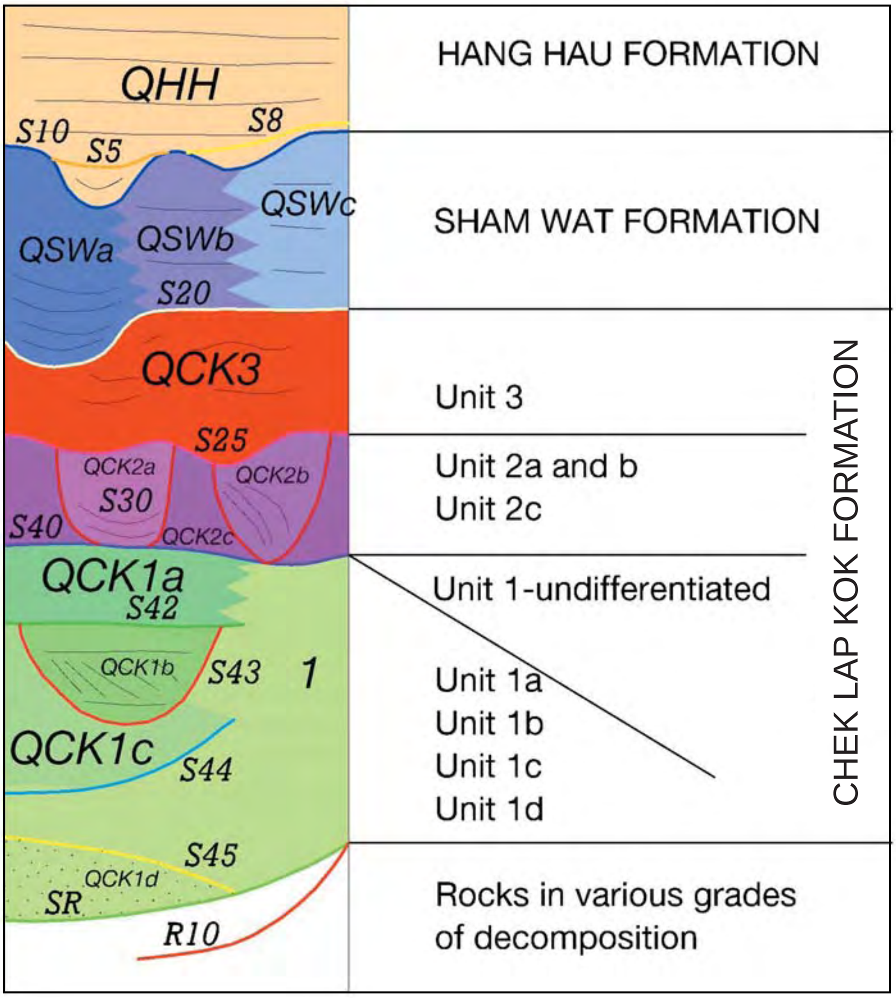


Figure 6.8.2 – Schematic relationship between the seismo-stratigraphic units and seismic reflectors at the airport reclamation site (Plant et al., 1998 after James et al., 1994 – Copyright BGS / NERC)

of ground investigation data. To manage the data and make it readily useable in developing geological and ground models, the Airport Authority established a comprehensive database of digital ground investigation data incorporating stratigraphic, material and test data with applications for drawing cross-sections, filtering data on the basis of test results, etc. While most projects would not warrant development of a special database, the airport project provides a good illustration of how such a system might be set up (Plant *et al.*, 1998).

Key Engineering Geological Issues

The characteristics of the Holocene and Pleistocene deposits in general are discussed in Section 5.8 but a few key engineering geological aspects of the airport site can be highlighted while noting that the design was to remove the soft Holocene mud and found the reclamation on the underlying firm to stiff palaeosol surface of the Chek Lap Kok deposits.

Palaeo-channel Network Across the Site

The geomorphology of the general area of North Lantau suggests that the present-day drainage system which issues into Tung Chung Bay would have extended north-northwestwards between Chek Lap Kok and Lam Chau during periods of low relative sea level. The ground investigation, particularly the seismic surveys, confirmed that, in addition to the channelised surface of the Chek Lap Kok Formation, there is a series of channel deposits within the Chek Lap Kok deposits along the palaeo-drainage line between Chek Lap Kok and Lam Chau.

Uneven Dredge Level

The soft, normally consolidated, grey mud of the Holocene Hang Hau Formation infills and covers a network of channels incised into the pre-Holocene land surface. The base of the Hang Hau Formation, the unit to be removed by dredging, is therefore uneven in both section and in plan. The design

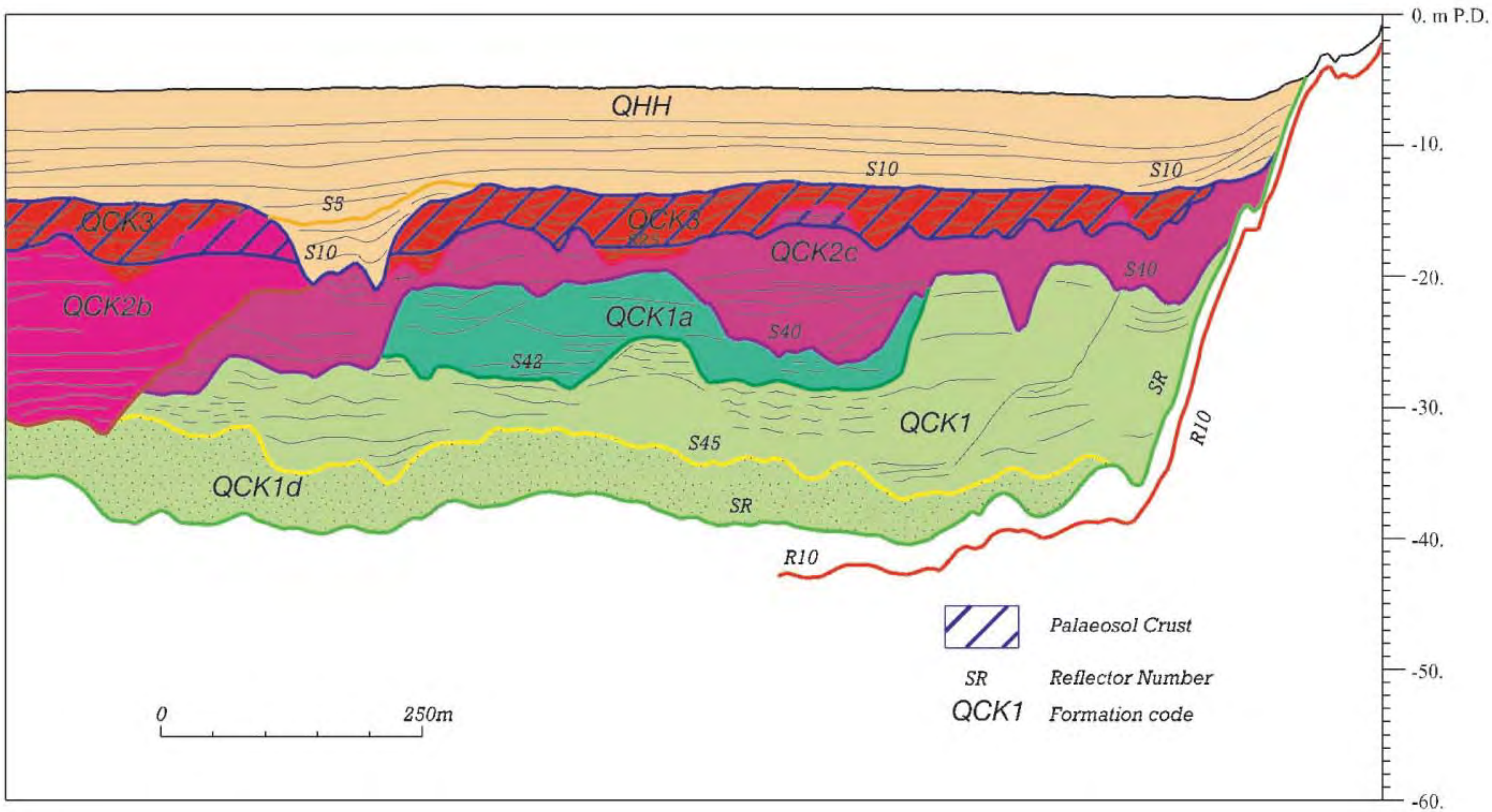


Figure 6.8.3 – A section showing the location of the weathered crust of the Chek Lap Kok deposits which was to form the founding layer for reclamation (Plant et al., 1998 after James et al., 1994 – Copyright BGS / NERC). See Figure 6.8.2 for an explanation of the seismo-stratigraphic units and seismic reflectors.

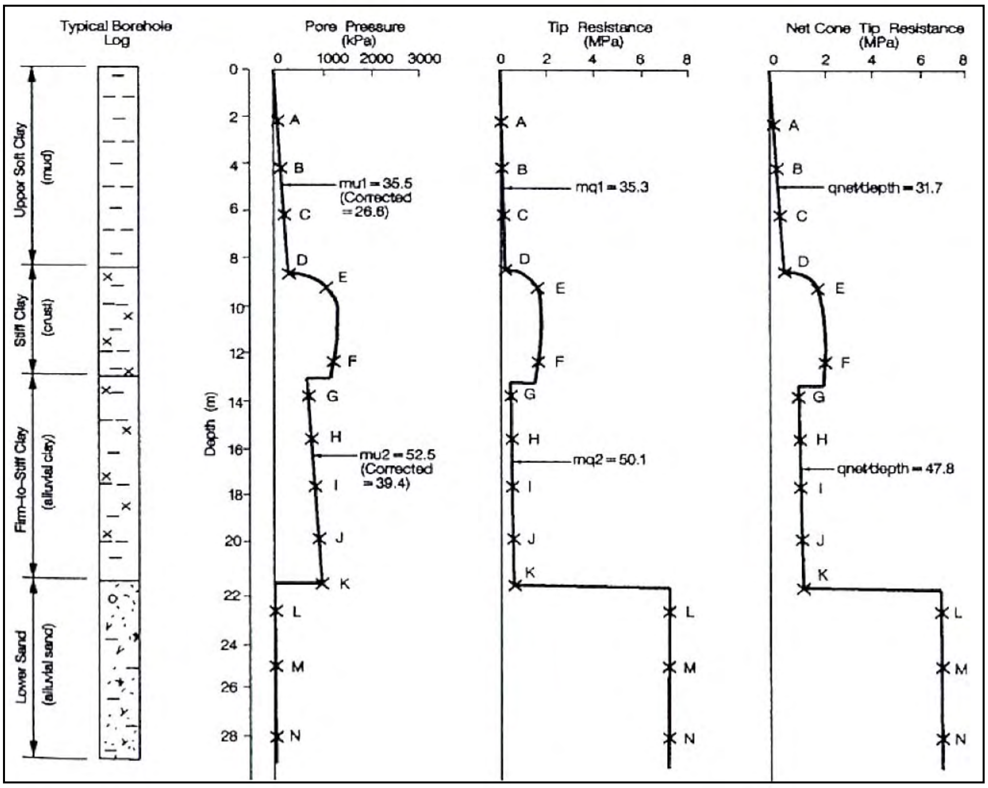


Figure 6.8.4 – An idealised CPT profile of the sea bed before the airport reclamation (Plant et al., 1998)

criterion adopted for the dredge level was 500 mm below the point where net CPTtip resistance reached 500 kPa. The reason for this was that thin sand lenses were commonly found about 500 mm above the base of the soft mud. One metre was added to the criterion for the seawalls.

Seismic Blanking in Critical Areas

The portion of the Hang Hau sediment in filling the pre-Holocene channels tends to have a higher organic content which results in "gas blanking" on the seismic records (see Section 5.8). Additional CPTs were therefore required in some locations to map out the channel base to fix dredge levels.

Good Founding Layer Locally Absent

The firm to stif f desiccated crust of the pre-Holocene ravinement surface formed of Chek Lap Kok alluvial deposits (Unit QCK3 in Figures 6.8.2 and 6.8.3) is poorly developed or absent along the drainage lines, especially the major channels.

Underlying Deposits Variable but Average Parameters Applicable

The geological model divides the portion of the Chek Lap Kok Formation beneath the desiccated crust, into two units, QCK2 & QCK1 (Figures 6.8.2 and 6.8.3), which are further sub-divided into sub-units. The QCK2 unit can be characterised as a firm to stiff over-consolidated clay, albeit with variations due to sand lenses, taken as having a uniform set of parameters for settlement calculation purposes (Plant *et al.*, 1998). The QCK1 unit, the dominantly granular basal unit of the Chek Lap Kok Formation, can be characterised as a sand with subordinate, discontinuous cohesive layers.

Deeper, Compressible Deposits in the Southwest

The Sham Wat Formation, comprising a firm grey silty clay, is present in the southwest corner of the reclamation site where it in fills a major channel system, with little or no desiccated crust to distinguish it from the overlying Hang Hau Formation. Possible correlations of the Sham Wat Formation with similar deposits in the centre of the site are problematic.The deposits in the centre of the site may just represent variations of the Chek Lap Kok Formation deposits, but their mottled and stif fer nature did not pose the same potential problems as the Sham Wat proper did in the southwest where seawall foundations had to be founded at deeper levels.

Construction
Accurate dredging was of paramount importance to avoid leaving significant amounts of compressible mud which would have had adverse effects on the overall settlement of the platform. This was particularly important in areas of pre-Holocene drainage channels where a combination of seismic profiles and CPTs was used to delineate the necessary dredge level (see Figure 6.8.5).

Design Verification and Post-construction Issues
Comprehensive site instrumentation was installed within the airport platform and along the seawalls in order, respectively, to calibrate the predictions of settlement, and measure any horizontal deformations of the reclamation edge. Plant *et al.* (1998) give a detailed account of the instrumentation philosophy, design and installation. One of the key requirements was the need to critically review the instrumentation data against the detailed geological and ground models to facilitate interpretation of the behaviour of the ground and prediction of future settlements. The settlement measurements taken after filling were of fundamental importance in determining the finished levels of the airport buildings and other civil works. Settlement monitoring included extensometers and surface settlement markers, as well as piezometers to record the gradual reduction in excess pore pressures within the consolidating cohesive deposits. Inclinometers were deployed along the seawalls.

6.8.4 Mud Dredging For Navigation Channels and Anchorages

In the planning and design of new navigation channels and anchorages, there are two main roles of engineering geology: firstly, to characterise the deposits so that dredging can be planned and stable side-slopes formed; and secondly, to identify any contamination and therefore the appropriate disposal arrangements.

The stability of dredged slopes depends not only on gravitational forces but also wave loading. Evans (1992, 1994b) discusses the stability of dredged slopes and notes that the 1(V) in 4(H) dredged mud slopes in the Urmston Road channel near Castle Peak are close to a theoretical maximum value of about 1(V) in 3.5(H).

In most of the harbour area, dredged channels and anchorages are formed within the soft, normally consolidated Hang Hau Formation. However, with

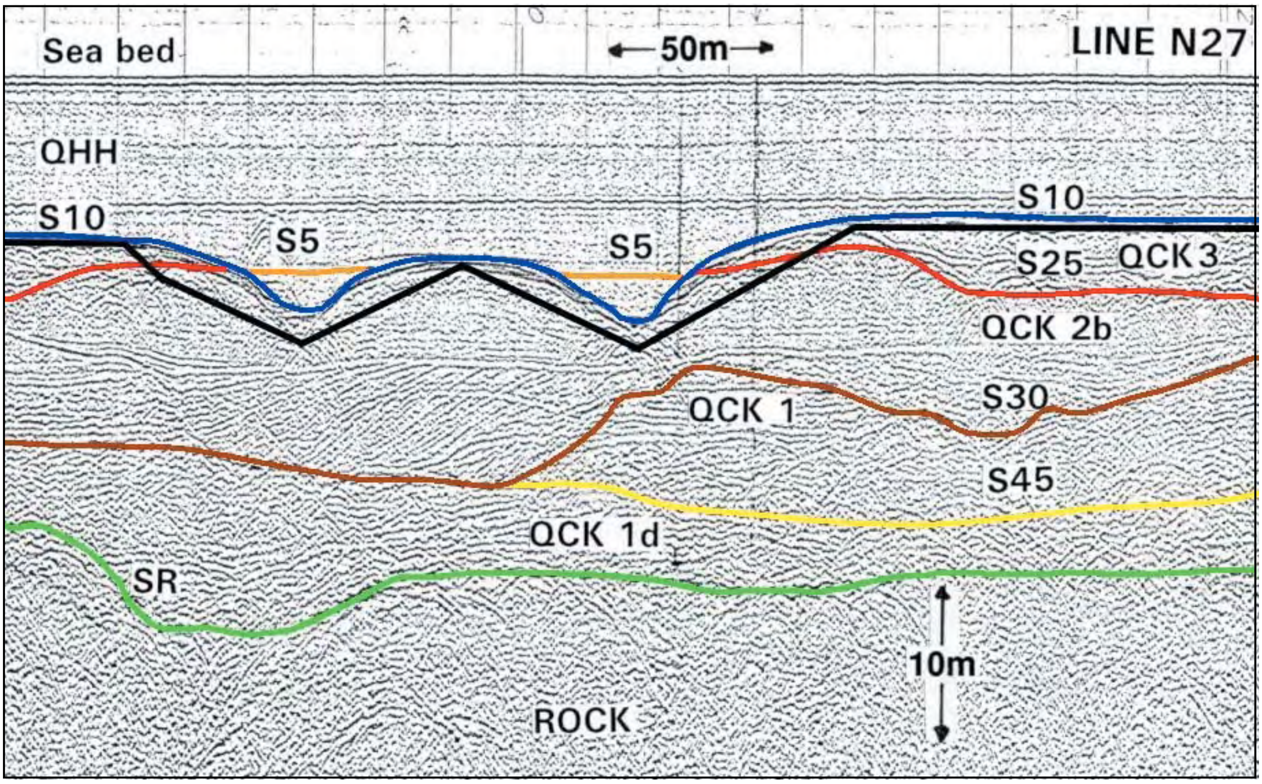


Figure 6.8.5 – Interpretative seismic profile showing the intended dredge level (in black) in areas of pre-Holocene drainage channels at the New Airport site,Chek Lap Kok (after Covil & James, 1997 – Copyright BGS / NERC). See Figure 6.8.2 for and explanation of the seismo-stratigraphic units and seismic reflectors.

vessel draughts increasing and more works being planned in areas further from shore where there is less existing information about sea bed conditions, it is particularly important for any new works that fundamental issues, particularly the possible presence of buried rock subcrops, are addressed. Encountering stronger materials than anticipated at tendering stage can have major cost and programme implications. Peaks of rock or weathered rock, the latter perhaps containing large corestones, are not uncommon and close-spaced seismic surveys should be undertaken to fully characterise the material to be dredged in such areas.

Maintenance dredging involves removal of material that has accumulated within, say, a navigation channel. Although such accumulated material may have been derived locally by slumping of channel sides, it may also have been transported from some distance away and deposited. It is therefore important that material to be removed is properly tested to identify any contamination even if the area itself is not contaminated.

6.8.5 Marine Disposal of Mud

Classification and Disposal of Dredged Sediment
For many years Hong Kong has been implementing controlling legislation to reduce water pollution at source. Nevertheless, the marine sediments close to developed areas are still contaminated by past pollution. Over many decades, both domestic and industrially polluted wastes have found their way via foulwater sewers, stormwater drains and watercourses, into Hong Kong's marine environment. Extensive sampling and laboratory testing have revealed large amounts of metallic and organic pollutants, especially around submarine outfall pipes. Once on the sea bed, pollutants have then been mixed deeper into the sediment by natural processes but also by ships' anchors. As a result, contamination is commonly present in the sea bed sediment to depths of up to 3 m and occasionally more. Details on assessing dredged material are given in ETWB (2002).

It is worth noting that natural geochemical variations in marine sediments have the potential to confound the 2002 classification because there are indications that, for instance, natural arsenic levels can be higher than the threshold level cited in the 2002 classification technical circular (Whiteside, 2000).

Contaminated Mud
The cation exchange capacity of the clay mineral component of the Hang Hau Formation has a significant sequestering effect on heavy metal contaminants which have entered the marine environment from uncontrolled industrial discharges. This characteristic is important because it means that such contaminants are not easily released back into solution during dredging of contaminated mud. Disposal of contaminated dredged marine mud has been undertaken in Hong Kong at East Sha Chau since 1992. In total about 40 million cubic metres of dredged contaminated mud has been safely disposed of (equivalent to an *in situ*, un-bulked, pre-dredge volume of 30 million cubic metres) in purpose-dredged sea bed pits and empty sea bed pits left after sand extraction for the Hong Kong International Airport. As of 2006, the monitoring results have indicated no adverse trends. Whiteside *et al.* (1996) describes the construction and management of the purpose-dredged sea bed pits. Further studies are being undertaken to identify more disposal sites and, as in the establishment and operation of the present facility, engineering geological aspects are centrally important.

The environmental acceptability of the disposal facility was established through studies to assess the environmental impact, and thereafter, actual performance is measured by a special monitoring programme including sediment and water quality, aquatic biota and biological effects testing (Shaw *et al.*, 1998).

Uncontaminated Mud
Open sea bed sites for disposal of dredged mud have been in existence for many years. Currently, two disposal sites are in use and are strategically located in southern waters (South Cheung Chau) and in eastern waters (East Ninepins). These sites were established before EIAs were required and were operated on the basis of maintaining sufficient water depth over the disposal mounds so as not to interfere with vessel movements. Water quality monitoring and benthic surveying are undertaken to examine the impacts in the water column and on the sea bed.

Sequential bathymetric surveys have provided information on the mechanisms of consolidation, erosion and spreading which were taking place at the disposal mounds. The sites had not been selected specifically as retention or as dispersion sites, but the studies of the mounds showed a combination of the two characteristics. Mud which had been grab-dredged and placed by bottom-dumping barges retained sufficient strength to remain more or less where placed. Trailer-dredged material on the other hand was much more mobile and weaker (Evans, 1992).

Side scan sonar surveys showed large areas of slowly flowing mud slurry surrounding parts of the disposal sites and where continuous disposal in the same location had resulted in the formation of peaks of material. Sequential bathymetric surveys showed that submarine landslides occasionally occured. In one instance (Ng & Chiu, 2001) some 420,000 cubic metres of mud were dispersed in a submarine landslide that extended over a kilometre (Figure 6.8.6). Initiation of this failure could be traced to the destabilising effect of the swell associated with a tropical cyclone. In another case, a mound of more than two million cubic metres of trailer dredged material with side slopes locally at 1 in 50 (V:H) was completely dispersed by the passing of a severe cyclone. The same event had little effect on 1 in 25 (V:H) slopes of grab-dredged material (Evans, 1994b). Although these figures are large, they translate into relatively small levels of sedimentation and suspended sediment when viewed in the greater context of the surrounding marine area. An understanding of the sea bed processes involved is essential to good site management and efficient use of the disposal capacity.

Whenever possible, exhausted sand borrow pits are used for mud disposal. This practice provides an environmentally sound arrangement by which the dredged part of the sea bed can eventually be restored to its original, natural state. Furthermore, re-instatement of the original bathymetry serves to re-establish the local wave climate and general hydrodynamics. Another reason for backfilling some of the deep sand pits is as a marine safety measure so as to reinstate emergency anchoring capacity for ships. A special study involving anchor pulling trials in back filled mud was undertaken by Wong & Thorley (1992) to establish the holding ability of mud infill.

6.8.6 Beach Replenishment

Hong Kong's attitude to its beaches has changed dramatically during the last hundred years. In the early days, beach sand was removed for building purposes. This practice effectively ceased when the Sand Ordinance was enacted in 1935. Since then the recreational use of beaches has gradually become more popular and in the 1990s, major beach replenishment schemes were undertaken to re-sand and enlarge key beaches.

Viewed over the long term, beaches are dynamic but relatively stable systems which nevertheless display short-term cyclic patterns of sand erosion and accretion. In Hong Kong, where the tidal range is not great, it is the occasional and seasonal storm effects that produce the greatest changes.

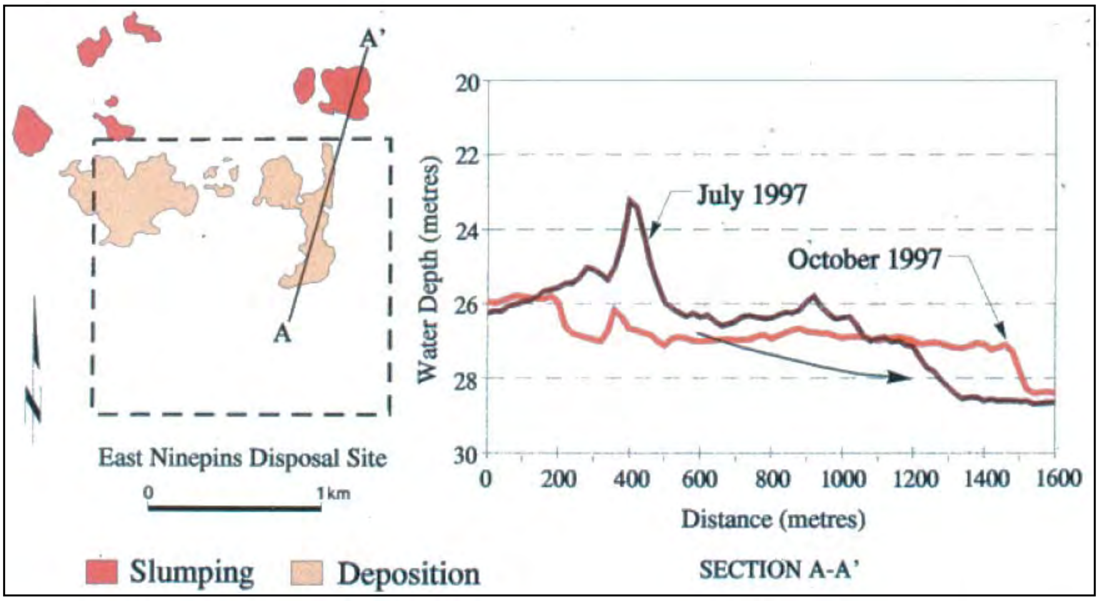


Figure 6.8.6 – Plan and section of large submarine landslide at East Ninepins mud disposal mound (Ng & Chiu, 2001)

Williams (1971a,b) undertook specific studies of the beach morphology at Shek O, Repulse Bay and Stanley Bay. Although spectacular changes in beach morphology can occur in just a day or two during a typhoon, the redistribution of sand is temporary and eventually, although this might be a period of many years after a particularly violent storm, the currents and wave action of the calmer periods restore the profile. Continuing sediment supply from the steep hinterland and coastal erosion tend to enhance the beaches.

Whilst in the long-term Hong Kong beaches are likely to continue accreting, the increasing popularity of beach-going means that more beach space is required and that it is unacceptable to wait for natural processes to reinstate beaches after periods of storm-related erosion. Beach replenishment projects have therefore been undertaken in a variety of settings around Hong Kong. Planning of a beach replenishment project requires a careful study of the beach morphology, both onshore and offshore, and an understanding of the local wave climate, distribution of sediment and its characteristics, as well as the historical records of the beach. There are different approaches to beach replenishment but that adopted for the replenishment of Repulse Bay on Hong Kong Island was to enhance the natural beach system by placing of sand and adding sand-retaining structures at either end of the beach to ensure that the arficially enlarged beach did not suffer sand loss by longshore transport (Davies *et al.*, 1996). Replenished beaches will continue to adjust their morphology long after the works are carried out and long-term monitoring is important.

Sand quality for beach replenishment is based on stability, comfort and aesthetics. Dredged alluvial sand deposits in Hong Kong are generally similar in colour to beach sand and so aesthetics is not an issue. On the other hand, stability tends to conflict with comfort. Fine sand, though most comfortable for beachgoers, will result in a gentle beach profile which will drain less well and will be subject to wind erosion. A balance between stability and comfort points to a mean grain size of 0.25 to 0.35mm, a fines content of <1%, and only minimal comminuted shell content, as suggested in the Port Works Design Manual, Part 5 (CED, 2002b).

6.8.7 Unexploded Ordnance

Encounters with unexploded ordnance from the Second World War have been a regular though infrequent occurrence in Hong Kong. Dredging works are particularly prone to this problem, which is particularly exacerbated by offshore disposal of ordnance in the past. These occurrences are dealt with on an ad hoc basis by the Explosive Ordnance Disposal (EOD) unit of the Hong Kong Police. While not a problem directly related to engineering geology, geophysical techniques have been used in an attempt to identify larger pieces. Also, larger pieces may have sunk into very soft marine clay or terrestrial pond deposits.

6.9 LANDFILL AND CONTAMINATED LAND

6.9.1 Introduction

The importance of engineering geology in identifying, characterising and overcoming potential problems related to landfills and contaminated land varies significantly from project to project but can be summarized generally as:

- understanding the geology of the site, in particular the hydrogeology prior to development,
- identifying changes that might have occurred to the groundwater regime affecting the site since its development,
- for contaminated sites, identifying the site's land use history, particularly any industrial use, and cataloguing potential contaminants,
- based on the geological model and site history, planning, undertaking and interpreting ground investigations to characterise the site geology, hydrogeology, the type and distribution of contaminants, and subsurface pathways of contaminant migration, and
- identifying practical design elements to be incorporated into new works to mitigate against the identified contaminants.

The range of information that may be required, and ground investigation methods which are sometimes used, are laid out in the UK's Environment Agency website (2004) and elsewhere as described below.

The study should also consider the natural geochemistry of the area in case any local anomalies are present. Sewell (1999) provides individual maps of the natural concentrations of the main elements for the whole of Hong Kong.

6.9.2 Landfills

Potential Hazards

Much of the material in landfills is biodegradable. Some relatively inert material is also present, including layers of cover material. The two potentially hazardous products of chemical and biochemical degradation of landfill waste are leachate and gas. In modern containment landfills, leachate is collected from the sealed base of the landfill, whilst the gas is intersected by wells and used or vented in a controlled manner.

Modern Containment Landfills

Waste disposal is currently undertaken at three strategic landfills SENT, NENT & WENT. These have been designed and constructed, with considerable engineering geological input, as containment facilities which incorporate multilayer composite liner systems to cover the entire surface area of each site. Surface water is managed with a conventional surface drainage system. Groundwater drains away from the site via a drainage layer below the base liner preventing hydrostatic build-up below the liner and avoiding contact of groundwater with leachate. Monitoring includes checking groundwater from this drainage layer to verify lack of contamination by leachate.

The strategic landfills are expected to reach capacity between about 2013 and 2017 and so new facilities will be required. Studies for similar landfills will require engineering geological input in site characterisation and investigation, particularly to identify the local groundwater regime, to identify subsurface pathways which could be followed if the containment system fails, and to monitor these pathways and identify remedial measures should this occur.

Restoration of Old Landfills

The restoration of closed landfill sites is typically restricted to developing them into various types of open-air recreational facilities, due to their potential for settlement and production of poisonous or flammable gases. The closed landfills can be divided into those which occupy inland valleys, and those located at coastal sites where the waste is retained within reclamation. The engineering design of the old landfills varied. The more recent ones were constructed with synthetic liners whereas earlier ones were unlined or had liners of natural, low permeability material. Included in the former category are coastal landfills, such as the Tseung Kwan O Stage 1 landfill, where refuse was placed directly into the water and onto the sea bed mud behind an exclosing seawall (Insley *et al.*, 1992; Blower *et al.*, 1993).

Site investigations of old landfills are generally focused on characterising contaminants and determining whether they are able to migrate out of the landfill site, and on providing information on which to base the design of gas and leachate collection systems. Powell *et al.* (1992) give a detailed account of the investigation and restoration of the Sai Tso Wan landfill. This landfill was originally formed in the late 1970s when the surrounding area was essentially undeveloped. The waste was placed in a natural valley, the head of which had been enlarged by quarrying.

Further groundwater and leachate observations were reported by Yim & Chan (1987). The entire landfill is underlain by fresh, fine- to medium-grained granite with widely spaced sub-vertical joints aligned along the valley and a series of sub-horizontal ones generally sub-parallel to the original topography. A variety of leachate containment layers were placed prior to infilling with refuse. The site investigation included careful field examination of the site and adjoining areas during which small traces of leachate were found to be seeping from nearby rock joints. The investigation also included a study of landfill gas production and migration. By chance, a landfill gas study had also been carried out only a few years earlier as part of the planning for a new railway station development adjacent to the landfill. Such studies are required for all new developments within close proximity to landfills as described later in this section. The findings of this earlier study were augmented by additional investigation and it was concluded that although gas generation was relatively active, most vented freely through the site surface and that there was little off-site migration.

The remedial works design incorporated a system of landfill gas collection wells (Nash & Tsang, 1992) and a 3 m deep vertical, impermeable barrier to reduce the risk of gas migration at one end of the site. The stability of the waste was improved by cutting back the slope and adding a concrete-retained toe weight, and surface drainage.

For discussion of a similar investigation, but for a coastal landfill site, the reader is referred to the account by Insley *et al.* (1992) of the investigation of the three stages of the Tseung Kwan O Landfills.

Developments Near Landfills

Development in close proximity to a landfill site is potentially at risk from the lateral migration of landfill gas (EPD, 1996, 1997). In general, an assessment of the risk is required for any development which is proposed within a 250 m "Consultation Zone" around a landfill site. All the landfills in the Territory and their associated Consultation Zones are delineated on plans held by EPD.

The types and sources of data which should be considered for the assessment are listed in EPD (1997). The assessment commences with a preliminary qualitative assessment based on a Source-Pathway-Target model, the source being gas or gas dissolved in leachate or groundwater, and the target being the buildings, manholes, etc., where gas could be released or accumulate. The engineering geological input is concentrated primarily on examining the pathways, particularly identifying any high permeability layers in the ground, and also discrete linear pathways such as may be provided by open joints, etc.

The results of the preliminary qualitative risk assessment will determine the level of additional ground investigation required to undertake a detailed qualitative risk assessment which, in turn will identify what protective measures, if any, need to be incorporated into the new development. It should be noted that during a ground investigation, gas from other sources unrelated to the landfill in question may be encountered. Such sources include natural accumulations of methane and hydrogen sulphide from organic rich sediments such as are found in some marine deposits, as well as natural sediments contaminated by human activities. In addition to normal ground investigation methods, specialist sampling and monitoring of groundwater and gas will normally be required when gas is expected to be encountered. Some guidance is provided in EPD (1997) which also cites more detailed sources of information including Hooker & Bannon (1993) and Crowhurst & Manchester (1993).

During ground investigation and later during the construction of the development, particularly within excavations and in enclosed spaces in direct contact with the ground, special safety procedures are required so as to minimize risk of fire, explosion, asphyxiation and toxicity effects (British Drilling Association, 1993).

The design of any protective measures to be incorporated into the new development will be very dependent on developing a robust model of the subsurface distribution and movement of any landfill gas.

6.9.3 Contaminated Land

General

Contaminated land usually refers to land which has been polluted by hazardous substances released into the ground, either deliberately or accidentally during a variety of industrial activities. Such pollutants can diffuse through the soil and rock and can be carried by groundwater to locations sometimes quite distant from the original source of pollution. Historically, typhoon anchorages and other enclosed coastal sites near built up areas have accumulated a wide variety of waste both domestic and industrial. Where reclamation has covered these marine areas, the resulting land is commonly contaminated at depth.

EPD (1994) sets out requirements for assessment of contaminated land sites and provides guidelines on how site assessments should be carried out. It also suggests types of remedial measures that can be adopted for cleaning up such sites. The type of contamination can vary widely but from an engineering geological perspective, key importance is attached to understanding the site hydrogeology, in particular the pathways by which contaminants and contaminated groundwater may disperse. This could involve many different factors such as identifying aquifers and aquicludes, phreatic and piezometric surfaces and their seasonal variation, presence of any fissure flow or preferential pathways, permeability and storage of saturated zones, sinks and discharge points, groundwater geochemistry, etc.

EPD Procedures and Guidelines

The nature and extent of potential problems are very site specific. The example provided by sites previously used for petrol filling stations, boatyards and vehicle repair/dismantling workshops serves as an illustration of how, in general, problems of contaminated land can be addressed. EPD (2002) gives detailed guidance on the procedures and technical requirements for investigating and assessing such sites.

Investigations
Contamination should always be regarded as a possibility when dealing with a site previously occupied by or close to industrial premises. A detailed desk study, including the examination of old maps, is key to providing information as to previous land usage.

The level of site investigation required on a particular site will depend on the likely severity of the contamination both in terms of contaminant types and amount of contamination. Some substances, particularly petroleum hydrocarbons, can float on top of groundwater and migrate some distance away from the sources of leaks or spillage. Selection of remediation methods also depends on geological factors, e.g. low permeability soil may not be amenable to soil venting.

The monitoring of gases during the investigation is good practice, in addition to mandatory tests for other contaminants. EPD (2002) provides guidance on the types of contaminants which might be expected from different site usages, as well as guidance on designing and undertaking ground investigation works to identify contaminants and their distribution.

Contaminated ground may also occur in areas with different site uses to those specifically identified by EPD. These may include sites on reclamations over old typhoon shelters which may be hydraulically connected to potentially organic-rich deposits or other ground in the vicinity of sewers and gas pipelines. A thorough desk study and ground investigation will be necessary, with the main focus being to:

- determine the site history,
- identify all potential contaminant sources,
- establish former and present natural drainage patterns,
- determine the present groundwater regime, soil and rock permeability,
- identify the nature, distribution, concentration and source of contaminants within the soil and rock and within the groundwater, and
- identify potential pathways for migration of contaminants.

The design of any clean-up works will be dependent on developing a robust model of the subsurface distribution and potential movement of identified pollutants. The decommissioning of the Cheoy Lee Shipyard at Penny's Bay involved a very extensive ground investigation to identify the groundwater pattern, to sample groundwater and soils so as to test for contaminants, and to map out the concentrations of these contaminants (MCAL, 2002).

6.10 ASSESSMENT OF NATURAL RESOURCES

6.10.1 Introduction

Natural resources extracted in Hong Kong for use in civil engineering and building projects comprise quarried hard rock and dredged offshore sand. Large quantities of soil and rock have also been extracted as part of site formations (Section 6.3).

The planning and operation of quarries require knowledge of the geological structure and weathering processes whilst offshore sand exploration and extraction require an understanding of the palaeo-environments and sedimentary processes as discussed in Section 5.8. The environmental aspects of utilising these resources are discussed below as they can often be the controlling factor in determining the viability of using a particular resource. For example, an understanding of engineering geology such as distribution of sand bodies and overburden, provides the vital information needed to understand and minimise the impact to the marine environment during dredging.

The discussion on rock as a resource excludes rock excavation considerations covered in Section 6.3, and the discussion of offshore sand is complementary to much of the discussion on marine works included in Section 6.8 and superficial deposits in Section 5.8. The discussion on offshore sand and related depositional environments draws on the information in Fyfe *et al.* (2000).

6.10.2 Hard Rock

Sources
In the 1970s Hong Kong produced almost all its own crushed rock for concrete, roadbase and seawalls with the bulk of material coming from quarries and limited amounts from site formations. As use of crushed rock increased, demand was satisfied by fewer and larger quarries in Hong Kong (Choy *et al.*, 1987) and an increasing proportion of rock imported from Mainland China. By 2004, the three remaining quarries in Hong Kong produced about

one third of total demand (GEO, 2004l). These quarries are located at Shek O, Anderson Road and Lam Tei and are expected to cease operation by 2009, 2013 and 2015 respectively. All three are operating rehabilitation contracts designed to leave attractive landforms when rock extraction is completed (Lam & Siu, 2002). The planning background to the rehabilitation of Anderson Road quarry is discussed in Fowler (1990). Current planning in Hong Kong does not include development of any new quarries when the existing three quarries are closed.

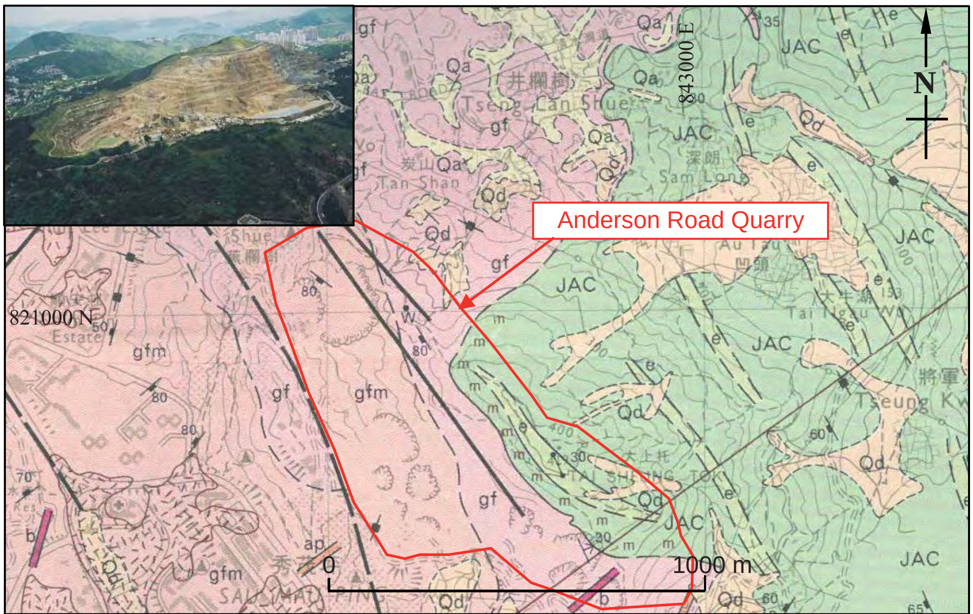
Because of the increasing dependence on imported rock products, optimising the use of Hong Kong's own resources is important. Consideration of rock by-products from site formations for use as aggregate, etc., may therefore increase in importance.

Concrete Aggregate

In order for rock to be suitable for concrete aggregate it should be strong and durable, lack directional fabric and be capable of yielding equidimensional particles when crushed. In Hong Kong's quarries, granite is currently used, almost exclusively, as concrete aggregate (although some volcanic rock was used in the past). At Lam Tei Quarry, the rock is a fine-grained granite, which is locally porphyritic, at Shek O Quarry the rock is a medium-grained granite, and at Anderson Road Quarry the rock is a fine- to medium-grained biotite granite (Sewell *et al*., 2000). The combination of geology and topography means that Anderson Road Quarry also produces some fine ash crystal tuff. Figures 6.10.1, 6.10.2 & 6.10.3 show the quarry locations superimposed on the 1:20,000-scale published geological maps.

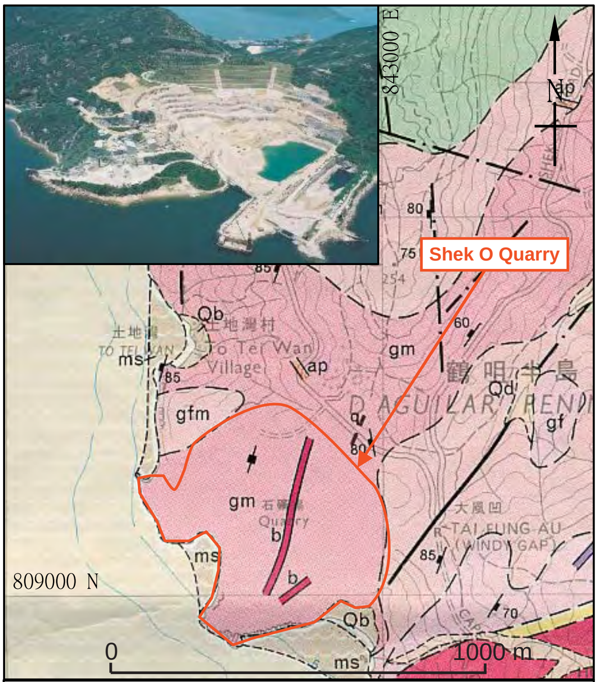
Whilst these particular sources have been used for many years, and although the performance of the aggregate in concrete is very well established, the aggregate produced is nevertheless regularly inspected and tested. Engineering specifications are included in the General Specification for Civil Engineering Works (CED, 2002a) which lays down required ranges of properties governing aggregate strength and durability, which are related to the materials origin and subsequent geological processes. The literature on the testing of aggregate properties is specialized and extensive but a summary is given by Smith & Collis (1993), including a comprehensive list of related British and American Standards.

Irfan (1994a) summarizes the results of laboratory aggregate tests on Hong Kong granites and discusses the relative suitability of the different granites for use as concrete aggregate. He concludes that while all the quarried granites are suitable, the fine-grained



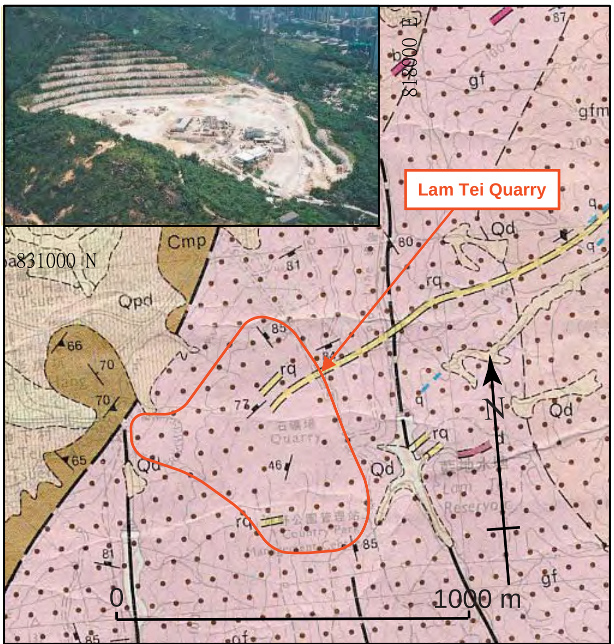
Legend:
gf - Fine-grained granite
gfm - Fine- to medium-grained granite
JAC - Fine ash vitric tuff
ap - Aplite
Qa - Alluvium
Qd - Debris flow deposits
b - Basalt
e - Eutaxite

Figure 6.10.1 – Geological setting of Anderson Road Quarry



Legend:
gm - Medium-grained granite
gfm - Fine- to medium-grained granite
gf - Fine-grained granite
Qd - Beach deposits
ms - Marine sand

Figure 6.10.2 – Geological setting of Shek O Quarry



Legend:
gf - Fine-grained granite
Cmp - Phyllite, meta-siltstone with meta-sandstone and graphite schist
rq - Quartzphyric rhyolite
Qd - Debris flow deposits

Figure 6.10.3 – Geological setting of Lam Tei Quarry

granites are superior in mechanical properties. Smith & Collis (1993) also note that, in general, coarse-grained igneous rocks tend to fracture along the natural cleavage and flaws of the crystalline components.

In addition to the mechanical properties, the chemical stability of the composite mineralogy is important. Petrographic examination is used for the idenfication of alkali aggregate reaction products and the standard procedures are outlined in ASTM (1996). Alkali-aggregate reaction (AAR) occurs when minerals in certain aggregates react with the soluble alkaline components of the cement paste. Two main forms of alkali-aggregate reaction have been recognised: alkali-silica reaction and alkali-carbonate reaction.

In Hong Kong most alkali-aggregate reaction is between alkaline components in the cement paste and reactive silica-bearing minerals in the accompanying aggregate. With respect to Hong Kong's granitic rocks, whilst AAR has occasionally been documented (e.g. Sewell & Campbell, 2001), it is not a major concern, and may be related to localised alteration zones. With reference to alkali-silica reactivity, despite the occasional presence of microcrystalline silica, the quartz is generally not present in a form that results in reaction with the alkali hydroxides from the cement. Nevertheless, some restriction on alkali content of concrete used below ground is often specified (see BD, 1995).

With respect to volcanic aggregates, Kwan *et al.* (1995) conclude that although in strength terms volcanic aggregate is superior to granite aggregate, the former has poorer shape, being flatter and more elongate, and has a higher fines content. The presence of micro or crypto-crystalline silica such as volcanic glass is a consideration when using volcanic rock as concrete aggregate. Poole (1994), Leung *et al.* (1995) and Liu & Tam (2004) examined the alkali aggregate reactivity of Hong Kong volcanic tuff and concluded the material to be potentially reactive. Most documented cases of alkali-silica reaction in Hong Kong have been related to the presence of volcanic aggregates from Mainland China (Sewell & Campbell, 2001).

If surplus rock from site formations and other excavations is to be used as concrete aggregate or other rock products, engineering geological mapping and more detailed testing and assessment are likely to be required during the site investigation stage. Variations in rock type and the presence of any modifying features such as faults and shear zones, differential weathering, etc., need to be carefully examined, characterised and delineated. In addition, petrographic examination may need to be used to identify whether micro- or crypto-crystalline quartz is present. Not only have the material properties to be assessed but also the continuity of the rock mass and its potential ability to yield the uniformly suitable material required in concrete mix designs.

Other Uses

Rock is also widely used in marine works as armour stone, pell-mell and filter for constructing seawalls and breakwaters, in reclamations and site formations as a general fill material, and as roadbase, bituminous material, railway ballast and filter material. In essence, fresh, strong durable rock is required. This tends to mean fresh igneous rock that is crushed and sized according to desired use. A variety of engineering specifications lay down the various requirements and reference should be made to these for individual specific needs. For example, Poole (1991) studied the potential for an area on northeast Lantau to yield armour stone by examining jointing patterns in outcrops through computer simulation of the fragmentation accompanying blasting. Related considerations were included in Yip's (1991) geotechnical assessment of the same area as a source of rockfill. Volcanic rock as well as granite is well-suited to such uses (Poole, 1991). Volcanic aggregate is also superior to granite for use in asphalt.

An important consideration, especially for larger excavations where considerable volumes of material may be involved, is a proper estimation of the bulking factor which determines the generated volume of fill per cubic metre excavated. Plant *et al.* (1998) report that bulking factors for fresh as-blasted granite rock averaged 1.34, while completely and highly weathered granite with occasional corestones had a bulking factor of 1.0.

6.10.3 Fill Material

Onshore Sources

Until the mid-1950s, excavations of weathered bedrock and colluvium provided most fill material needed to form reclamations in Hong Kong. Excavation into hillslopes and the complete removal of small hills was a process that worked well because the excavated land also provided sites for development. Historically, the decomposed bedrock so common in the Hong Kong Island-Kowloon area provided a mix of general fill material and good rock which could be used for breakwaters and seawalls.

Completely decomposed granite is suitable as general fill material for reclamations provided it has a plasticity index not exceeding 12, i.e. the clay content is not too high. Where decomposed granite fill is used as a founding layer for seawalls, its suitability depends on factors such as grading, plasticity index, permeability, coefficient of consolidation and the available construction programme. These factors will need to be assessed before deciding if the material to be excavated is suited to the proposed use. Further considerations are given in Choot (1993a).

Offshore Sand

Offshore granular deposits have been extracted for many years in Hong Kong, both for use as reclamation fill and as building sand. Early sources were generally near-shore areas where hydrodynamic conditions left coarse material exposed on the sea bed, or at drainage outfalls where granular material was deposited. Dredging of marine sand declined as deposits on the surface of the sea bed were used up. However, towards the end of the 1980s the pattern of the offshore geology and the potential sub-sea bed sources of sand became better understood.

Between the late 1980s and early 2000s, Hong Kong completed a series of major reclamations which required an amount of fill material considerably in excess of the entire volume of fill material used in its previous history (Brand *et al.*, 1994). Sand extraction continued after 2000 and Figure 6.10.4 shows the location of the borrow areas and the volume of sand extracted.

Two early projects, Tin Shui Wai (Dutton, 1987) and Container Terminal 6 (Wragge-Morley, 1988), highlighted some important engineering geological factors which have to be taken into account during investigations to locate economic fill resources:

- Sand sources below present-day tidal channels have little or no mud overburden.
- The overflow process, so essential to the economic filling and operation of trailer hopper dredgers, significantly reduces fines content of the sand delivered to site compared with the fines content of the sand *in situ*.

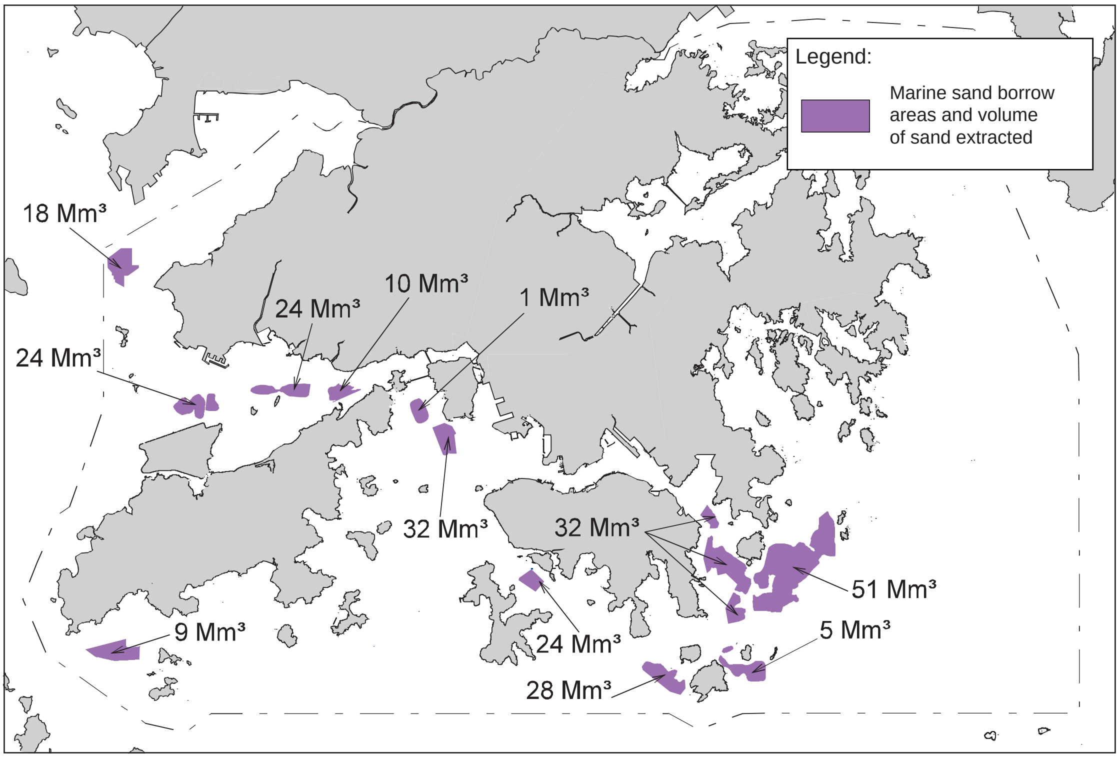


Figure 6.10.4 – Location of marine borrow areas and volume of sand extracted (CEDD, 2005)

- Plumes of suspended sediment settle rapidly to the sea bed and the environmental impacts are localised and short-term (Holmes, 1988).
- Coarse and angular sand causes significant wear in pipelines and pumping plant.

Systematic Sand Exploration Programme
Fyfe *et al.* (2000) and Section 5.8 include accounts of the geological setting and characteristics of the Pleistocene alluvial sands and early Holocene marine sand deposits which make up the offshore sand resources. The various stages of the offshore geological investigations and later sand exploration programme undertaken by the GEO have been documented in Choot (1988), Thorley *et al.* (1990), Whiteside & Cheung (1996) and Martin *et al.* (1997).

A territory-wide systematic sand exploration programme was carried out in two distinct stages (Whiteside & Massey, 1992). One of the critically important factors in both stages was extensive local experience in undertaking high resolution marine geophysical surveys (Ridley Thomas *et al.*, 1988). The first stage of fieldwork consisted of an approximately 3 km grid of seismic reflection. The seismic survey was complemented by a series of vibrocores and continuously sampled drillholes. This first stage investigation enabled the geological model to be refined and selected sand bodies to be targeted for more detailed investigation.

The second stage comprised further seismic reflection surveys, but on an approximately 350 m grid accompanied by continuously sampled drillholes and piezo-cone penetration tests (BCL, 1991-93). The continuous sampling sequence, repeated every two metres, comprised U100, then bulk sample, then SPT and then bulk sample. The relative density of sands dredged to date has generally been in the loose to medium dense range and not posed any problem for trailer dredgers, but an adequate number of SPTs is important in order to enable tendering dredging contractors to make their own assessment of new sand bodies. On average about ten drillhole / CPT stations were used per square kilometre. Drillhole and other investigation data such as particle size determinations, much of it supplied in digital format by the site investigation contractors, were stored on a Geographic Information System (Selwood & Whiteside, 1992) from which they could also be exported to a personal computer for 3-D modelling.

Dredging Considerations and Reserve Calculation
The relative importance of the different geological factors (PSD, relative density, strength and

consistency, layer thicknesses, etc.) can only be determined from a detailed assessment. Digitised, interpreted seismic sections with assigned dredging parameters based on a synthesis of the drillhole, CPT and laboratory data are used to construct a model of the area for reserve calculation and borrow area development design and planning. The process of working from a geological model, which describes a geological resource, to a dredging model, which describes a fill reserve, is described in Selby & Ooms (1996). General considerations in distinguishing between resources and useable reserves are discussed by Addison *et al.* (1988).

In economic terms, dredging is a plant-intensive operation, i.e. material and manpower costs are relatively insignificant whereas the daily cost of the dredger is high. The overall cycle time for each trip from borrow area to reclamation site and back plus 'un-productive' time spent removing and disposing of overburden are the most important parameters in determining the economics of a trailer dredging operation. For this reason, although the overburden ratio is of critical importance, there is no fixed value above which a deposit becomes uneconomic because it has to be considered in conjunction with the distance to the reclamation site and the distance to the overburden disposal site. For example, there was about 15 m of mud overburden in the East Sha Chau borrow area used for the airport reclamation but it was still economic because the area was immediately adjacent to the reclamation site.

The time taken to load a trailer hopper dredger is very dependent on the grain size of the sand and on the fines content. In the overflowing process, particularly in the latter stages when the hopper is nearly full, loss of fine sand is unavoidable although undesirable from an economic dredging perspective. In general, the finer-grained the sand being dredged, the longer time it takes to load, and hence the more costly is the sand delivered to site.

The cohesive strength of the overburden and the density of the sand deposits affect the slope angle necessary to maintain stability in the sides of dredged pits. The flatter the side slopes in overlying mud, the higher the overburden ratio becomes. The flatter the side slopes in the sand, the less sand can be extracted at any given depth for a particular sea bed plan area. These factors, and any other navigational and environmental constraints, have to be taken into account in calculating the volumes of economic sand reserves which can be extracted from a given geological resource. In estimating the volume of sand reserves available in a particular borrow area assumptions also have to be made about the depth which dredgers will be able to reach. To date, most borrow areas have been dredged to depths of between 35 m and 40 m, and in one case, dredging for Container Terminal 8, the use of a submersible pump part-way down the suction pipe enabled sand to be dredged from a depth of 54 m (de Kok, 1994).

Figure 6.10.5a shows the plan layout of the second stage site investigation at West Po Toi and Figure 6.10.5b is a cross-section through the borrow area showing the sediment layers defined for dredging purposes. This section, which is very similar to that which would be drawn on a purely geological basis, shows that the bulk of the sand resource comprised alluvial sand, either as originally laid down during the Pleistocene or as a layer re-worked during the early part of the Holocene transgression. The alluvial sand is dominantly fine- to medium-grained subangular quartz with about 25% fines content, while the re-worked alluvial sand tends to be subangular to subrounded, with about 5% fines content and contains comminuted shell fragments (Figure 6.10.6). Despite the presence of sand layers in the dominantly clayey interburden, these could not be dredged preferentially and the unit therefore had to be dredged and taken for disposal. This particular borrow area also demonstrates the fact that when sand resources are worked in a series of pits rather than one single pit, significant volumes of sand can be sterilised within the separating bunds if the earlier pits have already been backfilled.

Characteristics of the Sands used as Fill

Most of the offshore sands dredged for use as fill material are alluvial quartz sands derived predominantly from nearby weathered granitic bedrock. Distance of sediment transport controls grain angularity and sphericity, although some of the upper layers of the alluvial sands which were tidally reworked by the early Holocene sea can be more rounded.

By their very nature, alluvial sand deposits are finer grained the further they are from the original material source. There is therefore a tendency for the sand in the more distant borrow areas to be finer and to have a higher fines content. This factor makes it difficult

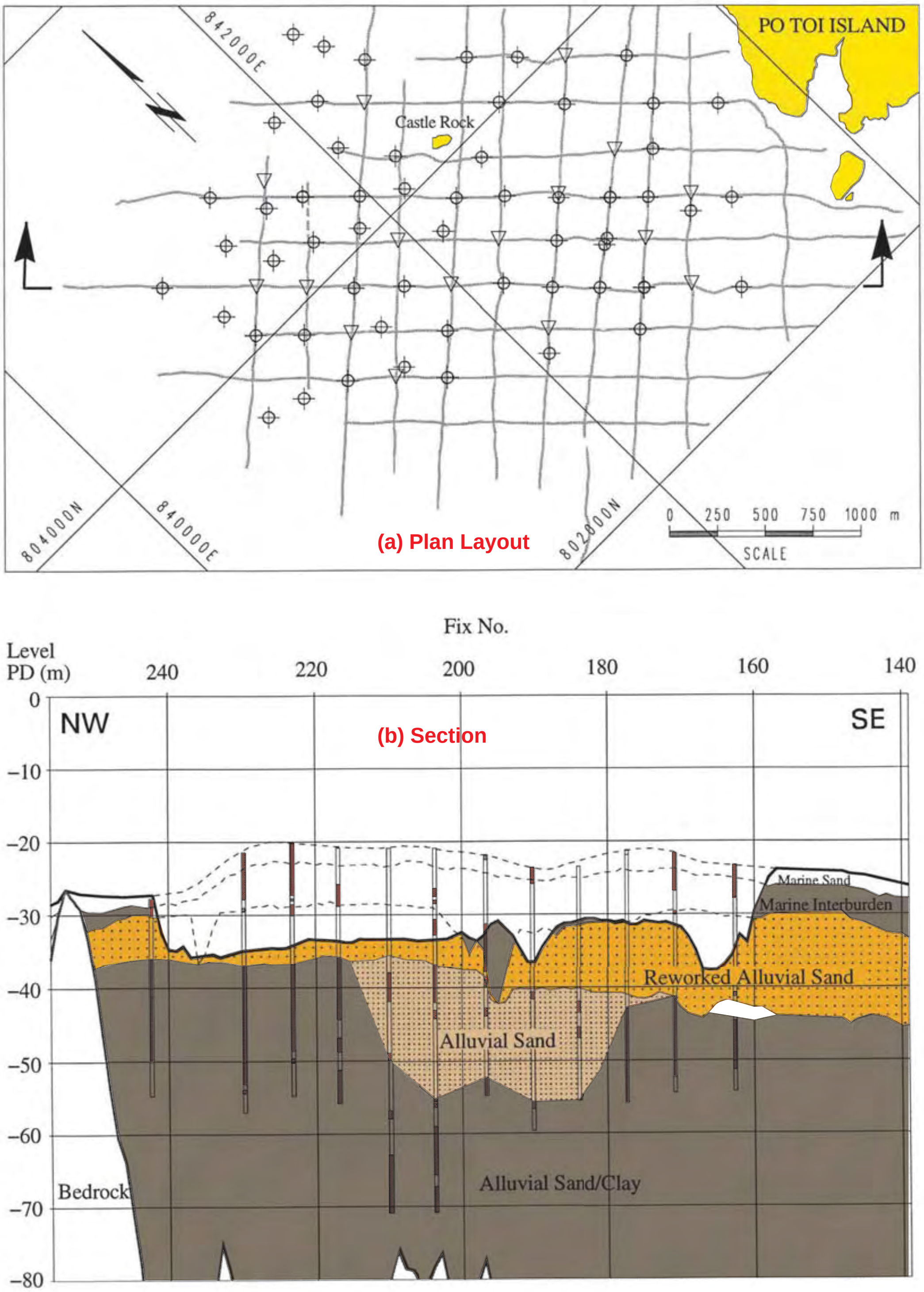
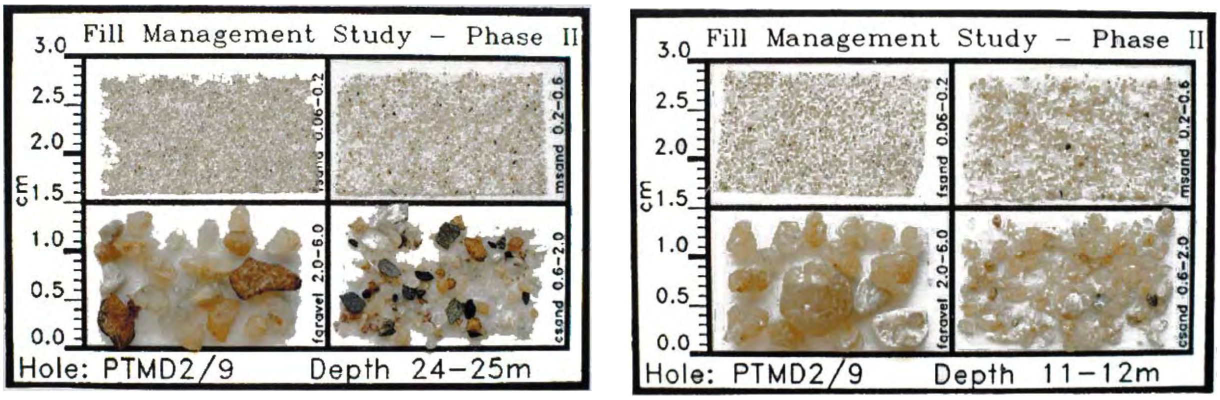


Figure 6.10.5 – Plan layout and section through the West Po Toi marine borrow area (after DEMAS, 1996)

to optimise the loading of the hopper so as to achieve the maximum rate of sand delivery at the reclamation site (Whiteside *et al.*, 1998). Long over flow times can also result in siltation of the sand dredging site which then has to be cleared and the silt taken for disposal.

Sand from Outside Hong Kong

Since the early 1990s, approximately 25% of the demand for sand fill in Hong Kong has been met by sand imported from Mainland waters (CEDD, 2004). Some 40 million cubic metres of alluvial and re-worked alluvial sand came from an area near Wai Ling Ding island which had been investigated as part



Alluvial Sand

Reworked Alluvial Sand

Figure 6.10.6 – Photograph of alluvial sand and reworked alluvial sand from West Po Toi marine borrow area (BCL, 1991-93)

of the Hong Kong territory-wide sand exploration programme. Geologically, this sand deposit is similar to those in Hong Kong waters but it is somewhat finer-grained, being a more distal deposit. As discussed above, this characteristic results in more difficult dredging conditions. Lesser volumes of low-fines, Pleistocene age, alluvial medium to coarse quartz sand have also come from the eastern side of Mirs Bay. A quite different geological setting in the area just south of Hu Men in the Pearl River estuary, has also produced large volumes of easily-extracted, low-fines quartz sand. Here, the Pearl River opens out into the estuary proper and as a result of the reduced current speed, large volumes of granular bedload are deposited. The depositional conditions also prevent accumulation of any fine-grained overburden and large volumes of this easily extracted sand have been imported into Hong Kong for use as fill material.

Specifications for Sand Fill

Kwan (1993) noted that specifications for sand fill designed to be used in similar engineering situations ranged from <10% fines content to <35% fines, with a tendency for faster-tracked, more critical contracts to be in the range <10% fines to <20%. However, almost all the sand was trailer-dredged, so achieving the specification posed no problems because the overflow during hopper loading reduced the fines content significantly. An exception, however, arose with some very small contracts that used grab-dredging to extract the sand because this method resulted in very little fines loss.

Sand for Beach Replenishment

Beach replenishment contracts have been undertaken using Hong Kong dredged sand. While the volumes involved are minimal compared to the amount of material used in reclamations, these are high profile projects and certain characteristics of the sand are more important than in general fill for a reclamation. Layers of sand which have suitable characteristics for beach replenishment (See Section 6.8) exist within the re-worked alluvial sands near Hong Kong. These sands were intertidal 'beach' deposits during the early Holocene. However, despite careful logging of drillholes and precise delineation of these deposits at the planning stage, selective dredging of such layers can be difficult.

6.10.4 Environmental Considerations

Quarrying

The potential problems caused by dust and noise (and the safety issue of fly-rock) are similar to those which have to be avoided in site formations and reference should be made to Section 6.3. The main environmental issue relating to quarries is that of aesthetics and beneficial after-use of the site. These issues have largely been addressed already in that all three quarries are working to pre-determined profiles designed to add to the environment when quarrying is complete (Lam & Siu, 2000).

Sand Dredging

Impacts During Different Stages of Extraction

Extraction of offshore sand deposits has the potential to cause adverse environmental impacts in several ways and each of these has to be assessed, normally through the environmental impact assessment (EIA) process. Such assessments require knowledge of the physical and chemical characteristics of the sediments to be dredged, knowledge of the marine environment, its processes and ecosystem, understanding of how and to what extent the dredging process will release material into the marine environment, and predictions of the dispersion and fate of released material and how it may affect marine life. Environmental aspects related to the placing of sand at reclamation sites are covered in Section 6.8. Stages of the extraction works requiring environmental assessment are:

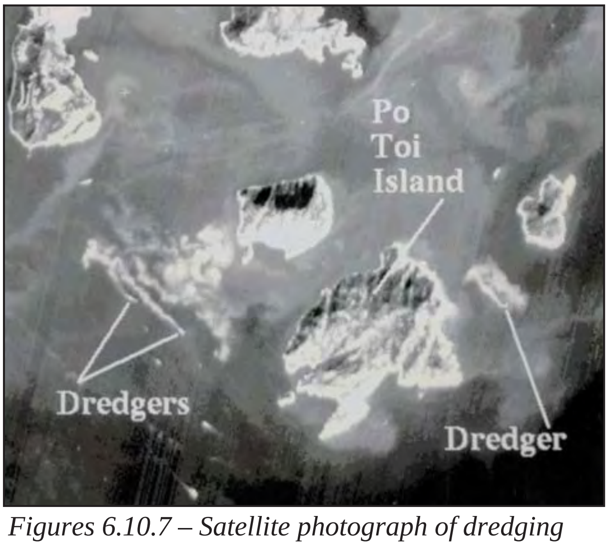
- Removal of mud overburden – release, dispersion and potential effect of sediment; nature of any contaminants present and fate of any released; loss of sea bed habitat.
- Disposal of mud overburden - sediment release during placing of mud; colonisation of disposed material by benthic organisms after placing.
- Dredging of sand - release, dispersion and potential effect of overflow material; effects of the formation of sea bed pits on local hydrodynamics.

Comprehensive EIAs were undertaken for sand dredging in Mirs Bay in eastern waters (1992) and in East Lamma Channel (1993) and the reader is referred to BCL (1992,1993) respectively for full details.

Studies of Impacts from Dredging

Reliable predictions of dredging impacts require not only a thorough characterisation of the sand *in situ* and of the sediment that will be released during dredging but also an understanding of the local oceanography, hydrodynamics, physical and biological sea bed processes and the marine ecosystem. Figure 6.10.7 is a satellite photograph showing dredging plumes around the Po Toi Islands, one of the areas where detailed studies of dredging plumes were undertaken. Detailed studies on the physical and ecological nature of the local marine environment are reported by Evans (1994a), Ng *et al.* (1998) and Ng & Chan (2004). Some main conclusions regarding the impacts of sand dredging are as follows:

- On the flattish muddy sea bed, grab sampling, faunal analysis, and innovative sea bed camera surveys (Germano *et al.,* 2002) demonstrated that the sea bed conditions are naturally very dynamic and that the ecosystem coped with all but the most intense sedimentation that occurred immediately adjacent to borrow areas – and even in these areas, colonisation started taking place immediately after cessation of dredging. Leung & Morton (2000) also similarly reported that dredging impacts to the marine ecosystem were short-lived and localised.
- On the rocky coastal sea bed, extensive dive surveys (BCL 1995a,b) identified some areas immediately adjacent to sand dredging where intensive sedimentation had smothered some soft and some hard corals. However, in other areas where dredging-related elevations of suspended sediment levels had been recorded, surveys indicated that high levels of suspended sediment did not result in coral mortality *per se*.
- Individual dredging plumes were studied intensively using satellite and aerial photography and multi-boat monitoring with water samplers, siltmeters and acoustic doppler current profilers (Land *et al.,* 1994). Field results were also compared to computerised hydrodynamic modelling predictions which were made using the actual dredging and overflow parameters. Results of plume monitoring at West Po Toi marine borrow area (Whiteside *et al.,* 1995) showed that in the first five to ten minutes after overflow, the behaviour of the sediment-water mixture is dynamic and the bulk of the material moves to the sea bed as a density current. Part of this density current, however, is entrained in the water column and forms a plume of suspended sediment that gradually decays as individual particles settle under gravity. Depending on the characteristics of the sand being dredged and the method of dredging, the plume almost completely decays to background level after about two hours. In a study of sand dredging plumes in the coarser sands of East Lamma Channel, Cheung & Ho (2004) report decay to background levels in one and a quarter hours. Studies have also shown that computerised modelling of plumes can give reasonably reliable predictions of plume movement and decay with a settling velocity of 1 mm/sec (Whiteside & Rodger, 1996). Also, Parry (2000) analysed continuous



Figures 6.10.7 – Satellite photograph of dredging plumes around the Po Toi Islands

monitoring siltmeter data spanning a period of sand dredging and passage of a typhoon and demonstrated that natural variations in suspended sediment can be an order of magnitude greater in level and duration than those caused by dredger overflow.

If unacceptable impacts are predicted, it is possible in some cases to design mitigation measures so as to reduce impacts to acceptable levels. Such measures have included restricting dredging to certain daily or seasonal tide or wind conditions, and the imposition of daily dredging quotas.

Sea bed Reinstatement

In order to reinstate the sea bed to its original condition after extraction of sand, Hong Kong has for some time been using exhausted marine borrow areas for disposal of uncontaminated dredged mud. Because most deposits of sand originally had a cover of mud overburden, such backfilling with marine mud reinstates both bathymetry and sediment type. Studies have shown that colonization of backfilled mud by benthic organisms commences immediately after placing, and after only a few years, communities similar to adjacent undisturbed sea bed are established (e.g. Qian, *et al.*, 2003). From an engineering perspective, it has to be borne in mind that the engineering properties of backfilled mud are different from those of undisturbed, *in situ* mud and therefore the presence of a backfilled borrow pit could constitute an important constraint to future sea bed development such as construction of an immersed tube tunnel. Such factors required special design considerations for the laying of a natural gas pipeline past the old backfilled borrow pit used to source sand for the Tin Shui Wai New Town project.

6.10.5 Disused Mines

General

Hong Kong has a varied history of mineral resource extraction ranging from small quarrying pits to extensive underground mines. Old mines and associated spoil can result in surface instability, pose problems to the driving of new tunnels (see Figure 6.10.8), discharge mineral enriched groundwater and require identification and assessment in natural terrain hazard assessments (e.g. HCL, 2001).

Overviews of the occurrences and past use of mineral deposits are given by Sewell (1999), Sewell *et al.* (2000) and Fyfe *et al.* (2000). The metalliferous and

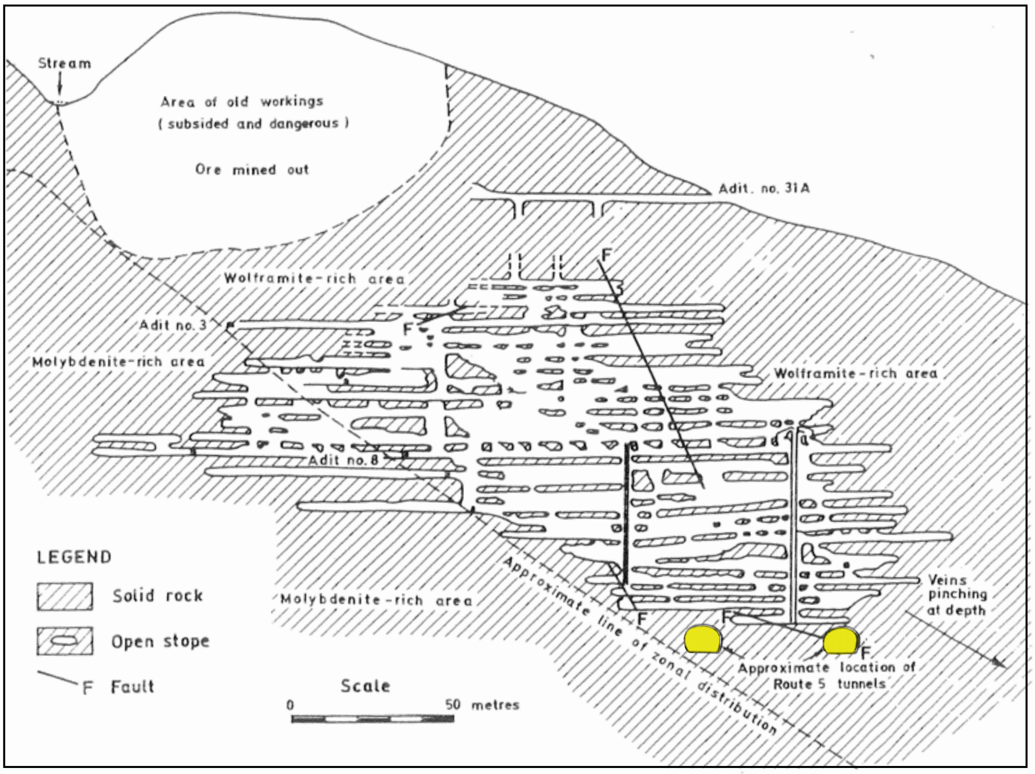


Figure 6.10.8 – Cross-section showing extent of underground mining and old subsided near-surface workings of the Needle Hill wolframite mine (after Roberts & Strange, 1991)

kaolin deposits are related to igneous activity, with minerals being concentrated along faults, plutonic contacts and zones of hydrothermal alteration.

Tin, Tungsten and Molybdenum

Tin, wolframite (tungsten) and molybdenite commonly occur in veins associated with pegmatite and greisen, and in granite contacts and northwest-trending faults.

Lead, Copper and Zinc

Lead, copper and zinc mineralization is mainly associated with epithermal veins in coarse ash crystal tuff in the New Territories and Lantau, with the richest deposits close to major northeast-trending shear zones, with lead and zinc being mined at Lin Ma Hang (Williams, 1991).

Iron

Iron mineralization is present in many areas of Hong Kong and in a variety of mineral occurrences. The largest deposit is at Ma On Shan where considerable quantities of magnetite were extracted (Strange & Woods, 1991). The magnetite is associated with skarn within the contact zone between granite and marble. Many parts of the mine are still accessible but large-scale hillslope instability has developed around some of the earlier open pit workings.

Non-metal Mineral Deposits

Non-metalliferous minerals have also been worked, notably graphite on West Brother Island (Woods & Langford, 1991) and at Mai Po. Many small deposits of kaolin have also provided material for porcelain for many years such as the centuries old kaolin pits at the Wun Yiu Village kilns at Tai Po (a Declared Monument). Larger workings of kaolin existed at Chek Lap Kok and Cha Kwo Ling.

Case Studies

Wolframite Mining and the Shing Mun Road Tunnels

The main workings for tungsten were the extensive underground wolframite mines at Needle Hill (Figure 6.10.8). Many of the workings are still intact, and portions of the mine had to be infilled when the Shing Mun Road tunnels were excavated nearby (Roberts & Strange, 1991). From 1949 to 1951, when the price of tungsten increased as a result of the Korean War, an estimated 5,000 people were working at trial pits and sediment sluicing

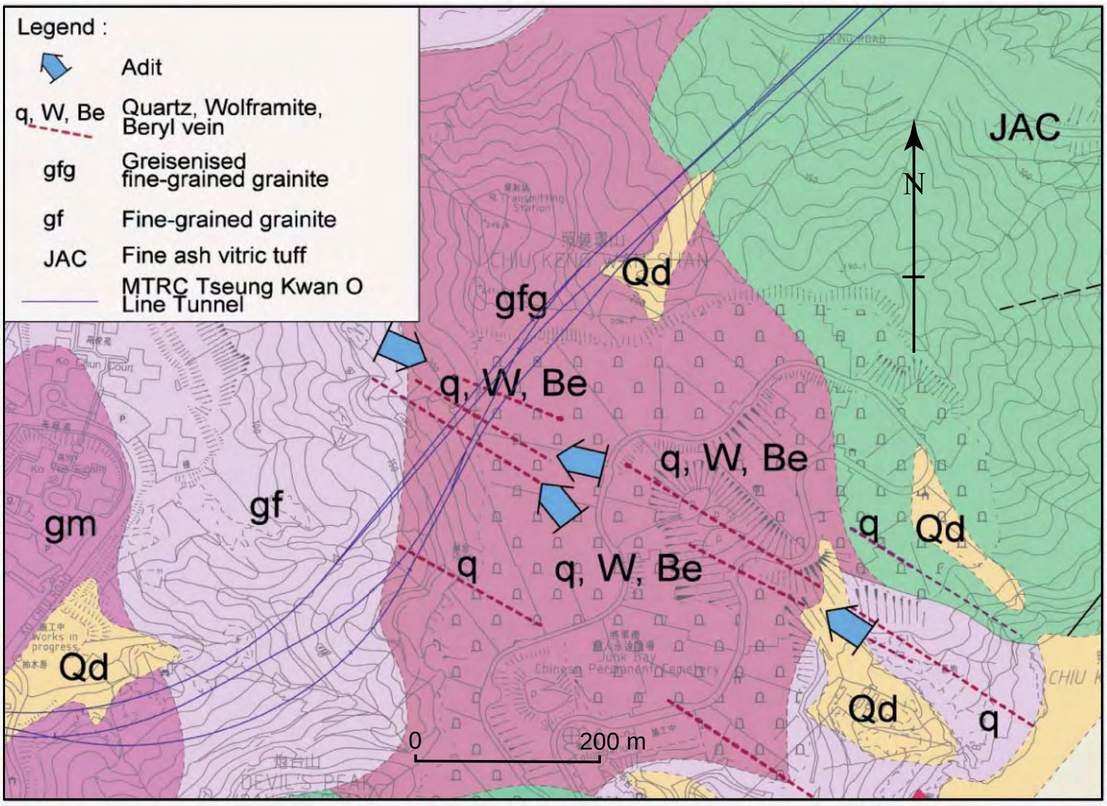


Figure 6.10.9 – Black Hill Tunnels - Extract from 1:20,000-scale geological map with mineral veins and adits associated with hydrothermal alteration highlighted

at various sites in the New Territories (Roberts & Strange, 1991). It was as a result of this widespread illegal mining, which was stripping hillsides, causing soil erosion, and polluting water courses, that the Government enacted the Hong Kong Mining Ordinance and created the post of Superintendent of Mines to supervise mining.

MTRC Black Hill Tunnels

Hencher (2006) describes a zone of hydrothermally altered granite which could be traced across several of the MTRC Black Hill tunnels at a depth of about 200 m beneath the Pau Toi Shan – Chiu Keng Wan Shan ridgeline, which separates Yau Tong from Tiu Keng Leng. The weak material in the zone was encountered unexpectedly and caused a collapse and some delay during construction. However , the 1:20,000-scale geological map of the area (GCO, 1986b) shows a number of mine adits associated with mineral veins that strike northwest directly across the alignment of the tunnels (Figure 6.10.9). The veins contain an assemblage of quartz, wolframite and beryllium and are hydrothermal in origin. The ground investigation also indicated a depression in rockhead in the area of the veins shown on the geological map. In addition, the geological map and accompanying memoir indicate that greisenisation is extensive in the area.

This example illustrates that the presence of old mine workings associated with minerals of hydrothermal origin can be an indicator of potentially poor-quality zones at depth, which can cause problems during tunnelling. These problems can be anticipated and minimised by the implementation of good engineering geological practice and sound geotechnical risk management.

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