Geoguide 4

# Guide to Cavern Engineering

Geotechnical Engineering Office
Civil Engineering and Development Department
The Government of the Hong Kong
Special Administrative Region

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First published, March 1992
First reprint, June 1994
Reprinted with minor correction, October 1998
Second Edition, January 2018

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# Foreword

This Geoguide presents a recommended standard of good practice for the civil engineering aspects of rock cavern developments in Hong Kong. It also serves as a reference document for non-specialists involved in the planning and administration of cavern projects.

The contents of this Geoguide are derived from international and Hong Kong practice. Whilst the scope of this document is defined as the design and construction of rock caverns, much of the material presented here is also applicable to tunnelling.

The previous version of Geoguide 4 was drafted by Mr O.J. Berthelsen of Berdal Strømme Consulting Engineers as sub-consultants to Ove Arup and Partners and was first published in 1992. During the "Enhanced Use of Underground Space in Hong Kong" study, which commenced in 2009, it was decided that a revised version of the Geoguide should be prepared to incorporate advances made since the original publication. This revision was prepared by a team led by Mr M.I. Wallace of Ove Arup and Partners under a consultancy agreement with the Geotechnical Engineering Office (GEO). The consultancy agreement was managed by Dr K.C. Ng and later Mr Y.K. Ho and his staff in the Planning Division of the Office.

The preparation of this Geoguide was overseen by a Working Group. The membership of the Working Group, including representatives from relevant government departments, local learned societies and the MTR Corporation Limited, are given on the following page. The Management Committee of the GEO provided overall steering to the preparation of the Geoguide.

To ensure that this Geoguide would be considered a consensus document by the practitioners, a draft version was circulated locally and abroad for comment to contractors, consulting engineers, academic institutions, professional bodies and government departments in 2016. Many individuals and organisations made very useful comments, which have been taken into account in finalising the Geoguide. Their contributions are gratefully acknowledged.

As with other Geoguides, this document gives guidance on good engineering practice, and its recommendations are not intended to be mandatory. As experience and good practice evolve, practitioners are encouraged to provide comments to the GEO on the contents of this Geoguide at any time, so that improvements can be made from time to time. This Geoguide is a continuously updated document. Updated information is released on the CEDD Website.

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January 2018

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# 1 Introduction

## 1.1 Scope and Objectives

The purpose of this Geoguide is to describe good cavern engineering practice in Hong Kong. The Geoguide mainly covers the geotechnical and civil engineering issues relating to cavern design and provides guidance on producing cost-effective and safe designs.

The Geoguide is primarily aimed at engineers engaged in the planning, design, construction and supervision of cavern developments. It is also intended to be a useful reference for specialists in other fields including planners, regulators and developers.

The topics cover the history of cavern development, cavern usage, planning considerations, local ground conditions, site investigation cavern design and construction, adits, tunnels and shafts, monitoring and maintenance. The focus lies on the caverns developed using drill and blast method in the granites and volcanic rocks of Hong Kong.

Caverns are similar to tunnels in terms of engineering principles. The differences between the two are their physical dimensions and their applications. Caverns usually have larger cross sections than tunnels. Tunnels are used essentially for enhancing connectivity, whereas caverns are usually associated with a specific usage, which might include storage, industrial processes, commercial activities, and possibly habitation. In Hong Kong, caverns have been constructed with spans up to 27 m and heights up to 17 m. The world's largest-span cavern for public use is the Gjøvik Olympic Cavern Hall in Norway, with a span of 61 m and a height of 25 m.

The Geoguide covers the typical range of designs for most uses of caverns. Special design aspects of caverns for particular usages such as radioactive waste repositories, fortification/civil defence, heat flow/containment, de-humidification and human habitation are not covered. Reference may be made to studies such as the Study of the Potential Use of Underground Space (SPUN) (Arup, 1990), the Enhanced Use of Underground Space in Hong Kong (Arup, 2011), and the Long-term Strategy for Cavern Development (Arup, 2016).

In using this Geoguide, it must be appreciated that the information and good practice contained herein presuppose the engineering judgment of a skilled and experienced practitioner; the Geoguide can never be a substitute for an experienced cavern designer.

## 1.2 Background

There is a growing interest in Hong Kong in placing facilities in caverns. This interest has arisen because of the favourable conditions to cavern development in Hong Kong and the many advantages caverns offer. The Enhanced Use of Underground Space in Hong Kong (Arup, 2011) assessed that approximately two-thirds of the land area of Hong Kong is suitable for cavern development. Further study under the Long-term Strategy for Cavern Development (Arup, 2016) provided a holistic approach in planning and implementing cavern development so as to render it as a sustainable and innovative means for expanding land resources.

Some of the typical advantages and potential disadvantages associated with cavern developments are provided in Table 1.1. Some of these disadvantages, such as restricted space and emergency management, are similar for any underground scheme whether involving rock caverns or basements.

The primary drivers for cavern developments are often lack of space, security requirements and the need to reduce environmental impacts, particularly visual impacts. Furthermore, in some cities, land values and costs of land formation are high, providing a further incentive to develop caverns. However, it is necessary that geological conditions are suitable as the ground conditions dictate how the piece of land can be used. Where hard rock lies close to surface, development of rock caverns will be preferred. Basements and cut and cover excavations may be better options where soils are present. It is also possible to locate caverns below bodies of water, as is the case for the recently completed fuel storage caverns at Jurong, Singapore.

The use of rock caverns also releases land for other uses. It may be easier to secure public acceptance for environmentally sensitive “not in my back yard” (NIMBY) land uses, such as refuse transfer stations and sewage treatment works, if these are located in caverns.

Cavern usage tends to be higher in densely populated cities and countries with mountainous or near surface hard rock terrains. Many countries in Scandinavia have taken advantage of the regions’ wealth of near surface hard rock to undertake substantial development of caverns in urban areas. There the use of caverns can mitigate the harsh seasonal climate, serve as emergency shelters, improve security, access and connectivity and generally improve the environment. Some Scandinavian countries have also given priority to relocating NIMBY facilities underground.

Given the potential benefits, countries such as China (including Hong Kong), Japan, Singapore and South Korea are increasingly interested in the development of caverns to solve some of the pressing issues in land formation and the effective use of high value land in urban areas.

**Table 1.1 Advantages and Disadvantages of Cavern Developments (Sheet 1 of 2)**

| Factor | Advantages | Disadvantages |
|---|---|---|
| Economic | • Serve as an additional source of land supply for public and commercial uses.<br>• Remove significantly blighting effect of surface NIMBY facilities on nearby property values.<br>• Offer significant, intangible benefits, while unquantifiable.<br>• Release at surface land which offsets the construction and operation costs.<br>• Provide an environment of steady climate, which is good for the storage of materials that degrade such as food, drink, documents and information technology.<br>• Result in savings in maintenance, energy use and insurance costs for some facilities.<br>• Produce good quality excavated rock material (possibly combined with underground quarrying).<br>• Reserve underground space to cater for ease of future expansion of cavern facilities. | • Incur higher capital and whole-life costs, particularly due to aspects such as ventilation and lighting.<br>• Need long lead-in times.<br>• Entail higher project risk in terms of cost and time due to uncertainty with respect to ground conditions.<br>• Require a higher construction cost for expansion of existing underground cavern developments than the surface scheme, if adequate provision has not been planned as part of the design.<br>• Incur further costs for decommissioning or suspending use of the underground space. |
| Transport and Connectivity | • Create an integrated network linking the surface and the subsurface structures when used in conjunction with other underground spaces such as basements, subways and tunnels.<br>• Provide an opportunity linking access points with main transportation nodes to improve connectivity.<br>• Improve traffic conditions and create parking space. | • Make connections more challenging to implement for caverns constructed at greater depths than other existing subsurface infrastructure.<br>• Cause greater transportation impacts locally during construction than an above ground scheme due to more restrictive access points. |
| Resilience | • Provide a space underground that are generally more resilient to natural disasters (including earthquakes) and extreme weather. | • Require more effort to restore after flooding if occurs.<br>• Rely on proper functioning of access points under adverse conditions. |

**Table 1.1 Advantages and Disadvantages of Cavern Developments (Sheet 2 of 2)**

| Factor | Advantages | Disadvantages |
|---|---|---|
| Sustainability and Environment | • Create space while preserving natural landscape and ecology at the surface.<br>• Allow more efficient use of land at the surface. Land released at surface can be used to provide other uses such as green spaces and recreational areas.<br>• Reduce visual impact. (Visual impact due to possible ventilation shafts and portals should also be considered.)<br>• Contain undesirable environmental impacts (e.g. dust, odour, noise and vibration) from certain uses, such as industrial operations.<br>• House NIMBY facilities in isolation from the rest of the community.<br>• Have high longevity of use and occupation and can be re-used for other purposes.<br>• Provide an opportunity for mining high quality rock materials and harvest geothermal energy as part of the development. | • Result in concentrated discharge of exhausted gases by the ventilation system, which may cause a local impact on air quality if not properly controlled.<br>• Create an irreversible underground space.<br>• Have higher embodied energy comparing to a surface scheme.<br>• Require consideration of psychological and physiological factors for certain highly populated uses, such as columbarium. |
| Safety and Health | • Isolate hazardous materials and processes and reduce societal risks.<br>• Provide additional protection to the facilities and materials within a cavern against external influences.<br>• Provide a space with potentially better air quality. This can potentially be better underground if appropriate ventilation and air filtering is provided. | • Require proper management of construction related safety hazards.<br>• Can be challenging to provide adequate means of escape due to distance to reach surface or ground level.<br>• Require additional plant and procedures as compared to surface facilities for mitigating fire and other hazards during operation stage (e.g. emergency power, ventilation, and compartment).<br>• Require effective control of gases such as radon and any emissions from the cavern uses. |
| Others | • Provide refuge points. | • Require consideration of the ownership and compatibility of the surface and underground land uses. |

# 2 Review of Cavern Usage

## 2.1 Introduction

Since the earliest times, largely opportunistic use has been made of naturally occurring caves for habitation and primitive industry. An example of an early use of man-made caverns is Neolithic flint mines. One of the most interesting discoveries of the early use of underground space is that of the limestone caves of Zhoukoudian where Peking men lived during the middle Pleistocene period, some 200,000 to 700,000 years ago (Yip, 1983).

The use of man-made underground space has been recorded from all the early and great civilisations. Many of these uses were funerary or ceremonial as in ancient Egypt. Extensive use of underground space can be seen in the Nabatean city of Petra, Jordan (temples and other uses), some 2,000 years old, and various sites in Turkey such as Derinkuyu and Kaymakli, dating from the sixth and seventh centuries, where extensive underground rooms and passages were excavated. At these locations, the underground space was created by excavating considerable volumes of soft rock.

The first modern development of underground cavern space for public use purpose, other than mining, appears to have been a small power plant installed in a rock cavern at Snoqualmie Falls in Washington, USA, in the late 19th century. At that time the cross-sectional area of caverns was rarely more than 30 $m^2$ and the excavation depended heavily on manual labour. Today's methods of excavation have made cross-sectional areas in excess of 800 $m^2$ technically feasible and economic.

## 2.2 Hong Kong

Cavern developments in Hong Kong began in the 1980s with the publication of the Underground Oil Storage Study (Neste, 1982a & 1982b) and the construction of the valve chamber in the Western District Aqueduct. Since that time a number of caverns have been constructed (Table A1 in Appendix A) and significant studies on cavern planning and design have been carried out in Hong Kong, including:

- Study of the Potential Use of Underground Space (SPUN) (Arup, 1990),
- Preliminary Engineering Geological Studies (PEGS) (Choy, 1990; Franks, 1991 & 1993; Roberts, 1991a, 1991b, 1992a & 1992b; Shiu, 1993; Lam, 2010; Cho, 2010; Wong, 2009; Lam & Roberts, 2010; So, 2011),
- Cavern Projects Study (CAPRO) (Arup, 1991a & 1991b),
- Cavern Area Studies (CAS) (Choy & Styles, 1992; Roberts, 1993 & 1994; Roberts & Kirk, 2000),
- Enhanced Use of Underground Space in Hong Kong (Arup, 2011), and

- Long-term Strategy for Cavern Development (Arup, 2016).

Guidance documents on the use of caverns have also been prepared, including:

- Geoguide 4 (GEO, 1992),
- Hong Kong Planning Standards and Guidelines (HKPSG) (PlanD, 2008), and
- Guide to Fire Safety Design for Caverns (BD, 1994).

More recently, caverns have been constructed by MTR Corporation Limited to house some new railway stations. In addition, the major underground infrastructure projects carried out by Water Supplies Department (WSD) and Drainage Services Department (DSD) have provided additional experience and good examples of the use of underground space in Hong Kong. Table A1 in Appendix A includes examples of recent cavern uses in Hong Kong.

### 2.3 Overseas

The variety of uses of cavern developments in different parts of the world is considerably wide. Some uses are common, with many installations in operation, e.g. fuel storage, power stations and laboratories. Other uses are specialist in nature, e.g. nuclear waste repositories, bunkers and ammunition storage. Many are associated with mining operations, transport projects (e.g. for railway stations), community uses (e.g. civic centres, art galleries/museums, columbaria/mausoleums, mortuaries/crematoriums, refuse transfer facilities, service reservoirs and sewage/water treatment, ice rinks and swimming pools). Table A2 in Appendix A gives a list of examples of cavern uses worldwide. Appendix A also describes two particularly notable cavern developments in Norway and Singapore.

# 3 Cavern Planning

## 3.1 Introduction

A holistic and forward-looking approach is required for planning a cavern as a sustainable and resilient form of development that meets the present and future needs.

There is a growing trend worldwide of viewing the underground as a natural and strategic resource that can provide society with benefits of underground space, minerals, groundwater and geothermal energy. With good planning, the best economic outcome of the development can be achieved. Well-planned cavern developments can also improve the resilience of a city or region to natural disasters and extreme weather.

Unlike surface structures, it is often impractical to restore an underground opening to its original condition once created. The presence of this opening may affect all future uses of the surface and subsurface in the vicinity. Detailed planning is required to ensure coordinated and effective use of resources and maintain development opportunities for future generations.

This chapter provides a general guide to the planning approach in Hong Kong for cavern developments and also discusses the broad planning and consultation approaches used worldwide.

## 3.2 Review of Planning Approaches

### 3.2.1 Hong Kong

The HKPSG (PlanD, 2008) covers the planning of rock caverns in Hong Kong. It lists the potential land uses of rock caverns and outlines of the planning process. In brief, for project proposals, involving land uses with potential for development in rock caverns, a planning assessment is required to be carried out by the project proponent at the initial planning stage to evaluate any cavern development options against the non-cavern options in terms of land use compatibility, safety, traffic and financial viability aspects.

Owing to the complexity of the space developed within rock caverns, there are practical difficulties in defining plot ratio and site coverage. A three dimensional development envelope should therefore be delineated for incorporation in the engineering/lease conditions. The development envelope defines a space within which cavern construction is permitted and a surrounding space within which excavations or other works should not be permitted apart from access tunnels, drainage adits, and ventilation shafts specifically for the cavern and incorporates any other maintenance requirements. A protection zone should also be defined to secure the operation and structural safety of the cavern development.

A Cavern Suitability Map (CSM) for Hong Kong (Figure 3.1) was developed as part of the study on the Enhanced Use of Underground Space in Hong Kong. The CSM considers a variety of suitability criteria including existing and proposed surface developments and underground facilities, engineering geological conditions, topographic constraints, country parks, areas of special scientific interest, availability of ground investigation data, reclamation/fill/landfill areas, areas of poor geology, scheduled areas, impounding reservoirs and whether sites are below +10 mPD (as it is considered that conditions will generally be less favourable for a cavern development around or below sea level).

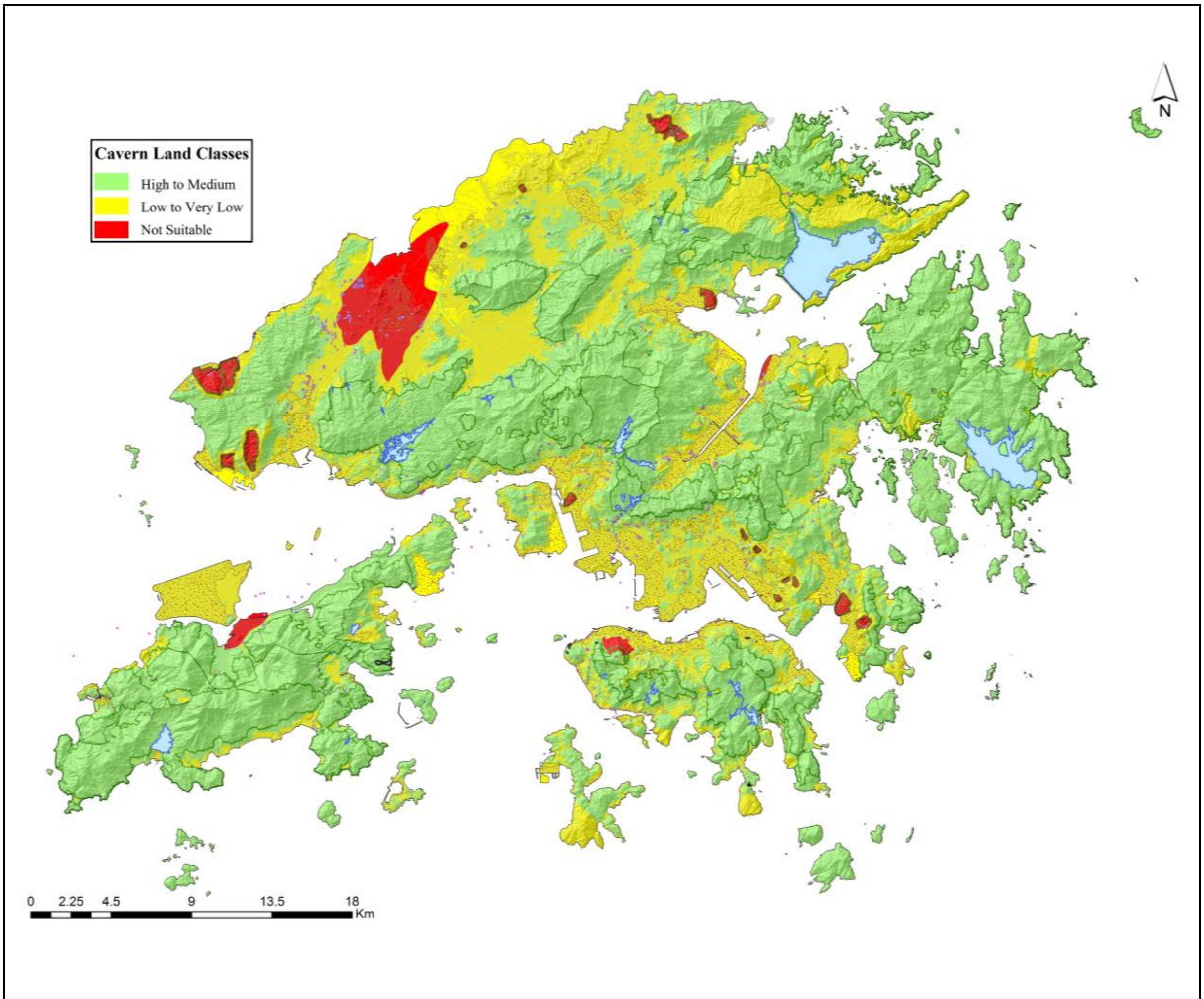



**Figure 3.1 Cavern Suitability Map for Hong Kong**

The Cavern Master Plan (CMP), which was developed as part of the study on the Long-term Strategy for Cavern Development, is a planning tool providing a broad strategic planning framework to guide and facilitate territory-wide cavern development in Hong Kong. Areas that are considered strategic for cavern development (referred to as Strategic Cavern Areas, SCVAs) in terms of geological conditions and the current planning perspectives to support future development need are delineated on the CMP. SCVAs identified are easy to access, sufficiently large to accommodate multiple facilities and located at the urban fringe with supporting infrastructure network. The CMP aims to make known these SCVAs and their essential information to project proponents such that they can identify suitable cavern sites for their developments.

### 3.2.2 Overseas

In Beijing, China, the Beijing Central District Underground Space Development and Utilisation Plan (2004-2020), has been implemented since 2005. The Plan is aimed to encompass both current utilisation of the underground and proposed requirements for future developments. The city of Helsinki in Finland launched the Helsinki Underground Master Plan in 2009 for the holistic planning of underground space. Other cities such as Singapore are also implementing policies and master plans appropriate for their unique requirements for use of underground space. Goel et al (2012) and Arup (2011) summarise further details on overseas experience in planning caverns and developing underground space.

## 3.3 Type of Facility

HKPSG (PlanD, 2008) listed the land uses which have the potential to be located in rock caverns.

Table 3.1 presents the land uses with potential for rock cavern developments in Hong Kong. Appendix B reviews the key features of these land uses and identifies the potential issues that need to be addressed.

**Table 3.1 Land Uses with Potential for Rock Cavern Developments in Hong Kong**

| Land Use Category | Potential Land Uses | |
|---|---|---|
| Commercial | Food/Wine storage<br>Food and beverage | Retail |
| Industrial | Container storage<br>Data centre<br>Industry<br>LPG bulk storage | Oil bulk storage<br>Research/Testing laboratories<br>Storage/Warehousing |
| Government/ Institution/ Community and Other Specified Uses | Archives<br>Civic centre<br>Columbarium/Mausoleum/Mortuary<br>Cultural/Performance venue<br>Explosives depot/magazine<br>Incinerator<br>Indoor swimming pool/complex<br>Leisure/Sports centre<br>Maintenance depot<br>Recreational complex | Refuse transfer facility<br>Sewage/Water treatment plant<br>Service reservoir<br>Slaughterhouse<br>Transport connections & networks<br>Vehicle (including bus) depot<br>Vehicle parking<br>Wholesale market<br>Underground quarrying |
| Public Utilities | Power station | Public utility installation |

### 3.4 Key Planning and Technical Considerations

Given the complexity of planning for caverns, a wide range of issues as discussed below should be considered at the planning stage of a development:

(a) The development should be placed on a sound economic basis and the opportunity should be taken to realise potential economic benefits including improved land values, redevelopment of the land released at the surface, reduced land take for new facilities, integrated connectivity and shared development costs.

(b) The ownership and obligations of those occupying the underground space should be considered in detail.

(c) The engineering and architectural feasibility must be adequately demonstrated.

(d) The opportunity should be taken at the planning stage to optimise the location, orientation, shape and size of the development with respect to the geological setting as well as architectural and operational requirements, as it becomes increasingly difficult to do this when the project progresses.

(e) Sufficient access points should be made available. Wherever possible, the development should improve integration and connectivity with nearby land uses and communities, including interconnectivity with existing surface and underground facilities and infrastructure.

(f) Sufficient consultations and/or engagement should be carried out to build consensus among government, stakeholders and the community on the development. The suitability of each site for any cavern development should be identified individually through review and assessment.

(g) The statutory and regulatory requirements that are relevant to the development (e.g. land use zoning, environmental assessment, Buildings Ordinance for private developments) have to be identified and met.

(h) The safety aspects of the potential cavern development throughout its life cycle, particularly fire safety, including provisions of emergency access and means of escape, and protection zones, should be adequately addressed.

(i) The environmental impacts of the scheme during construction and in the long term should be minimised.

(j) The potential for future expansion or upgrading of the cavern development should be considered in the proximate surface and underground land use planning.

The considerations in the above various areas are often overlapping and may also be in some circumstances contradictory in their requirements. An appropriate balance has to be achieved for each cavern development.

Table C1 in Appendix C presents further elaboration on these issues. The guidance is not intended to be exhaustive as every cavern development is unique.

# 4 Geological Input into Cavern Engineering

## 4.1 Introduction

Knowledge of the geology is the starting point for all geotechnical investigations of cavern developments. Geological information is useful for the early assessment of the suitability of a particular cavern site and serves as a basis for developing geological models and site investigation plans.

The following aspects of the geology of a site are of fundamental importance to cavern design and construction and therefore detailed assessment is required:

(a) The solid geology, or rock type, gives an indication of the mechanical properties, discontinuity spacings and the patterns of weathering that may be expected. In the strong igneous rocks of Hong Kong the discontinuities govern engineering behaviour. The rock type also influences the cost of drilling and blasting.

(b) The geological structure gives a guide to the orientation of major discontinuities, such as faults that may be encountered; discontinuity orientations tend to follow regional patterns.

(c) Typical weathering patterns give an indication of the total cover required for a cavern and the works that may be required in establishing portals, access adits and shafts.

(d) The groundwater conditions may be important for some sites, and some types of cavern use and an initial evaluation may be made on the basis of generalised data.

This section discusses geological models, including the overall model approach and their relationships to ground models and design models. It also gives an account of the broad geological settings of Hong Kong including the solid geology, structures, weathering, soils, in-situ stresses and groundwater.

## 4.2 Geological Models

### 4.2.1 Overall Model Approach

A three-step process is commonly adopted in both Hong Kong and worldwide as the framework for engineering geological input into cavern developments and other projects. This process involves an iterative development of a series of models that in Hong Kong practice are typically referred to as geological, ground and design models, although a variety of other terms and concepts are used elsewhere.

As noted by Barbour & Krahn (2004), the maximum benefit from modelling occurs when it is applied to the entire process of data collection, interpretation, design and prediction of behaviour. The modelling process must envelop site investigation (geological model), field and laboratory testing as well as monitoring (ground model) together with theoretical idealisation and numerical analysis (design model). A case study of the geological input to the design and construction of the Tai Koo MTR Station cavern is discussed by GEO (2007).

The first step in the process is the development of a geological model. Figure 4.1 shows the typical development and application of a geological model for a major project. Details are given below. Reference may be made to GEO (2007) for guidelines on development of ground model and design model. Further details are given in Sections 6.6 and 6.7 of this Geoguide.

### 4.2.2 Definition of a Geological Model

A geological model characterises the geological, geomorphological and hydrogeological features of a site for engineering applications. The model should include considerations of the materials, geomorphology, structure, groundwater and stress conditions of the ground. It should also cover geological processes, geological hazards and boundary conditions. The geographical extent of the model depends primarily on the type of proposed works and the hazards that may be relevant.

Fookes (1997) noted that models can be presented in written descriptions, sections, plans, block diagrams and 3D models (including computer visualisations), and may focus on some particular aspects, such as groundwater and major structural features (e.g. discontinuities), to suit a specific engineering objective.

### 4.2.3 Development of a Geological Model

There is a symbiotic relationship between geological (and ground) models and the site investigation. This is because these models facilitate effective site investigation to be carried out and the findings of the site investigation in turn enrich the development of the models. The process, like so many facets of a cavern development, is iterative. The models should be refined and updated throughout the project. The geological model is of particular importance during the early stages of site investigation and at this point may be little more than a series of hypotheses and uncertainties to be investigated, defined, confirmed, disregarded or added to the geotechnical risk register as appropriate. As the project progresses, the focus of the site investigation will move onto the development of a ground model, although there is no clear delineation between the two.

Successful development of a geological model and the site investigation requires the knowledge, skills and judgment of an experienced engineering geologist. Given that the human factor is critical, it is essential that suitably experienced personnel are involved in the supervision of the preparation and review of the models.

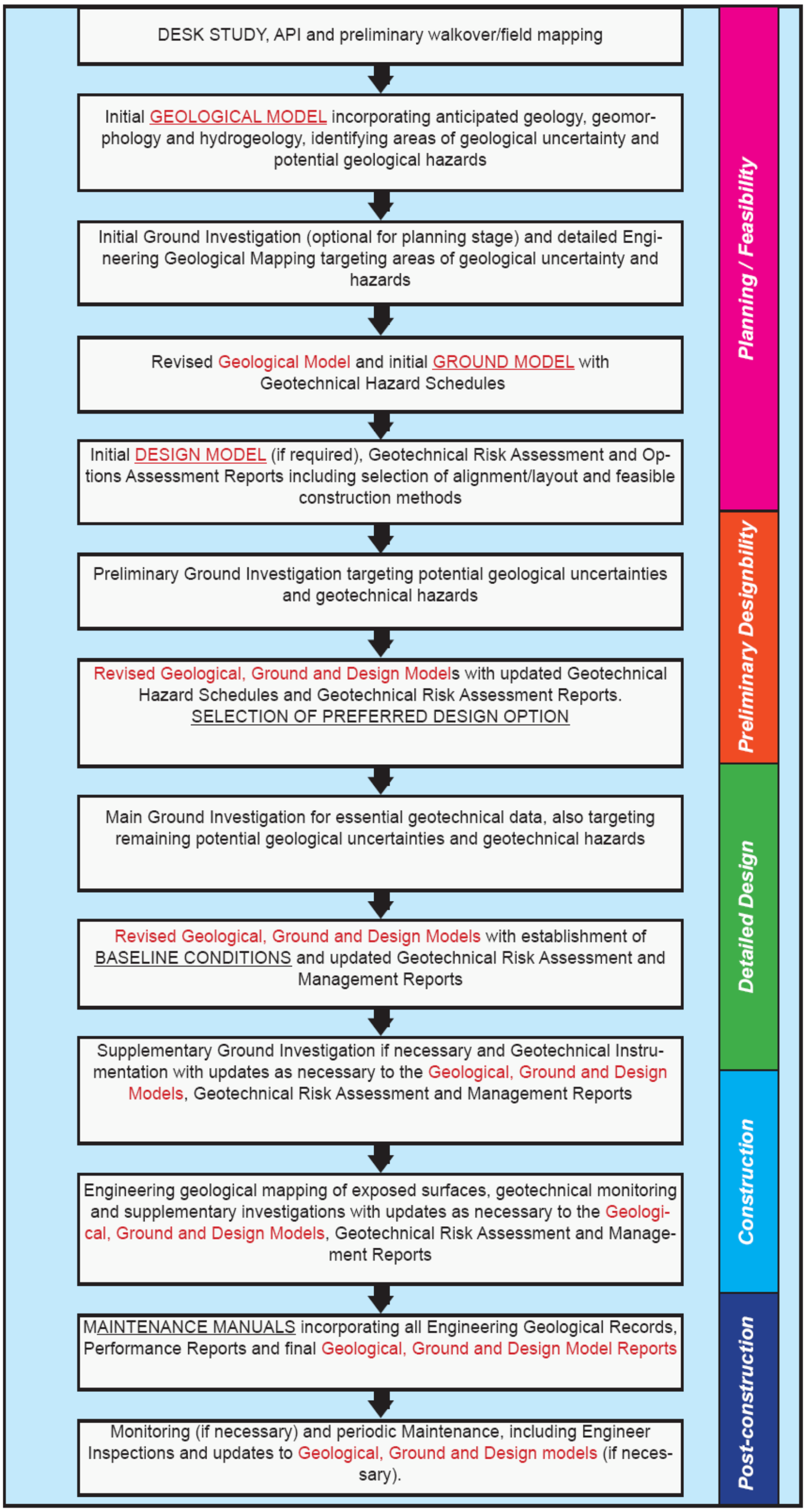



**Figure 4.1 Typical Development and Application of a Geological Model for a Major Project (GEO, 2007)**

The strategy is to begin with a simple geological model and for the site investigation to proceed with an initial investigation. As the investigation proceeds, the geological model will become increasingly detailed (but should not be any more complicated than is required to define issues of engineering interest) and the site investigation methods will typically become more sophisticated.

For a cavern project, the initial working geological model is developed on the basis of an assessment of existing data and a preliminary site visit. At this point, it is important to assess and include the regional scale geology as well as local scale geological observations and data where applicable.

This is followed by a detailed engineering geological mapping of the cavern site and its environs. The area to be covered must be sufficient to give a good understanding of the geological settings, in particular the larger scale structures and groundwater features that may affect the cavern scheme. The results of the engineering geological mapping should be added to the geological model.

Ground investigation, using the methods to be described in Chapter 5, is required to further develop the geological model and to give other geotechnical information, including the information needed for the development of the ground model. Engineering geological mapping alone rarely yields sufficient data to define an unambiguous and complete geological model of a site. In particular, weakness zones are often recessive in the topography and do not provide surface rock exposures without carrying out the ground investigation.

Figure 4.2 shows a good example of the development of a geological model, of which ground data are incorporated into the model gradually as the site investigation progresses.

### 4.2.4 Prediction, Uncertainties, Hazards and Communication

An important function of geological models is that they allow ground conditions to be interpreted, predicted and for these predictions to be tested during the site investigation. The information shall be used to assess the potential geological hazards and the suitability of construction methods. Features including in-situ stress conditions, major structures in rock, weakness zones, groundwater conditions, etc. are of direct relevance to cavern developments. Parameters to define the excavatability will have implications on the selection of construction methods and the construction programme.

Another use of geological models is to identify engineering geological uncertainties and reduce these to an acceptable level through the site investigation. Important uncertainties that have not been resolved should be added to the geotechnical risk register. Examples of those uncertainties may include the orientations and characteristics of weakness zones and groundwater conditions.

During the early stages of a cavern development, the uncertainties are typically large. However, as the project progresses, the uncertainties would be reduced, provided that an effective and comprehensive ground investigation has been carried out. Some unresolved engineering geological uncertainties may be sufficiently important that alternative geological models are required to ensure the variability of the ground is adequately addressed in the design.

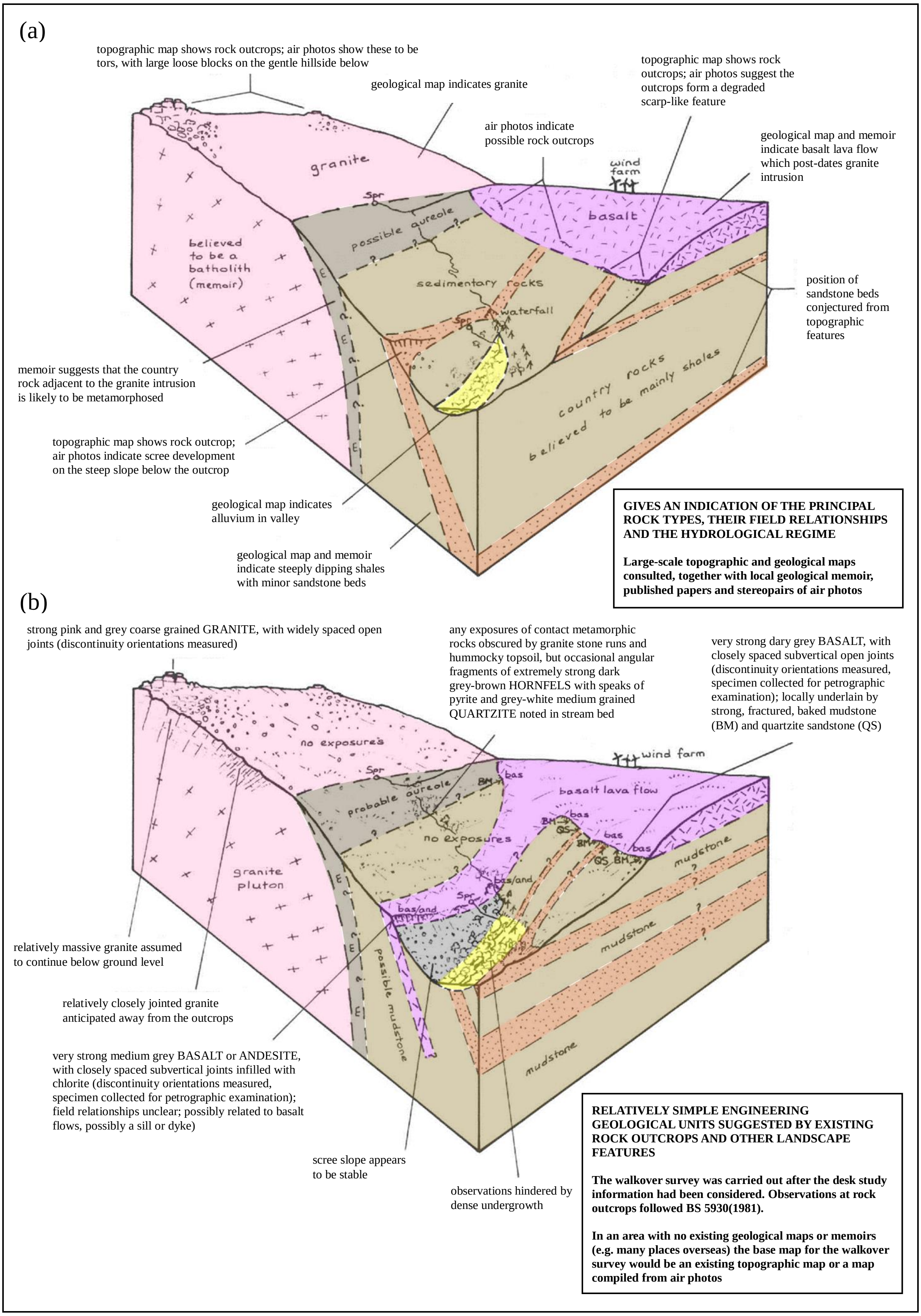



**Figure 4.2 Example of the Development of a Geological Model (modified from Fookes, 1997) (Sheet 1 of 2)**

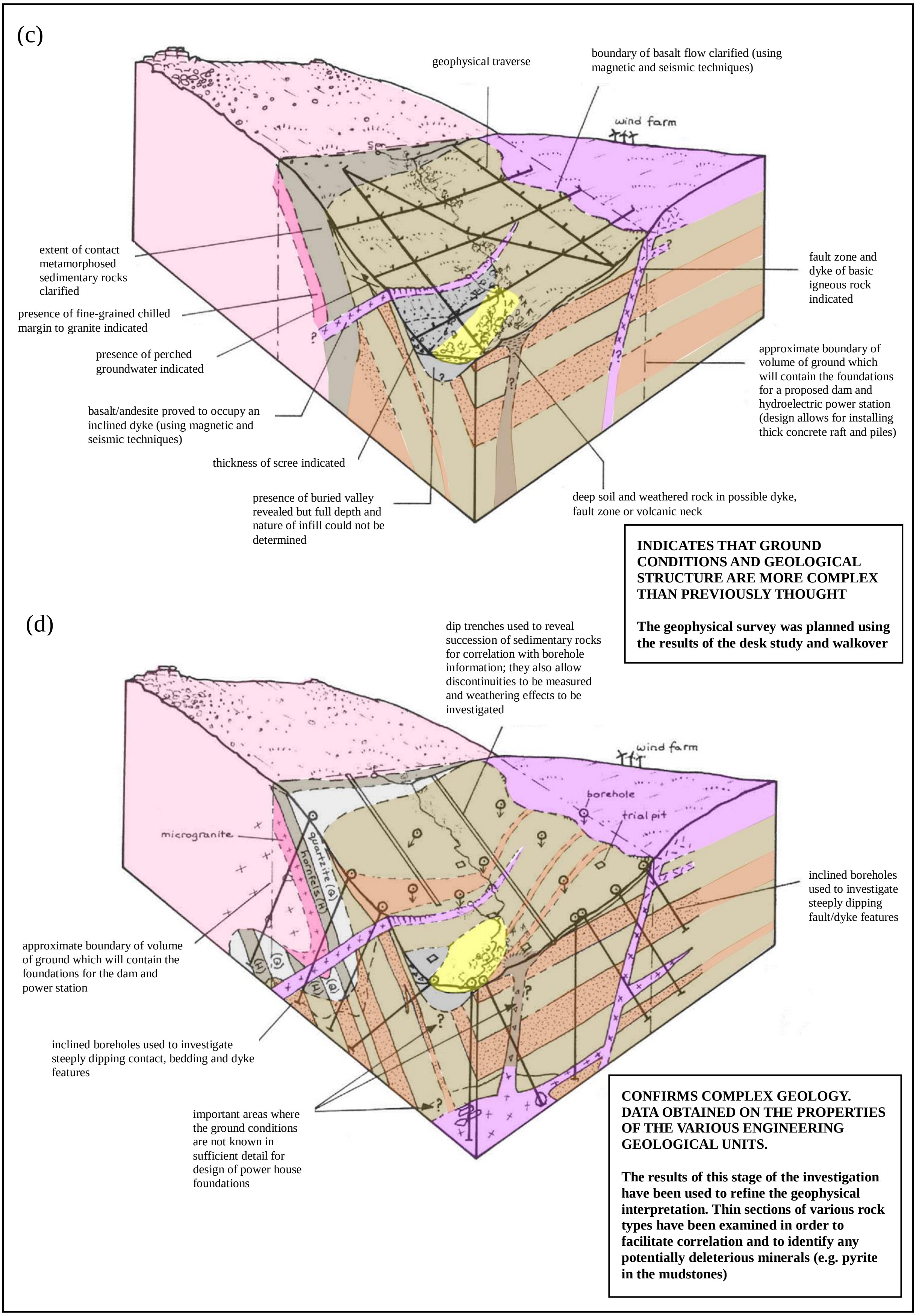



**Figure 4.2 Example of the Development of a Geological Model (modified from Fookes, 1997) (Sheet 2 of 2)**

Finally, a geological model is of limited value if its assumptions, limitations, uncertainties and risks are not carefully documented and communicated effectively to the project team. Software for two- and three-dimensional (2D and 3D) visualisations is available and commonly used for modelling. Effective communication can be facilitated by clear visualisations, simple terminology, and the use of a geotechnical risk register and systematic risk management.

## 4.3 Geology of Hong Kong

### 4.3.1 Solid Geology

A variety of rock types of igneous, sedimentary and metamorphic origins are found in Hong Kong. Of these, the igneous rocks, principally granite and the various volcanic rock types, have the greatest potential for cavern development and comprise over 80% of the rocks found in Hong Kong. Figure 4.3 shows a simplified geological map of Hong Kong indicating the distribution of the principal rock types. Within the main urban areas, granitic rocks underlie Kowloon, the northern parts of Hong Kong Island, parts of Lantau, Tsing Yi, Lamma and other smaller islands, Sha Tin, Tsuen Wan and Castle Peak, although in places these are concealed by superficial deposits. Generally, volcanic rocks underlie the middle and upper levels of Victoria Peak, the southern parts of Hong Kong Island, Western Lantau Island and much of the central, eastern and north-east New Territories. In north-east Lantau and Tsing Yi, a major swarm of feldsparphyric and quartzphyric rhyolite and rhyodacite dykes has intruded into both the granitic and volcanic rocks.

Sedimentary rocks crop out in two main areas in the north-west and north-east New Territories. They have undergone regional and dynamic metamorphism to varying degrees. Marble is also known to exist in the following areas (although there are no surface outcrops): Yuen Long to San Tin, Ma On Shan, Tung Chung and north shore of Lantau Island.

Hydrothermal alteration has affected both the sedimentary and igneous rocks but the effects are usually localised. Intrusion of the granite plutons has resulted in thermal metamorphism of the country rocks, as shown by recrystallisation of the original rock-forming minerals.

The granites and volcanic suite of rocks range in age from Middle Jurassic to Early Cretaceous. Discontinuities in the granites are generally widely spaced (i.e. between 0.6 and 2 m). Sheeting joints are often present near the surface. The granitic rocks are normally composed of feldspar, quartz and biotite but vary in grain size, texture, composition and colour. Near the contact with the country rocks, the granites are sometimes finer-grained and the contact is usually sharp. Granodiorite and quartz monzonite in the form of sheet-like plutons, stocks, and dykes crop out in many areas.

The volcanic suite of rocks has a varied lithology and includes fine ash tuff, coarse ash tuff, trachydacitic and rhyolitic lava flows, and rhyodacitic and rhyolitic dykes. Tuff is the most common rock type. The discontinuities in these rocks are generally closely spaced (i.e. between 60 and 200 mm); however, in the coarse grained varieties, discontinuity spacing up to 3 m may be found.

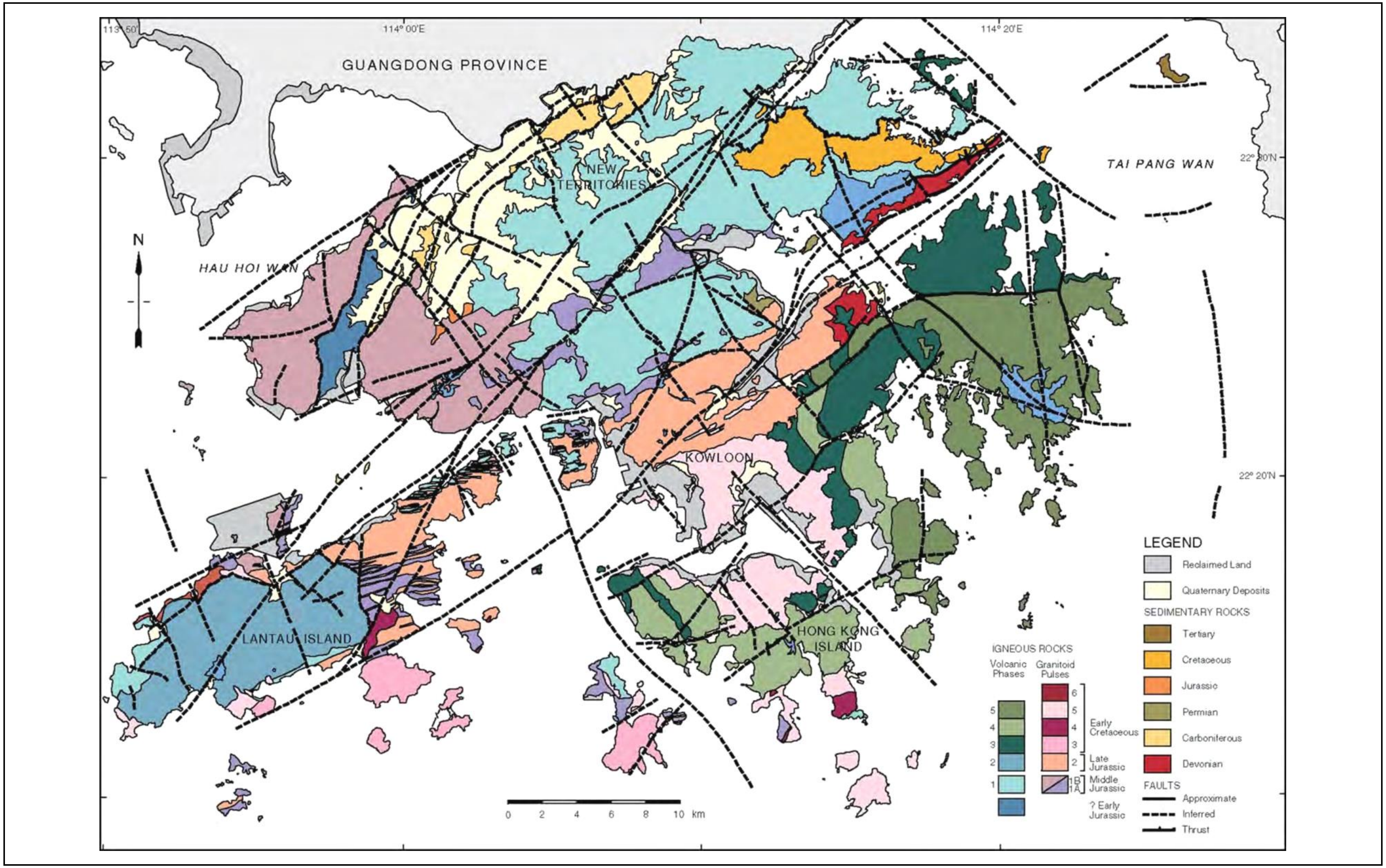



**Figure 4.3 Simplified Geological Map of Hong Kong (Fyfe et al, 2000)**

Mafic dykes of varying width, but generally not exceeding 2 m, have been intruded into the granitic, volcanic and some sedimentary rocks. The dykes tend to follow the regional trends, generally dip very steeply, and are the youngest intrusions in Hong Kong.

### 4.3.2 Structure

Many of the rocks mentioned above have been faulted and sheared. The main faults in Hong Kong strike north-east varying to north-northeast, and north-west varying to north-northwest. There are also occasional easterly trending faults and a few northerly-striking faults in the eastern reaches of Hong Kong. Regional dyke swarms generally follow the north-east trend. Fault zones, which vary in width and which can be many tens of metres wide, are generally vertical or steeply inclined (> 70°) with the significant exception of two thrust faults in the northern New Territories which dip at moderate angles. The volcanic rocks in the vicinity of the major faults in Hong Kong, particularly the major thrust faults, are found to be significantly sheared and altered. This may result in a significant change in the engineering properties. Faulted rocks may be weathered to considerable depths and, adjacent to faults, the rocks may be comminuted or very closely-jointed. Dykes are commonly closely-jointed and may have associated shear zones at their margins. The approximate locations of the more prominent faults are shown on the 1:100,000-scale geological map, but more accurate locations are shown on the 1:20,000 and 1:5,000-scale geological maps. In addition, smaller faults can be commonly identified in the field. Hydrothermal alteration is frequently associated with fault zones and infilling by quartz mineralisation is common.

A review of the earthquake data for Hong Kong and its nearby regions is given in GEO (2012). Arup (2015) reported that there was no active fault in Hong Kong, and the faults identified in the geological maps were formed by events that occurred during Jurassic and Cretaceous (Sewell et al, 2000). The seismicity in Hong Kong is considered low to moderate.

### 4.3.3 Weathering

The depth of weathering of the igneous rocks of Hong Kong can be considerable and can vary greatly over short distances. In closely-jointed volcanic rocks, this zone can be as little as a few hundred millimetres thick whereas in the granites the thickness can be several tens of metres (GEO, 1984). The process of sub-aerial weathering preferentially exploits faults and other zones of weakness where alteration can extend to great depth. Deep-seated alteration of hydrothermal origin, which can be difficult to distinguish from extreme cases of sub-aerial weathering, also occurs. The weathering of granites in Hong Kong is described in the Hong Kong Geological Survey Memoir No. 2 (Strange & Shaw, 1986), which provides an introduction to the formation and nature of these materials. A detailed discussion of weathering in Hong Kong is given in Chapter 3 of Fyfe et al (2000). GEO (2017a) recommends a standard for describing rocks and weathering profiles.

### 4.3.4 Soils

Soils are products of chemical and physical weathering. They may be in-situ weathered rock or transported weathering products that have moved down slope and covered the in-situ materials. The thickness of transported deposits rarely exceeds 30 m, but the thickness of the weathered rock mantle can be 60 m or more (GEO, 1984; Fyfe et al, 2000).

In many low-lying areas, recent marine and estuarine deposits are common. These may entail soft ground excavation for construction of portals or access tunnels. Section 8 provides guidance on the design and construction of portals, access adits, tunnels and shafts.

## 4.4 In-situ Stresses

There is no evidence of high tectonic stresses in Hong Kong rocks. Indications from test data suggest that high stresses will not be a problem for cavern construction at modest depths given the high strength of most of the rocks encountered.

Hydraulic fracture stress measurements to about 200 m deep have been conducted in Hong Kong for various construction projects. GEO (2007) and Free et al (2000) provide a summary of the in-situ stress measurement results. At a depth of about 30 to 50 m, the average horizontal stress ($S_h$) in rock is above twice the vertical stress ($S_v$). However, the measurement results show a large degree of scatter, which can be 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. For depth below 100 m, the average stress ratio (i.e. $S_h/S_v$) drops to about 1 to 1.5.

Free et al (2000) reported the orientation of principal horizontal stress in rock at some locations in Hong Kong. The principal horizontal stress directions appear to be generally consistent with the tectonics of the region.

When formulating geological model, 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.

## 4.5 Groundwater

The groundwater regime can be complex and it is not easy to have it characterised. The groundwater table in the hills of Hong Kong is generally 10 to 30 m below ground level. The depth of rock cover of most caverns can be greater than this, thus most of them are likely to be located below the groundwater table. Groundwater may occur as ephemeral groundwater perched on the weathered rock or superficial deposits. The ephemeral groundwater would normally have little impact on cavern schemes.

The hydraulic conductivity of the rock mass in Hong Kong is dominated by fissure flow, and is generally low in competent rock mass.

The low hydraulic conductivity is reflected in the steeply sloping groundwater surfaces that are often observed and in the relatively low seepage rates experienced in some previous tunnel excavations. However, substantial seepages from major discontinuities (e.g. faults and heavily fissured zones) of high hydraulic conductivity have been observed in a few tunnel excavations in granitic rocks. Heavy water inflows can affect the local stability of underground excavations.

Groundwater conditions can be the controlling factor for the siting and engineering design of a cavern. Some types of cavern installations, such as oil products stores and sewage treatment works, rely on groundwater seepage towards the caverns for containment of liquids and gases.

# 5 Site Investigation

## 5.1 Introduction

This chapter describes the objectives of site investigation for cavern developments, the methodology and scope, as well as the ground investigation techniques pertinent to cavern developments. General guidelines on site investigation are given in Geoguide 2: Guide to Site Investigation (GEO, 2017b).

The following are usually carried out at each stage of a cavern development:

(a) *Planning* – Desk studies and site reconnaissance including interpretation of aerial photographs and use of remote sensing data (e.g. airborne LiDAR).

(b) *Preliminary design* – Detailed engineering geological mapping (including discontinuity surveys), geophysical investigation, drillholes and initial groundwater investigation.

(c) *Detailed design* – Additional drillholes (likely including inclined and/or directional drillholes), geophysical investigation, detailed groundwater investigation and additional engineering geological mapping as necessary.

(d) *Construction* – Engineering geological mapping including rock mass classification and probing where necessary.

## 5.2 Objectives of Site Investigation

The main objective of site investigation for a cavern development, in common with those for most other civil works, is to provide data for the:

(a) assessment of the suitability of the site for cavern development and for the site selection where more than one site is considered,

(b) development of the ground model (particularly the rock mass characterisation and groundwater model),

(c) design of the cavern development including optimisation of size, shape, orientation and rock support,

(d) assessment of construction methods, time and cost, and the implications of any geological hazards and their impacts, e.g. adverse effects on hydrogeology, and

(e) assessment of environmental impact due to the cavern development, e.g. groundwater, vibration, noise, etc.

For achieving the above objectives, the site investigation should focus on the identification of potential adverse geological conditions such as weak or water-bearing features including faults, dykes and deep zones of weathering. In addition, the site investigation should establish relevant characteristics of the ground, which may include the use of rock mass classifications for determination of excavation methods, temporary support types, groundwater inflow control schemes and permanent lining types.

In some circumstances, site investigation is also needed to provide data for environmental impact assessment (EIA), such as potential noise and vibration impacts. The assessment may also examine the effect of the cavern development on the neighbouring natural environment including groundwater impacts and loss of yield to nearby streams and ecosystems. Investigation of any presence of contamination within the development site may also be required.

The required quantity and quality of design data increase as the project develops from the planning to construction stage. The amount of ground investigation that should be carried out depends on many factors including the availability of existing data, availability of good rock exposure and complexity of the geology.

Improved cost estimates require better geotechnical data and an increased amount of site investigation during the design phase. In addition, improved data enables project proponents and contractors to better understand the ground conditions they will encounter, possibly allowing the quantity of risk money reserved for adverse ground conditions to be reduced.

Subject to the arrangement of the construction contract, a geotechnical baseline report (GBR) to present a set of reference ground conditions may be prepared for incorporation into the contract documents. GBR is used as the basis of tenders, which draws a contractual baseline for measurement and dispute resolution and for reference during construction. Adequate site investigation should be carried out for the preparation of a reliable GBR.

## 5.3 Site Investigation Costs

The cost of site investigation for a particular cavern project depends greatly on the quality and adequacy of available information, ground conditions, extent of the affected zone and the types of sensitive receivers within the zone, as well as the risk of the types of works involved.

Adequate site investigation works should be carried out for the design and construction of the proposed cavern works in order to reduce project risks and uncertainties. However, there is no universal benchmark that gives the cost of adequate site investigation as a percentage of the project construction cost. A guide for assessing the needs of site investigation works for tunnels in rock is given in USACE (1997). Based on this guide, the typical cost of site investigation for a deep tunnel project located in difficult ground conditions in a dense urban area is about 3-4%. Some useful data on site investigation costs for tunnelling projects vis-a-vis the corresponding construction costs is given in ITA (2015).

If there is geological or hydrogeological complexity at the site (e.g. adverse geological features are found or suspected), or the proposed cavern development works may affect sensitive facilities thus requiring major risk mitigation works, it may be necessary to increase the ground investigation cost allowance.

However, there are inherent uncertainties in the subsurface geology and hydrogeology, regardless of the extent of site investigation. Also, physical constraints, e.g. existing buildings and subsurface installations, could limit the pre-tender site investigation for particular sections of tunnel works. Therefore, it is essential to make provision for additional ground investigation in the works contract to check and monitor continuously the actual conditions against those assumed, and to take measures to deal with conditions not anticipated but having significant impact on the design, construction, or on life and property.

The above should be used as a guide only as each project is unique. Professionals involved in the design and construction of the project should exercise engineering judgment to determine the scale of site investigation required.

## 5.4 Evaluation of Existing Data

### 5.4.1 Objective

The objective of a desk study is to obtain as much available information as possible on the ground conditions and other factors that may influence the siting of a cavern development and its design. The results of the desk study also form the basis for ground investigation. The desk study aims to reveal:

(a) the geology of the site such as rock type, structure, weathering, the presence of landslides, the nature, thickness and extent of natural soils / fill material, as well as the groundwater conditions; facilitating the development of the geological and ground models,

(b) previous and current land uses that may affect the scheme, particularly underground works such as mines and tunnels,

(c) sensitive receivers, including buildings, their foundations and their occupants, above or adjacent to the cavern development that may be affected by the works,

(d) the location of utilities which may affect or be affected by the scheme,

(e) suitable means of providing access to the cavern, including portal and shaft locations that will satisfy environmental, traffic and engineering considerations,

(f) existing geotechnical information related to nearby underground works, slopes, buildings, etc.,

(g) environmental conditions that may impose restrictions on scheme implementation, and

(h) conditions imposed by the land lease, which include requirements, restrictions and liabilities.

The application of a 3D model for consideration of the above information can effectively facilitate planning of a cavern development.

### 5.4.2 Sources of Information

GEO (2017b) together with GEO (2009) give a comprehensive guidance on carrying out a desk study. Important information for the preliminary assessment of a site is listed below:

(a) topographic maps at scales of 1:500 to 1:20,000,

(b) geological maps at a scale of 1:20,000 and associated memoirs (1:5,000 maps are available for some areas),

(c) aerial photographs showing history of the site; good quality aerial photographs available for most areas and for various years from 1963 onwards, at scales of 1:2,000 to 1:40,000,

(d) records of nearby underground construction and previous geotechnical works (e.g. records of rock slope mapping, tunnel mapping records, borehole logs, groundwater monitoring records),

(e) records of foundation plans kept by the Buildings Department and other works departments,

(f) records of utilities and other underground features such as tunnels and pipelines obtained from various utility companies and facility owners,

(g) site reconnaissance records of major geological features, and

(h) CMP.

The Hong Kong Geological Survey (HKGS) is responsible for maintaining up-to-date geological maps of Hong Kong and the Geoscience Database. The HKGS has published many geological maps and memoirs. The Geoscience Database contains a bibliography of Hong Kong geology, geological and photographic collections. The Geotechnical Information Unit (GIU), located in the Civil Engineering Library (CEL), is a data bank for all geotechnical information collected by the GEO and is open to members of the public. Ground investigation and laboratory testing reports can be obtained via the Digital Geotechnical Information Unit (DGIU) system in the CEL. Many HKGS maps and publications are now available digitally and some can be downloaded from the CEDD Website. HKGS shall be consulted on geology and geological records of past projects involving tunnel works.

### 5.4.3 Aerial Photograph Interpretation

Aerial photographs, particularly when examined stereoscopically, can often be used to identify and delineate specific ground features that need to be assessed for cavern developments, particularly portals and caverns. They illustrate the spatial distribution of different soil types, soil thicknesses, bedrock types, depths to bedrock, discontinuity patterns and spacings as well as local relief. Aerial photograph interpretation (API) is useful for mapping *photolineaments*. This term refers to linear or gently curving features that are usually the surface expression of variations in the structure or types of the underlying bedrock. Well-defined linear depressions usually indicate the location of less resistant bedrock or of discontinuities in the bedrock structure such as faults, fracture zones or major joints. Local linear topographic highs or lines of boulders may indicate the presence of rock that is more resistant to weathering.

The hillsides of Hong Kong often have a dense cover of vegetation and a mantle of weathered rock or colluvium, both of which tend to obscure features of interest. Although major zones of weakness, including faults, can usually be located on the photographs, minor weakness zones are frequently concealed. Less prominent structures, such as joint patterns, cannot always be identified. Nevertheless, API can normally give a good indication of the major rock structures and thus is a very useful tool for developing the rock mass characterisation.

Aerial photographs can also assist with the identification and study of old or new/active landslides, rock outcrops and boulder fields. Landslides and rockfalls may be related to the presence of important geological structures such as weakness planes and/or soil or rock type boundaries. Such structures should be considered both with respect to location and necessary overburden of underground excavations and also with respect to location of portal/entrance areas and other above-ground site usage.

The use of aerial photographs can also aid the identification of hydrogeological features such as drainage patterns and spring lines to locate obstructions and other features that may affect the portal locations. API is one of the main methods to identify potential natural terrain landslide hazards.

## 5.5 Planning of Ground Investigation

### 5.5.1 Approach

(1) *General*. The ground investigation required for a cavern development project should cover the locations of the caverns, portals, access tunnels and shafts. It is aimed at establishing the geology and groundwater conditions of the proposed site as well as identifying adverse ground conditions, such as major discontinuities and zones of weakness.

A ground investigation is used to develop and refine the preliminary geological model, and identify uncertainty and risk (see Section 6.6.4). Two or more stages of investigations are required for most cavern projects. Each stage should be designed on the basis of a postulated ground model to provide data for refinement.

(2) *Planning Stage*. In the project planning stage, it is sufficient to have a general picture of the subsurface geology and hydrogeology in order to confirm the project's engineering and economic feasibility, to identify suitable cavern sites for obtaining public acceptance, and to estimate the likely project cost. Only a limited amount of ground investigation is required to support the findings of the desk study, site reconnaissance and API.

(3) *Preliminary Design Stage*. In the preliminary design stage, recommendations are made on the preferred cavern orientation, shape and layout. A ground investigation comprising a geophysical survey, some widely-spaced drillholes and field measurements (e.g. in-situ stress measurements and groundwater monitoring) is required. Locations of drillholes should be planned after the desk study, site reconnaissance and API with an aim to refining the geological model and gaining additional information on significant subsurface features.

(4) *Detailed Design Stage*. In the detailed design stage, when the cavern development site has been confirmed, the main aim of the ground investigation is to further reveal conditions at problematic areas along the carven alignment. The investigation data should be sufficient for designing the ground support, ground treatment, groundwater control works and any risk mitigation measures. The investigation should cover an assessment of the hydrogeology of the site including the hydraulic connectivity of the discontinuities that may intercept the proposed cavern. It is also necessary to assess the need for ground treatment and groundwater control works to prevent excessive drawdown of piezometric pressures in the rock and the soil overburden.

It is important that consideration is given to minimising the environmental impacts of the investigations, especially when carried out within Country Parks, Sites of Special Scientific Interest and Geoparks.

### 5.5.2 Ground Investigation Methods

A ground investigation should progress from low-cost, simple surface observations to more costly deep drillholes and specialist testing. Geoguide 2 (GEO, 2017b) describes a range of techniques which can be considered for investigating a cavern site. The typical methods and sequence are:

(a) engineering geological field mapping,

(b) geophysical surveying,

(c) soundings to bedrock, with or without sampling,

(d) core drilling and sampling, and

(e) in-situ and laboratory testing.

Details of additional investigation methods that may be applicable to cavern developments have been described by the Norwegian Tunnelling Society (NFF, 2006).

Digital maps on a geographical information system (GIS) platform can be used to analyse the information collected and identify features of potential interests such as regional geological structures.

In a ground investigation, it is imperative that the quality of equipment, experience of operators and accuracy in workmanship are maintained at high levels. For core drilling in a rock mass, it is important that the drilling procedure is continuously adapted to the actual ground conditions in order to recover as much material for the undisturbed samples as possible.

### 5.5.3 Information to be Obtained

A ground investigation should proceed from identification and studies of large-scale features that may dominate engineering and economic feasibility to features that would normally have minor effects. In particular, it is important that the information required to characterise the rock mass and the groundwater conditions are obtained. Priority must be given to obtaining information on:

(a) rock type,

(b) detailed discontinuity characteristics,

(c) location, orientation and detailed characteristics of weakness zones,

(d) depth and nature of weathering, and

(e) groundwater conditions and field permeability.

In cases where rock exposures are present, measurements from the engineering geological mapping (Section 5.6) may provide valuable information for the preparation of a preliminary geological model for a cavern design, especially for the design of the cavern layout and orientation.

At the later stage, it may be desirable or necessary to obtain site-specific data on some or all of:

(a) rock strength,

(b) in-situ stresses,

(c) rock deformation characteristics, and

(d) chemical properties of rock and groundwater.

Most rocks in Hong Kong are strong when fresh and thus strength measurement for intact rock has a low priority in a ground investigation. Rock strength and abrasiveness may, however, be of importance from the point of view of drillability and excavability. Typical uniaxial compressive strengths (UCS) for common rocks in Hong Kong are given in Table 5.1. Testing for drillability is discussed in Section 5.8.11.

**Table 5.1 Typical Uniaxial Compressive Strengths of Hong Kong Rocks**

| Material Decomposition Grade | Uniaxial Compressive Strength (in MPa) | | | | |
|---|---|---|---|---|---|
| | Granites | Volcanics | Granodiorite | Quartz Monzonite | Yuen Long Formation Marble |
| Fresh | 80 - 150[7]<br>101 - 179[4]<br>150 ± 25[6]<br>200[1]<br>[$Is_{50}$ = 4 to 11[7]] | 150 - 340[2]<br>150 - 250[7]<br>can be above 300 and up to 400[1]<br>[$Is_{50}$ = 6 to 14[7]] | 125 - 175[7]<br>170 - 290[2]<br>150 - 200[5]<br>[$Is_{50}$ = 7 to 11[7]] | 100 - 150[7]<br>300[3]<br>[$Is_{50}$ = 5 to 8[7]] | 40 - 140[1]<br>up to 190 where silicified |
| Slightly Decomposed | 101 - 179[4]<br>82 ± 22[6]<br>100 - 150[1] | 110 - 190[2]<br>can be above 300[1] | 120 - 215[2]<br>75 - 175[5] | - | - |
| Moderately Decomposed | 85[4]<br>33 ± 9[6]<br>10 - 80[1] | 10 - 120[2]<br>Grade III < 50[1]<br>Grade III/IV < 25[1] | 10 - 145[2]<br>15 - 100[5] | - | - |

Notes: (1) Table based on:
[1] GEO (2007),
[2] Irfan (1985),
[3] Irfan & Nash (1987),
[4] Irfan et al (1991),
[5] Irfan & Powell (1984),
[6] Roberts (1991), and
[7] GCO (1989).
(2) $Is_{50}$ refers to Point Load Strength Index.
(3) Numbers after ± are standard deviations.

In addition to the factors listed above, rock temperature can be important in the design of air conditioning and ventilation systems and for the design of heated, chilled and deep freeze stores. Guidance on the measurement of rock temperature is given in Section 5.8.10.

## 5.6 Engineering Geological Field Mapping

Geological mapping for cavern and tunnel projects emphasises on rock structures. Particular attention should be paid to locating and describing zones of weakness, such as faults and heavily jointed or crushed zones. The mapping should focus on providing information for estimating the rock conditions at depth and on surface features that may affect the construction of the surface components of a scheme, including portals. Trenching and other surface excavations will be most useful in assessing the surface conditions and may facilitate the understanding of the solid geology.

The area covered by the field mapping should be sufficiently large in order to identify the main geological structures of the proposed site. The mapping should be carried out by an engineering geologist who has experience and understanding of the implications of geological features for cavern design and construction.

Guidance given by Geoguide 3 (GEO, 2017a) should be followed for the description of rocks, rock masses and discontinuities.

A good starting point for geological mapping of the site would be the relevant topographical maps at a suitable scale, normally 1:1,000, aerial photographs and 1:20,000-scale geological maps. The mapping should provide data on rock types including strength, colour, texture, weathering and alteration, structure and discontinuities including location, orientation (with respect to north), spacing, persistence, roughness, aperture, infilling and seepage. Where fresh rock is exposed, rock mass classification systems such as the Q-system, Rock Mass Rating (RMR), Rock Mass Index (RMi) and Geological Strength Index (GSI) (see Chapter 6) may be used.

Rock mass classification from surface exposures of weathered rock should be used with caution since such rock may not be representative of the conditions at depth, for reasons including weathering and washing out (or washing in) of discontinuity infill. In addition, judgement should be exercised when using discontinuity information collected from surface exposures, especially the orientations and spacings of discontinuities, as they may vary with depth. An assessment on the variation of discontinuity information collected at surface exposures versus that collected at depth from drillholes may be useful.

The data collected should ideally be incorporated into a 3D model or at least be depicted on plans and sections. Discontinuity data may be represented graphically as discontinuity rosettes or polar diagrams (Figure 5.1). Discontinuity rosettes lend themselves to easy visual interpretation, but polar diagrams give more complete information and can be used analytically (Hoek & Brown, 1980).

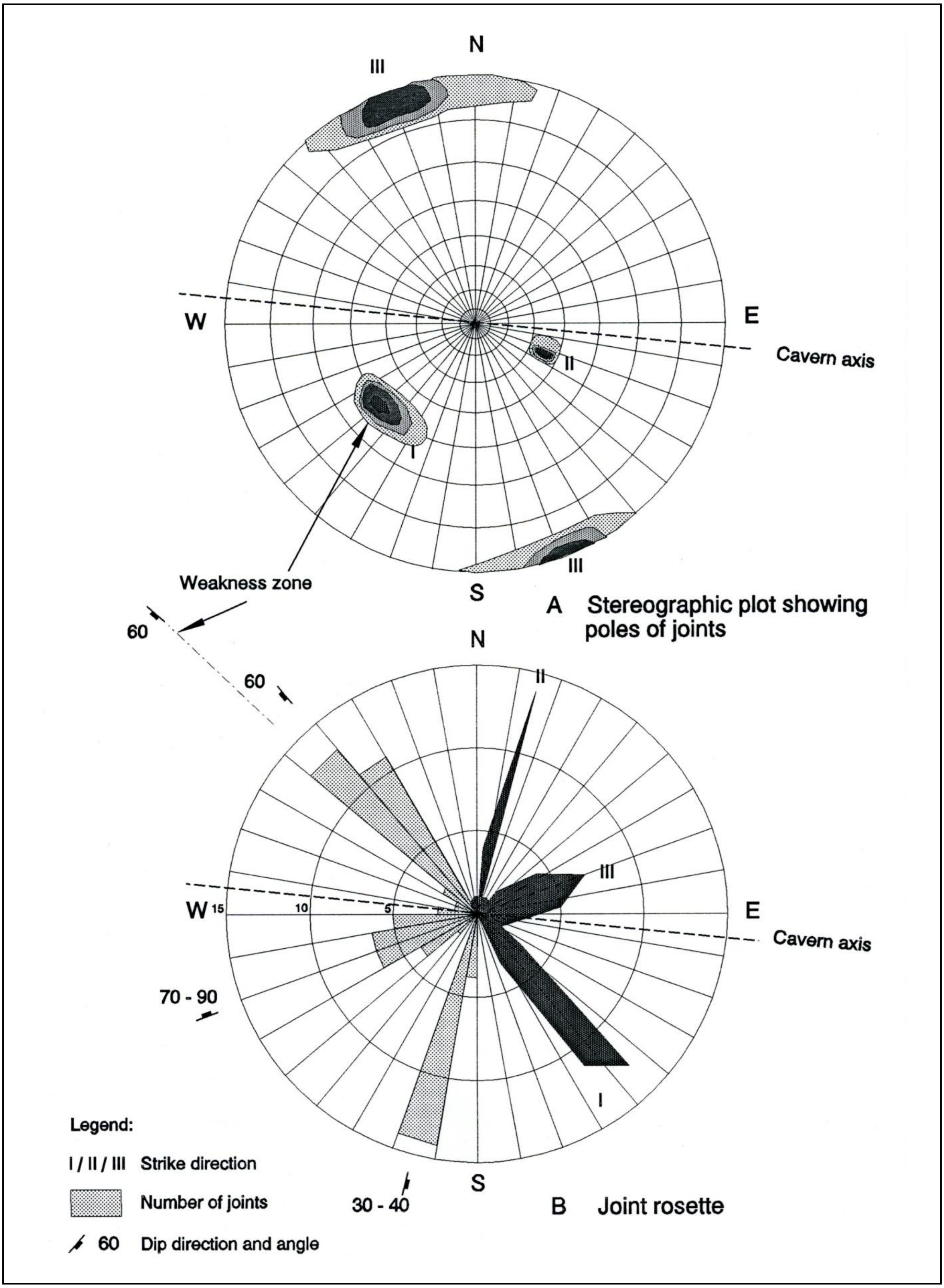



**Figure 5.1 Polar Stereographic Projection and Discontinuity Rosette**

## 5.7 Field Investigation

### 5.7.1 Extent of Ground Investigation

(1) *General.* Chapter 10 of Geoguide 2 (GEO, 2017b) gives general guidance on the extent of a ground investigation that is required.

(2) *Geophysical Surveying.* The extent of geophysical surveying depends on the size of the scheme, the complexity of the geology and uncertainties in the geological model developed from the engineering geological field mapping. The extent of geophysical surveying should only be assessed after the results of the geological field mapping are available.

For seismic refraction survey, the area covered should always be greater than the area under investigation. The extent actually carried out is commonly found to be greater than that thought necessary prior to commencement, and generous allowances for additional lines should be made. However, it should be noted that conducting seismic refraction survey on vegetated hilly terrain can be challenging and may not yield useful results.

Geophysical investigation must not be used alone or without a reasonable amount of reliable ground data for correlation. Further information on geophysical investigation methods is provided in Section 5.7.2.

(3) *Drillholes.* The number of drillholes required depends on the geological and ground models developed from the mapping data and the geophysical surveys. The number must be sufficient to resolve uncertainties in the models, particularly the location and orientation of weakness zones, and to give data on the properties of the rock in the weakness zones at appropriate elevations, as well as data on the properties of the rock mass in general. As most weakness zones dip steeply, inclined drillholes should be used extensively in most investigations.

Drillholes should be deep enough for the proposed cavern development. Some holes should penetrate below the lowest probable base level of the proposed installation by at least 5 m and may extend to half the cavern span or more, depending on the uncertainties in the cavern location at the time of the investigation and the geological factors. Investigation holes should be taken to a generous depth below the proposed invert level to cover the zone of influence beneath the cavern.

Often the orientation or even the location of a cavern may be shifted during the preliminary design and optimisation phase. In some cases the design is based on results from site investigations designed and carried out for a previous alternative layout or location. Although prior investigation results may still be valid for the new location, supplementary investigations should be carried out. Due consideration should also be given to the risks of unforeseen ground conditions.

### 5.7.2 Geophysical Investigation Methods

(1) *General.* Chapter 33 of Geoguide 2 (GEO, 2017b) gives an outline of the

capabilities of various geophysical investigation techniques used in site investigations. Geophysical investigation methods are used to make a preliminary and rapid assessment of site conditions and to supplement the surface mapping of a skilled engineering geologist. Subsequent drilling should provide correlation and confirmation of the interpretation.

Of the techniques available for preliminary assessments, seismic refraction surveying has proved, in the hands of expert practitioners, to be a powerful tool for investigating rock conditions. Barton (1996, 2002 & 2006) and Barton & Grimstad (2014) provided guidance on the interpretation of seismic velocity for the assessment of rock mass conditions using Q-value.

Cross-hole seismic surveys have been used to investigate rock conditions. With the aid of tomographic data processing, a detailed picture of the rock can be developed. The method can be adversely affected by some ground conditions, such as very high velocity contrasts between different materials. The method is best applied to investigating specific problems in limited areas as it is time-consuming and expensive, not least because of the drillholes needed.

Other geophysical investigation methods have been employed from time to time. Microgravity can be useful for the assessment of depth to rockhead. Electrical resistivity measurements can reveal weakness zones and details of superficial deposits, and detect groundwater table. Ground penetrating radar can reveal rock structures where the depth to rock is small, say less than 10 m, but the presence of clay minerals can severely limit penetration. Salt water in the rock effectively prevents radar penetration. Borehole radars with tomographic data processing can give useful information on rock structure, but the method is expensive and is not in common use.

In general, no geophysical method should be completely relied upon in design, without being supported by adequate subsurface data.

(2) *Seismic Refraction Surveying.* The geophysical investigation technique most commonly used for cavern engineering is the seismic refraction method. The rock mass quality, the position of weakness zones in the bedrock and the depth of overburden can all be determined by this method. Within the overburden, layering can frequently be revealed and an indication of the probable nature of the material in the layers can be given. The data collected gives the basis for optimising the location, orientation and depth of the drillholes.

Seismic refraction survey lines should be connected to form a closed loop or grid. The number of survey lines and the amount of data collected should be sufficient for interpretation of the results, in particular, the low velocity zones should be able to be located with confidence. Sufficient data should also be collected to give a firm indication of the depth of weathering for a site as a whole, with appropriate details along the proposed access tunnels and portals. Seismic refraction surveys only show weakness zones that extend up to the bedrock surface. Weakness zones that terminate against other weakness zones at depth cannot be revealed.

The method can be rapid. The field work consists of placing geophones at fixed spacing in a straight line on the ground surface, applying an energy source at selected points along the line of geophones and recording the arrival times of the vibrations at the geophones. In ideal conditions, one team can survey three to four layouts of about 200 m in a day and the interpretation of the data can be available within a few hours of completing the field work.

Seismic refraction surveying is based on the phenomenon that the velocity of seismic waves (sound waves) varies with the type of material through which the waves propagate. The success of the method depends on there being differences in velocity in different materials and that the velocities in successive layers generally increase with depth. The contrast in velocity between layers causes the seismic waves to refract (according to the same laws by which light is refracted) and these refracted waves are used for the interpretation of the geology. The principles of the method are illustrated in Figure 5.2. The method allows interpretation of the depth to various layer boundaries and the variation of these depths across a site. Thus a subsoil profile can be drawn with rockhead shown. Lateral variations in velocity within the rockhead can be measured. These low velocity zones, which imply deeply weathered or heavily fractured material, are commonly the surface expression of faults and other zones of weakness and are the most important data obtained from the surveys. However, complications have been encountered with the definition and interpretation of rockhead due to corestone development.

The results of a seismic refraction survey are presented as cross-sections showing the various layers and zones with the velocities indicated (Figure 5.3). The location of the various velocity zones in the bedrock is indicated on plan with normal (high), intermediate and low velocity zones differentiated by legends. Three-dimensional variation of the bedrock profile can be established by running the survey lines in several directions.

The depths of superficial deposits and weathered rock can be considerable, and a high energy source is required to ensure sufficient propagation of the seismic waves to the intact rock and back to the geophones. Poor energy propagation conditions are frequently experienced, particularly where the groundwater table is low. Explosives are commonly the only practical source of energy at locations with difficult access. Where vehicular access is possible, drop weights may be used. Permits for the use of explosives must be obtained from the GEO. The depth of the holes may be 1 to 2 m in rural locations and up to 10 m in urban areas to ensure adequate safety zones for installations in the ground. Restrictions on proximity to structures, services and sensitive receivers may have to be imposed. The size of the charge is normally small and is usually less than one or two kilograms. Depths of penetration can exceed 80 m if sufficient energy is available. The method works as well on hillsides and in rugged terrain as it does on flat ground.

At locations with significant background vibrations (i.e. noise), the noise-reduction process known as stacking may be used. Multiple records are made and added together (stacked) such that the random noise is cancelled out and the wanted signal is enhanced. At such locations, drop weights may not give a sufficient signal to noise ratio to produce meaningful results.

The accuracy of the method is normally good, with depths to refractors measured to within 10% or 1 m, whichever is the greater.

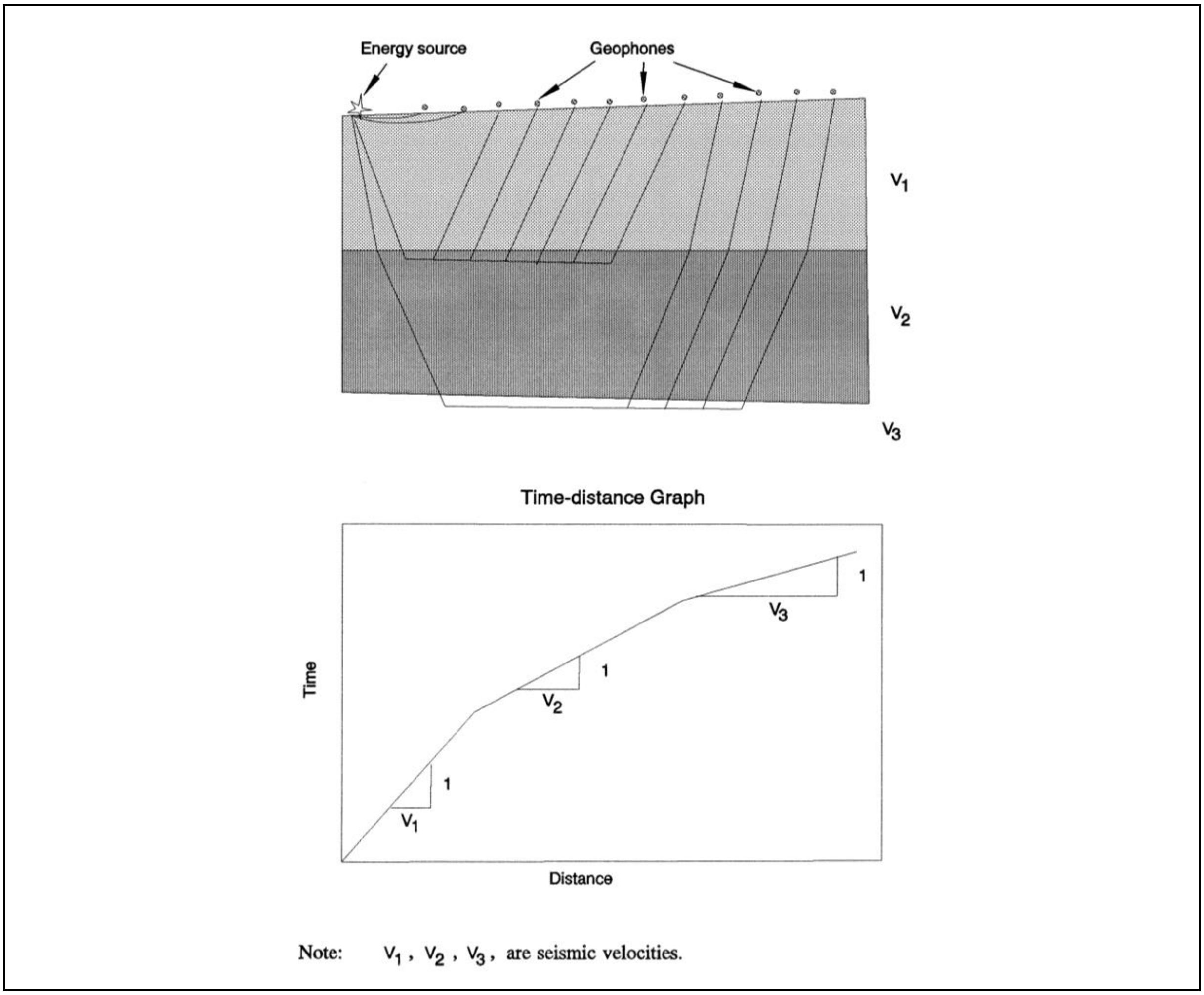



**Figure 5.2 Principles of Seismic Refraction Surveying**

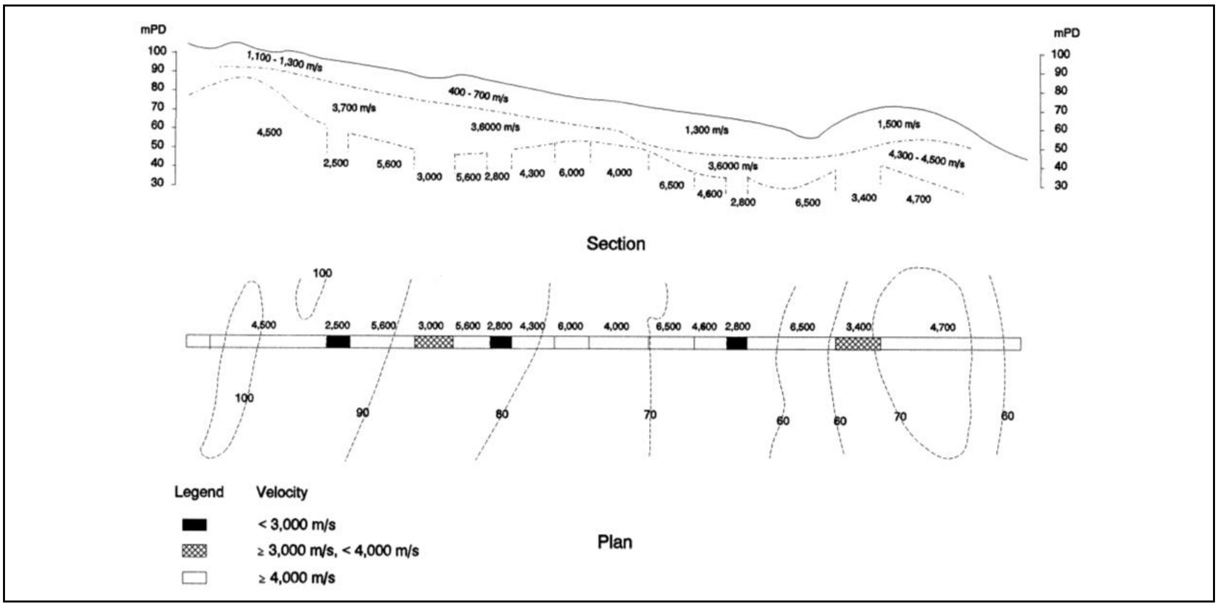



**Figure 5.3 Typical Results of a Seismic Refraction Survey**

Seismic refraction surveys, combined with drillhole investigations using the core drilling technique, have been found to work well in Hong Kong for the assessment of rock mass conditions. Seismic refraction modelling using suitable software may be carried out for extracting useful information from the seismic refraction survey data.

(3) *Cross-hole Seismic Investigation.* The cross-hole seismic method can be adopted to obtain data pertaining to the structures of large volumes of rock mass. The method provides information on the cross-sections of the rock between drillholes, or between a drillhole and an exposed surface, such as a tunnel or the ground surface.

Cross-hole seismic surveys are carried out between drillholes orientated in the same direction. As this requirement is frequently at variance with the drillhole locations required to discover the geology of the site, additional drillholes will normally be required for such surveys. The total cost of cross-hole seismic surveys is relatively high and it should therefore be used only where sufficient information cannot be obtained by other cheaper means. In addition, it is difficult to carry out in densely populated urban areas. The cost of this investigation must be justified by the potential saving in construction costs resulting from the information gathered. Cross-hole seismic surveys, with tomographic processing of the data, are therefore usually used to provide a more detailed picture of the rock structures, which is proved to be critical for successful construction.

Traditional cross-hole seismic investigation commonly gives only average seismic velocities between drillholes and variations with depth. Computerised acoustic tomography uses the cross-hole technique and can produce a detailed picture of the rock structures (By, 1987). Figure 5.4 shows the results of a cross-hole seismic survey with tomographic data processing. From a single source, signals are transmitted to receivers in two or more other drillholes or exposed surfaces and then transferred to a data acquisition system. The spacing between drillholes for a cross-hole seismic investigation may be controlled by various factors, such as the type of seismic source, the depth of the drillholes and the velocity differences between strata, and should be specified depending on the site settings and conditions.

(4) *Gamma Density Method*. The gamma density method can be used as a supplementary technique to help identify subsurface weak layers. The principle of the method is to irradiate the target material with medium-to-high energy gamma rays and to measure their attenuation between the source and the detector. The attenuation is a function of the electron density of the target material, which in turn is closely related to its mass density.

Based on the relative density contrast between target materials, the technique can be used, within a drillhole, to identify weak layers in the ground at a practical logging speed of 1 m/min. Such weak layers include clay-rich layers, weathered seams and disturbed zones that are of comparatively lower mass density. The resolution of the method increases with the increase in relative density contrast between the target and the adjacent materials. The resolution decreases if casing is used and as the dip angle of the weak layer becomes aligned with the drillhole axis (i.e. sub-vertical in a vertical drillhole).

GEO (2004) provides some guidance on the use of this method.

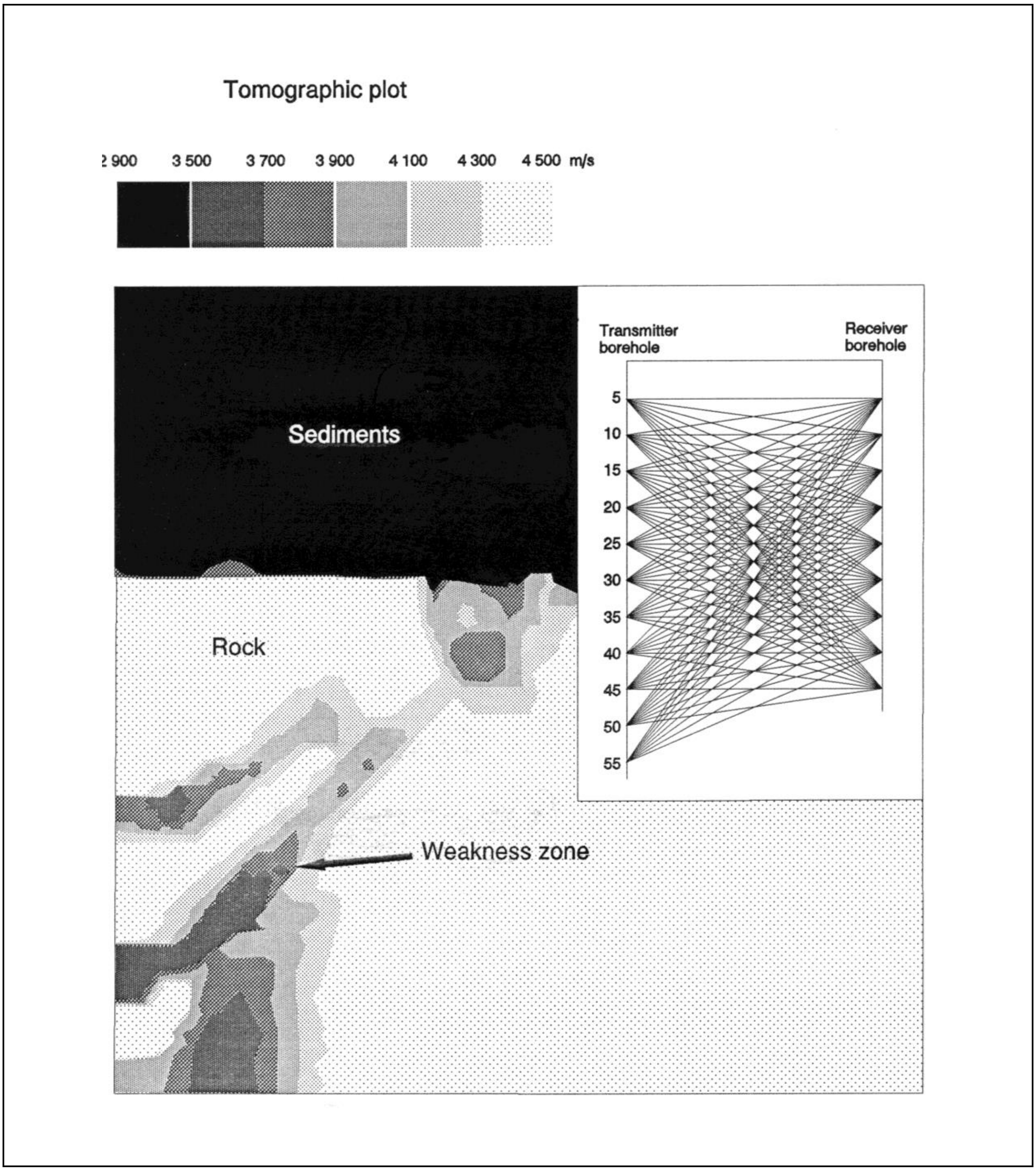



**Figure 5.4 Typical Results of a Cross-hole Tomographic Survey (By, 1987)**

(5) *Spectral Gamma Ray Method*. Another downhole geophysical method to collect supplementary information about subsurface weak layers is the spectral gamma ray method. This method is based on the principle that decomposition of potassium-bearing minerals leads to a progressive loss of potassium ions (K). Naturally occurring potassium contains radiogenic, K. Thus the amount of radiogenic, K, present in a material is related to the degree of decomposition of potassium-bearing minerals in the parent rock and hence the degree of weathering of the rock mass. The spectral gamma ray method produces a log of the potassium count rate along a drillhole.

The location along the drillhole where the count rate shows a significant reduction compared to the adjacent materials can be interpreted as a more weathered, weak layer. However, interpretation of the data is dependent on the origin of the target material, which may significantly affect the potassium count rate. For instance, where a clay layer does not originate directly from the decomposition of the adjacent materials, the potassium count rate of the layer may not necessarily be lower than those of the adjacent materials. Also, if thin layers are to be identified, a slow logging speed will be required, e.g. 0.05 m/min for a resolution of 50 mm.

This method does not require the use of any radioactive source and can be used in drillholes lined with different types of casing. In addition, it is generally cheaper than the gamma density method. The spectral gamma ray method, when applied to materials of the same origin and preferably backed up by site-specific calibration, can give an indication of the degree of weathering. Although the method is not as good as the gamma density method in identifying thin weak clay-rich layers, there is potential for it to be used in verifying ground conditions, especially in the case of a large number of drillholes, without the need for sampling, e.g. drillholes for installing soil nails, raking drains, observation wells and piezometers, etc. Further guidance on the use of this method shall refer to Geoguide 2 (GEO, 2017b).

(6) *Others.* Neutron porosity method may be useful in identifying weak layers in saturated ground. However, in unsaturated ground the method may give misleading results.

Microgravity method has been carried out in Hong Kong to provide an indication of rockhead at slope excavations and at portals for tunnels. This technique may be considered when a good contrast of densities between soil and rock is anticipated, and when the rockhead is deeper and has an abrupt change.

2D resistivity survey can reveal the physical properties of rock mass below the surface. Interpretation requires good geological knowledge of an area. The best resolution is achieved at depths of 50-70 m below ground. Electrodes are located at 10 m intervals along the cable and cable lengths in multiples of 200 m, up to a total of 800 m may be used. The results from a 2D resistivity survey may be used in combination with other methods of investigation, but cannot replace seismic refraction survey.

Other methods such as cross-hole seismic surveying with tomography and trial adits and shafts with large-scale testing may also be worthwhile in some circumstances.

### 5.7.3 Drillhole Investigation

Drillhole investigation is the most common and direct way of examining rock mass at depth. Because of the limitations of field mapping and API, a drillhole investigation, with or without preceding geophysical surveys, will generally be necessary for a cavern design. The methods employed in drilling are described in Sections 18.7 (rotary core drilling), 18.9 (backfilling boreholes) and 19.8 (rotary core samples) of Geoguide 2 (GEO, 2017b).

Problems can occur with rock collapsing into drillholes and procedures to stabilise such holes should be included in a ground investigation contract. Stabilisation of a hole commonly entails grouting and re-drilling the collapsed section. Permeability testing, if required, of the hole down to the collapsed section must be done prior to grouting to prevent the grout from affecting the permeability test results. Other problems include situations where a hole is drilled over long lengths, resulting in large strain on the equipment due to accumulated friction and weight of the rods. Inadequately grouted drillholes can cause problematic groundwater inflows into underground excavations. Therefore, drillholes must be fully grouted upon completion to avoid water inflow problems.

High-quality core recovery is essential, and requires selection of an appropriate hole diameter and suitable core barrels. The likely rock strength and rock structure, in addition to the depth to be attained, will influence the choice.

Drillhole diameter is governed by the drill depth, the minimum size of core required, and the dimensions of test equipment to be inserted. The minimum core diameter should be not less than 50 mm. Smaller diameters can be considered for deep holes in good rock if there are considerable cost advantages.

A drillhole log should give comprehensive information as described in Geoguide 2 (GEO, 2017b), including as much details as possible on the discontinuities and the required data for use with rock classification systems. Colour photographs of the cores should also be taken as a permanent record, even though cores are normally stored at least until the completion of construction. It is important that cores are stored in an environment that ensures their preservation.

Inclined and horizontal drillholes can be used to target specific features of interest such as faults and soft ground sections. Inclined drillholes are generally not a technical challenge; however, it requires suitable equipment and experienced personnel to operate the rigs. Benefits of inclined drillholes include the following:

(a) They can remove a bias of vertical holes where horizontal and slightly inclined discontinuities are over-represented in the samples, providing that the use of inclined and horizontal drillholes does not lead to a relative deficiency in the length of vertical drillholes, i.e. overall length of vertical and horizontal drilling needs to be approximately equal to reduce bias.

(b) They provide more data on vertical discontinuities as there is preferential intersection of sub-vertical discontinuities which can be more prevalent in igneous rocks, particularly at greater depths.

(c) They can penetrate to areas that vertical drillholes may not be able to reach.

(d) They can investigate suspected vertical faults and features of interest to gauge thickness and, if used in combination, it may be possible to identify orientations (see Section 5.8.3).

However, groundwater monitoring may not be possible in shallow inclined drillholes.

Alignment of drillholes could be deflected by some difficult ground conditions, e.g. steeply inclined soil-rock interface. The accuracy of a drillhole alignment may be improved by the use of directional core drilling (DCD), a technology permitting control of the direction of long drillholes. The technology makes it possible to drill in a forced straight line as well as controlling a drillhole in one or more curves, hitting the target with very high precision. The technology generally consists of a forced steering mechanism of the drill string combined with a logging function that maps the exact position at any point along a drillhole. An example of a DCD core barrel is shown in Figure 5.5.

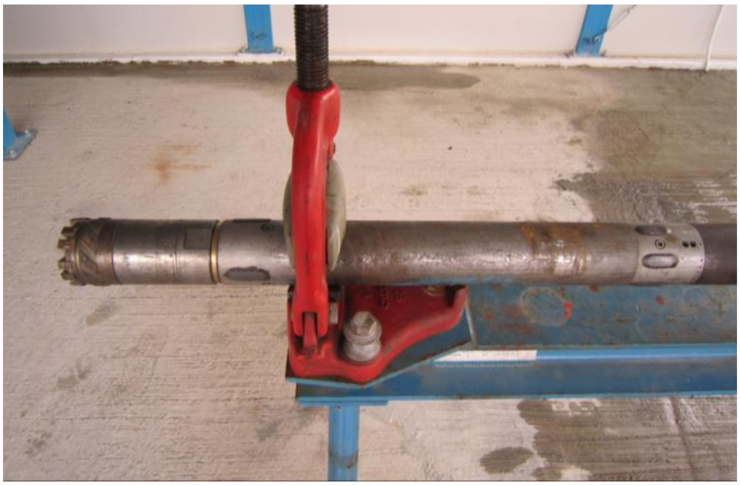


**Figure 5.5 Directional Core Drilling (DCD) Equipment**

DCD has been carried out for various projects in Hong Kong (e.g. Po Shan Drainage Tunnels for landslip prevention and mitigation works, Tsuen Wan Drainage Tunnel and recent DSD subsea tunnels) along the tunnel alignment to determine the ground conditions and assess potential inflows during construction. DCD is capable of taking continuous core samples. Adverse ground conditions if present along a tunnel alignment may be revealed, and hence help reduce engineering geological uncertainties and risks.

The cost of DCD can be high. In order to avoid abortive work, it may be more appropriate to carry out DCD at a later stage of the feasibility assessment or at the detailed design stage. DCD must be carefully planned such that the time required is allowed in the project programme.

Normally DCD holes can be of a length of 200 to 300 m. Given suitable equipment and experienced operators, drillholes in the range of 600 to 800 m are well within reach. The longest DCD hole sunk in Hong Kong is over 1,000 m. However, drilling time and thereby cost (in addition to the risk of drillhole failure) increase with drillhole length.

The directional coring together with pumped down packer tests may provide useful information on the geology and hydrogeology along a tunnel alignment which may not be obtained from vertical or inclined boreholes.

Apart from packer tests, rates of groundwater inflow into DCD holes may be useful for estimating the inflow rate into advance probing at the construction phase. Lo & Cheuk (2006) presented a case study of the Eagle's Nest Tunnel construction. Based on the field measurement data, they demonstrated that the water inflow rates in DCD and advance probing were of the same order of magnitude.

Collapsing ground and difficulty in drilling through fault zones can occur when using DCD. A particular risk that should be considered is the loss of the drill string in a drillhole. There have been cases that significant lengths of drill string cannot be retrieved from DCD holes. In some circumstances, the drill string had been left within the tunnel/cavern envelopes. This can cause significant problems for construction. Therefore, it is recommended that this risk is mitigated by keeping DCD holes close to, but outside, a proposed cavern envelope.

Further guidelines on the use of DCD in Hong Kong are provided in AGS(HK) (2012).

Probe drilling, with instruments installed to obtain measurements of drilling parameters (including drilling rate, thrust and torque), may be carried out to assess the rock mass conditions ahead of an excavation. Section 7.7.3 provides further guidance on the technique.

### 5.7.4 Groundwater Investigation

An understanding of the groundwater conditions affecting a cavern development is essential for assessment of groundwater inflows into underground excavations, surface settlement, effects on water catchments, aggressive groundwater, contamination, containment for certain cavern uses and water pressures on discontinuities and linings.

For a groundwater assessment of a cavern project, the ground investigation should be targeted to define the permeability profile of the soils and rocks within the area of tunnels or underground space. Usually, long-term monitoring is required to define seasonal variations in the groundwater level over at least one dry and one wet season. The use of remote data logging equipment for drillholes can be incorporated into the investigation programme to provide sufficient information for the impact assessment for the project. Predictions of groundwater changes are considered on the basis of rock mass permeability, groundwater levels, recharge and drainage into the excavations. This section deals with water table and piezometric observations. Section 5.8.2 discusses permeability measurements in detail. A groundwater investigation should include:

(a) desk study and API,

(b) field mapping,

(c) piezometers,

(d) variable and constant head tests,

(e) packer tests,

(f) pumping tests,

(g) dyes and tracer materials,

(h) downhole temperature sondes in unlined boreholes,

(i) geophysical investigation,

(j) geochemical and contamination laboratory testing, and

(k) observations and tests carried out during and after construction.

Geoguide 2 (GEO, 2017b) describes methods of determining groundwater pressure. Water level observations in investigation holes give the first information on groundwater and they form the basis for specifying the location and type of permanent groundwater level and pressure-measuring devices. These devices are piezometers of various types. Commonly standpipe piezometers are suitable, but piezometers with a shorter response time may be required from time to time.

Pumping tests may be required in certain areas to provide data for verifying the groundwater model and the mass permeability characteristics of the ground.

In addition to pumping tests, a sensitive downhole temperature probe can sometimes be used to detect groundwater inflows into a drillhole down to at least 0.01°C differences. The discontinuities transmitting water can then be determined from the core and televiewer data.

Falling head and rising head tests should be carried out during construction and, wherever possible, continuous lugeon or packer tests should be carried out as drilling proceeds. In addition, the amount of inflow into probe holes should be recorded as correlations exist between probe hole inflow and the inflow into underground excavations. Downhole flowmeter may be used to identify high inflow zones.

### 5.7.5 Pilot Shafts and Adits

Pilot shafts and adits carried out in advance of a final design and full construction provide access to rock exposures, allowing the rock mass properties to be investigated in detail. They can also help to resolve particular problems of design or of construction planning.

The main objectives of excavating trial shafts and adits are:

(a) investigating the rock mass structure,

(b) obtaining detailed information on particular zones in the rock mass,

(c) in-situ testing for shear strength, deformability, permeability and rock stress,

(d) performing geophysical measurements,

(e) taking samples for laboratory tests, and

(f) making trial blasts to establish vibration criteria.

In addition, it is common to incorporate pilot headings and systematic advance probing and coring during construction to gather information on features of concern such as faults or shear zones prior to the main excavation encountering the features. Pilot adits may be useful for groundwater control by forming drainage galleries or galleries for pre-grouting. Furthermore, they can provide access for instrumentation and pre-reinforcement installation. In some projects, they served as pilot holes for raise-boring operations.

However, pilot adits are expensive and should only be used when the cost of excavations can be fully offset by the financial benefits resulting from the removal of conservatism otherwise required because of uncertainties in design parameters and constraints. These uncertainties are normally related to the characteristics of rock mass in critical areas, such as locations, orientations and strength and deformation properties of significant discontinuities and zones of weakness such as faults.

The possibility of carrying out pilot shafts or adits also depends on factors such as size of development, complexity of ground and whether there is enough time to identify and allow advance works to proceed.

## 5.8 Field and Laboratory Testing

### 5.8.1 General

The testing requirements must be evaluated for each scheme in terms of technical requirements, budgets and stages of project development. Some of the requirements can be met by drilling investigations, geophysical surveys and field mapping as discussed earlier in this chapter. The remaining items, including rock temperature, are discussed in this section.

### 5.8.2 Permeability Testing

Permeability testing is normally carried out as packer (water absorption) tests as described in Section 21.5 of Geoguide 2 (GEO, 2017b). The primary use of permeability test results is to assess the groundwater inflows that may be encountered during construction and to provide a rational basis for the design of mitigation measures. Permeability tests should be carried out systematically at the level of a proposed cavern construction.

Correlation of test results with drillhole logs can give an indication of the openness of fissures and thus a general idea of the state of stress and stress variations. Permeability can also be assessed using the results of variable and constant head tests carried out in drillholes.

Pumping tests should be performed where investigation of groundwater regime is required and are essential for unlined fuel and gas stores. Pumping tests are, however, not a substitute for packer tests.

Double packer testing can be carried out in vertical, inclined and DCD drillholes. This allows continuous permeability profiles to be made. In particular, DCD allows measurements to be made along the alignment of an underground excavation. There are risks with using a double packer in that the chance of not getting a good seal around the packer is higher. However, if done properly, double packer tests can be usefully applied in Hong Kong. The following points related to packer testing should be considered:

(a) The simplicity of carrying out packer tests and the comparability of results may be improved if the tests have a uniform length, for instance 3 m long.

(b) Packer tests should not only be carried out in areas of fractures or higher inflows but rather should be performed sequentially along a borehole to reduce the potential for bias.

(c) Packer test methodology should include the pressure loss due to water flow in delivery hoses, which can be significant for long lengths of hoses.

### 5.8.3 Discontinuity Orientation

The presence and character of discontinuities within a rock mass are likely to be the major influence on the stability of the underground excavations and hence they need to be examined in particular detail. Engineering geological mapping aims to collect relevant field data and provides geological assessment for the underground scheme, as long as the mapped rock areas are representative of the rock mass to be excavated. Field mapping should not be replaced by the convenience of obtaining discontinuity orientation information from drillholes. Analysis of discontinuity data needs to take into account the possibility of bias in the data due to the data sampling technique (e.g. orientations of the mapped surfaces and drillholes relative to the discontinuity orientations). Common drillhole techniques to gather discontinuity data are:

(a) impression packer test,

(b) core orientators,

(c) acoustic televiewer, and

(d) optical televiewer.

The impression packer is designed to make an impression of the sides of a drillhole wall on a wax-covered paper. This provides the information needed to determine the locations of discontinuities in the strata at different depths.

The impression packer consists of a split steel tube with the same diameter as the drillhole itself. The two halves of this tube are pressed against the sides of the drillhole by expansion of a pneumatic packer in the centre of the assembly. A thin layer of compressible rubber covers the outer surface of each half of the split steel tube. This provides a backing for the wax-paper which is attached by adhesive tape or rubber bands.

After taking an impression, the packer is deflated and reverts to its original diameter and the split steel tube closes by the action of a spring. The assembly is then easily removed or re-positioned inside the hole.

In the past the impression packer was the most commonly used technique to collect discontinuity data from drillholes in Hong Kong. However, its use has largely been superseded by the use of televiewers.

Core orientators are not often used in local practice. However, it is recommended that their use in conjunction with televiewers should be considered. This approach allows data on the physical nature of discontinuities logged from cores (e.g. roughness, wall strength and infilling) to be related to the more reliable and extensive televiewer orientation data, with one method complementing the other. Various devices for orienting rock cores are available. Craelius core orientator had been widely used (Hoek & Bray, 1981). This mechanical device is installed in a fixed orientation in a core-barrel. Upon retrieval of the core sample, the uppermost core segment is again matched against the core orientator and its relative orientation determined.

The acoustic televiewer (ATV) has been used extensively in Hong Kong. The ATV log provides a very-high resolution, sonic image of a drillhole wall. The tool consists of an ultrasonic transducer coupled with a downhole inclinometer. These devices are used to generate an oriented image of seismic velocity variation and wave amplitude. These images are then examined and highlighted to reveal fractures, bedding planes and orientation of those features. The log is useful for strata and fracture delineation, and can also be used to evaluate compressional-wave velocity, drillhole deviation and eccentricity.

The ATV log is extremely useful where orientations of failure surfaces, fractures and bedding are critical for design. The log reduces the need for oriented cores or manned-hole logging, with a resulting reduction in cost, and can be operated through drilling fluids. The generated image is extremely detailed, with millimetre resolution, and may be presented as a flat, “unwrapped” diagram or as a three-dimensional “pseudo-core”. Attitudes of features may also be presented on stereonet plots to aid assessments and kinematic analyses.

The travel time of the acoustic pulse is used to determine exceptionally accurate drillhole diameter data (> 0.1 mm), which makes the tool ideal for drillhole deformation description (stress field analysis, including breakout analysis) and casing inspection.

The amplitude of reflection from a drillhole wall is representative of the acoustic (elastic) properties of the surrounding rock. Therefore, the tool is suited for detection of fractures and thin beds/seams in a rock mass, lithological characterisation, and geotechnical rock classification. In addition, the ATV may be used to inspect casing, and cement bond quality may be checked if the reflection signal from behind the casing is analysed. The tool must be run centralised (or in centralised PVC) in open, fluid-filled holes.

More recently the optical televiewer (OTV) has been used in Hong Kong. The OTV generates a continuous, 360° oriented image of a drillhole wall, using an optical imaging system. The tool includes an orientation package, which consists of a precision three-axis magnetometer and three-axis accelerometer package, allowing accurate drillhole deviation data and precise image orientation data to be recorded in the same log. It should be noted that OTVs are effective only when the wall of a drillhole is clean, as discontinuities are easily masked by a coat of fines or mud.

The tool provides detailed, oriented structural information including discontinuities, bedding and thin beds/seams and also allows lithological characterisation and casing inspection. It is particularly applicable where an ATV cannot be utilised (for instance above the groundwater table or within a borehole that cannot be filled up with water). The tool must be run centralised in air or water-filled holes.

### 5.8.4 Borehole Surveying

Deep boreholes, particularly when inclined, can significantly deviate at depth from the intended line, both in dip and bearing. Correct interpretation of the location of rock structures and the orientation of discontinuities is dependent on knowing the actual location and orientation of a hole. The boreholes may therefore have to be surveyed.

The types of instruments available for surveying boreholes are as follows:

(a) *Gyroscope instruments* – these instruments can be used in drillholes subjected to magnetic disturbance, for example in cased holes.

(b) *Magnetometers* – downhole three-axis magnetometers can be used to determine the azimuth.

(c) *Inclinometers* – where the inclination of a hole is measured by electronic accelerometers.

These instruments are often employed in combination. For instance, magnetometers are often used in DCD to measure drilling azimuths in combination with inclinometers which measure drilling inclinations. The choice of methods is dependent on the availability and suitability of instruments, cost and accuracy required.

### 5.8.5 Rock Stress Measurement

The stresses which exist in an undisturbed rock mass are related to the weight of the overlying strata and the geological history of the rock mass. The main objective of in-situ stress testing is to check that stresses would not cause undue problems during a cavern excavation such as spalling (due to high stresses) or excessive fall of rock (due to low stresses). The latter case is the one that is most likely to pertain to cavern developments at shallow depths in Hong Kong. Further discussion on the influence of in-situ rock stresses on cavern design is given in Section 6.4.6.

The three main methods of in-situ stress measurement are listed below (Hudson & Harrison, 1997):

(a) over-coring, where the rock stresses are found by measuring strain relief of a section of rock at the bottom of a drillhole when an annulus of rock surrounding it is removed,

(b) hydro-fracture (hydraulic fracture) testing, where intact rock is split by pressurising a section of the borehole, and

(c) flat-jack techniques.

The triaxial state of stresses can be assessed for every point of measurement in a drillhole using an over-coring method. Figure 5.6 (Hiltscher et al, 1979) illustrates the over-coring method proposed by the Swedish State Power Board (SSPB). With this method, tests can be performed in drillholes at any depth down to several hundred metres. However, the tests should not be used at depths of less than 100 m due to instrumental error, unless a large and statistically valid number of tests are made. Other methods such as the United States Bureau of Mines (USBM) overcoring torpedo and the Commonwealth Scientific and Industrial Research Organisation (CSIRO) gauge may suffer a limitation in their application to depths of less than 50 m (Ulusay & Hudson, 2007). Experience from the use of over-coring in Hong Kong has shown that it can be difficult to obtain a test section that is free of fissures, particularly in volcanic rocks. The results commonly display large variations in stress level and direction which are most likely caused by the discontinuities in the rock. A large number of tests should be conducted, perhaps 20 or more, to obtain a statistically significant result.

Hydraulic fracture tests can be done in both shallow and deep drillholes as shown in the schematic layout in Figure 5.7. A set of straddle packers is lowered into the drillhole and water is pumped into the un-cased section between the packers until the breakdown pressure is reached for cases where the test sections are free from fissures. The surrounding rock then fails in tension with fracture development. When the pumps are shut off with the hydraulic circuit kept closed, a shut-in pressure is recorded. This is the pressure just necessary to keep the fracture opens. The shut-in pressure is normally equal to the minimum principal stress. Finally, the direction and inclination of the fracture and hence the direction of the minimum principal stress (which is normal to the strike of the fracture) is determined using methods described in Section 5.8.3. A common assumption made during the interpretation is that the axis of the drillhole corresponds to the axis of a principal stress that is known. Normally the tests are carried out in holes that do not deviate from the vertical by more than 15°, with the further assumption that the principal stress corresponds to the overburden pressure. Based on these assumptions, the other principal stresses can be determined (Goodman, 1989). Assessment without the aid of these assumptions can also be made using a more sophisticated approach, but more tests are required (Hayashi et al, 1989). In fissured rock, existing fissures would open up during the test, and an interpretation of the triaxial state of stresses may be made (Cornet, 1986).

Flat-jack tests can be done in accordance with the method described by Ulusay & Hudson (2007), but are not commonly used as they require an exposed rock surface at depth. It is reported that flat-jacks were used successfully during the construction of the Aberdeen Road tunnels (Twist & Tonge, 1979), but there has been no report on its use in Hong Kong in recent years.

The cost of in-situ stress measurements by any of the above methods can be high and can be increased substantially in heavily fissured rock due to the high number of failed tests. In a well-designed ground investigation, the drillholes are positioned to locate and discover the nature of any poor rock present, i.e. the weakness zones, which increases the difficulty of locating a suitable zone for testing.

Experience from stress measurements by SSPB's over-coring method, in volcanic rocks in Hong Kong (Söder, 1990; Ingevald & Strindell, 1990), showed that up to half of the measurement attempts failed because of close fissuring. The test results indicated that the interpreted horizontal stresses were higher than the vertical stresses. No firm conclusions on the stress ratio (Section 4.4 refers) were made.

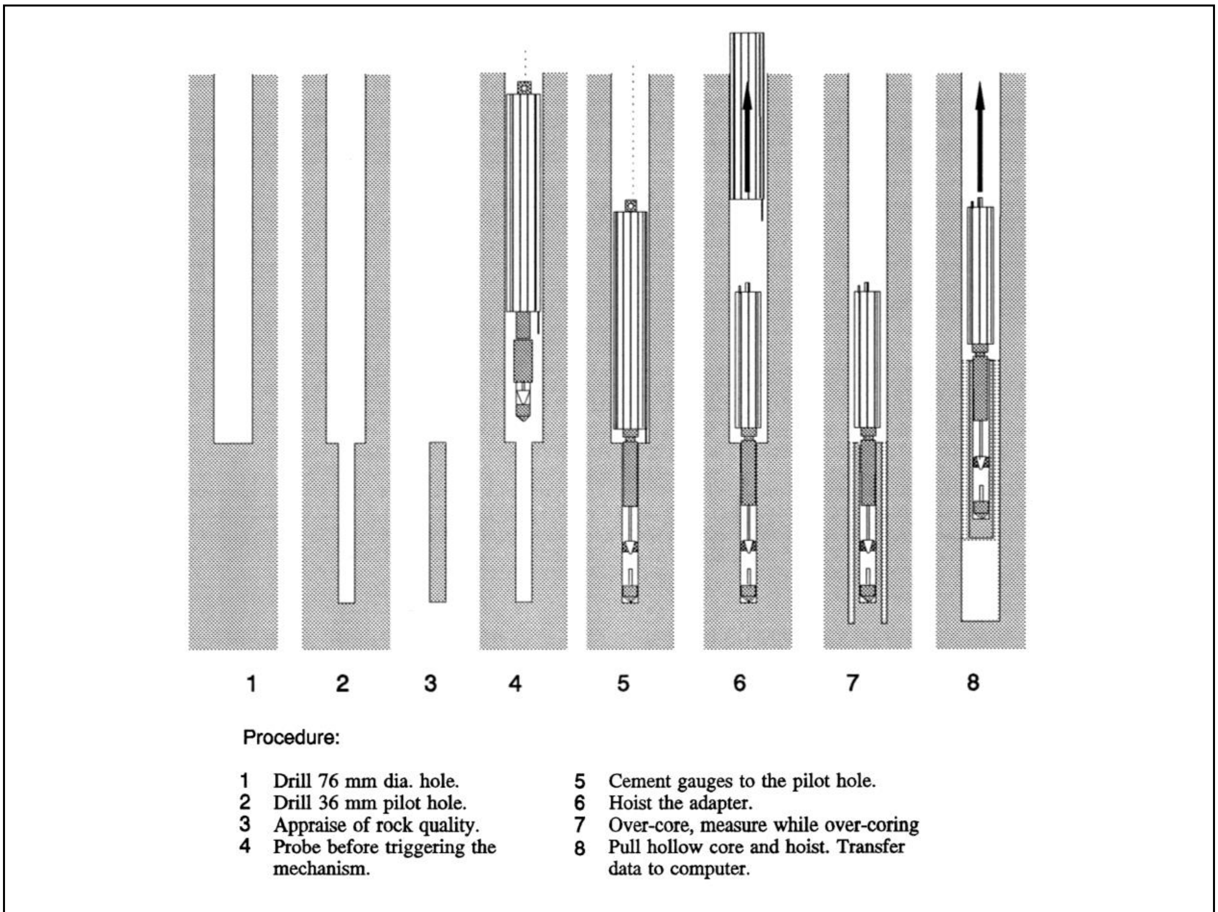



**Figure 5.6 In-situ Stress Measurement by Over-coring (Hiltscher et al, 1979)**

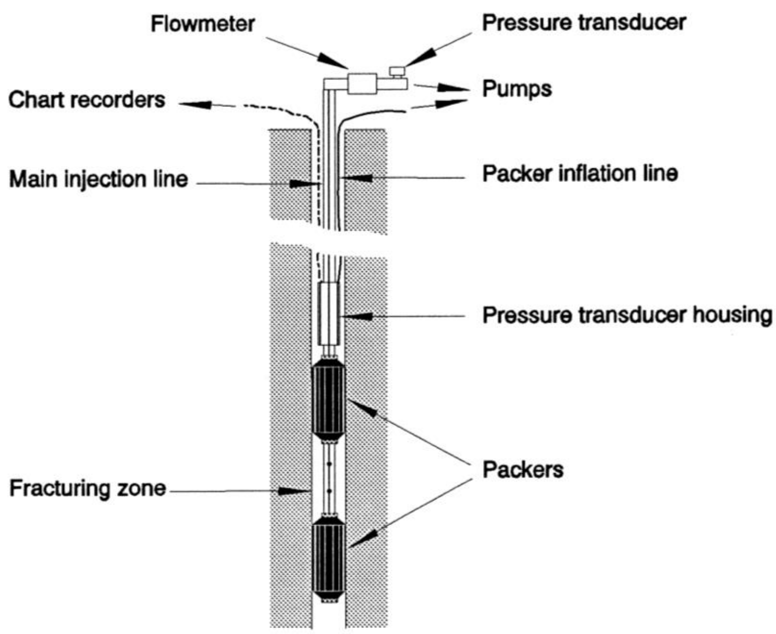
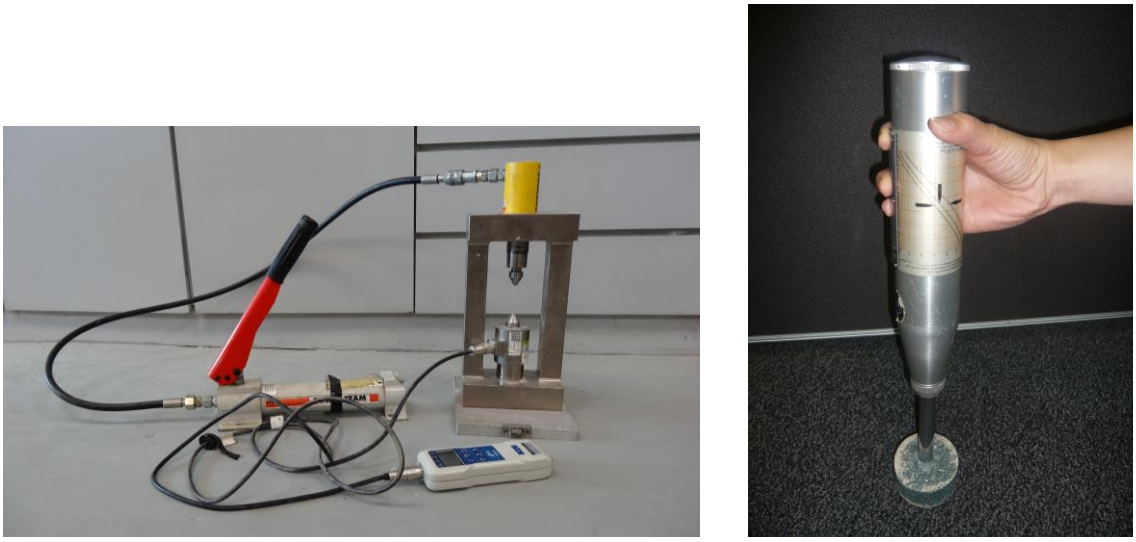



**Figure 5.7 In-situ Stress Measurement by Hydraulic Fracture**

### 5.8.6 Mechanical Strength

(1) *Compressive Strength of Intact Rock.* The strength characteristics of fresh and weathered rock can be measured using uniaxial or triaxial compression tests (Ulusay & Hudson, 2007). These tests should be performed in limited numbers on fresh rock to confirm the high strength expected at most cavern sites. Tests on weathered rock may have more significance because of the deep weathering that exists in some parts of Hong Kong and the potential problems of excavation in such materials. The strength may be verified by carrying out simple point load or Schmidt hammer testing (Figure 5.8). It is advisable to clamp rock samples for Schmidt hammer testing for determination of joint compressive strength. Table 5.1 presents typical unconfined compressive strengths for rocks in Hong Kong.

(2) *Discontinuity Shear Strength.* The rock classification methods referred to in Section 5.5 do not require experimentally determined discontinuity shear strengths as an input. For particular stability problems, such as sliding wedges in cavern walls, knowledge of the discontinuity shear strengths may be important. Experience has shown that visual assessments of discontinuities are generally adequate to assess discontinuity properties. Some tests may however be carried out to support and verify visual assessments and strength estimates. These tests are normally laboratory tests; only on very rare occasions would in-situ tests be justified. Shear box tests following Ulusay & Hudson (2007), tilt tests following Geoguide 2 (GEO, 2017b), Barton & Choubey (1977), Barton et al (1985) and Barton & Bandis (1990) are all considered appropriate.



(a) (b)

**Figure 5.8 Portable Point Load Test Equipment and Schmidt Hammer Rebound Equipment**

### 5.8.7 Deformation Parameters

Deformation parameters for intact rock (e.g. Young's modulus and Poisson's ratio) have limited importance for caverns constructed in rock that has high strength compared to the stresses applied. This will be the normal situation for caverns constructed in Hong Kong.

When conditions warrant, deformation parameters can be measured by laboratory uniaxial or triaxial tests or by using a high pressure pressure-meter or Goodman Jack (Ulusay & Hudson, 2007). Shear and normal stiffness of discontinuities can be obtained by shear box tests (Ulusay & Hudson, 2007; Bandis et al, 1983).

Data on the properties of discontinuities and intact rock should be obtained by appropriate tests for design purposes. Generally, laboratory test results will be used as a reference point to be compared with estimates derived from empirical methods such as correlations with the Q-system and the Generalised Hoek-Brown Criterion. This is because of scale effects and the difficultly of relating laboratory and field scale measurements to rock mass behaviours at an excavation scale.

Analyses for an underground excavation are usually not very sensitive to variations in the Poisson's ratio, and neither does this parameter vary significantly within the rock mass.

### 5.8.8 Chemical Properties

The chemical properties of a rock that are of interest are those related to possible re-use of the rock excavated and groundwater properties for assessing the durability of cavern linings. Any possible aggressive ground or inclusions within the rock that may lead to reactivity with the rock support will need to be investigated during the initial stages of a rock cavern study. In particular the effects of any leachate in the groundwater may be detrimental to the structural elements of the cavern, particularly concrete and structural steel. Minerals may also react with fluids stored in the cavern. One of the examples is the detrimental reaction between sulphur minerals and oil products. Appropriate tests should be carried out for such cases and any other special circumstances.

In-situ weathered rocks and their associated soils in Hong Kong are generally not chemically aggressive (GEO, 2017b), and the same applies to the groundwater derived from these materials. However, in circumstances where cavern developments are situated in rock types with mineral components that may be deleterious to concrete or steel (e.g. pyrite breaking down to form sulphate), then the requirement for geochemical testing will need to be considered. Particular attention should be given to caverns constructed under or near the sea where salt water is likely to be present that may cause severe corrosion of construction steel including concrete reinforcement.

Designers should refer to the relevant structural codes for the classification of exposure conditions for the structural elements in caverns, such as the Code of Practice for Structural Use of Concrete (BD, 2013) and the Code of Practice for Structural Use of Steel (BD, 2011b). The durability requirements for prestressed ground anchors are given in Geospec 1 (GCO, 1989).

For the determination of exposure conditions, tests on the sulphate content and the pH of groundwater are often required. The recommended standard test methods are given in Geospec 3 (GEO, 2017c).

### 5.8.9 Mineralogy and Swelling Properties

It is prudent to carry out mineralogical analysis of fault gouge materials to check for the presence of swelling clay minerals. Should the presence of such swelling minerals be identified, swelling tests in an oedometer should be carried out. Further information on the mineralogy of granitic and volcanic rocks can be found in Irfan (1995 &1998).

### 5.8.10 Rock Temperature

Rock temperature may be recorded in drillholes using a variety of recording thermometers. It is important that the water in the drillhole is in temperature equilibrium with the surrounding rock when measurements are taken. Generally, the near surface rock temperature will be close to the yearly average temperature. Temperature logs can indicate the entry of groundwater into a drillhole and give an indication of the discontinuity sets that are potentially transmissive.

The ambient rock temperature is site-specific but is typically around 23 to 24°C at depths of between 50 and 100 m (recorded at Po Shan Tunnel, Cape Collinson and Mount Davis), which is approximately similar to the yearly average temperature of Hong Kong.

However, a warm water spring of 36°C at a depth of about 650 m below Tai Mo Shan during the excavation of the Butterfly Valley Aqueduct was reported (Robertshaw & Tam, 1999).

### 5.8.11 Tests for Drillability

The drillability of a rock is defined as the rate of drill bit penetration into the rock. The CERCHAR abrasiveness index test (CAI) can be used to assess the drillability of a rock (ASTM, 2010). This test is commonly used in Hong Kong to assess the abrasiveness and drillability of a rock and hence for assessment of the performance of TBMs (e.g. TBM penetration/advance rates, cutter wear rates, etc.). Ho et al (2016) presented the ranges of CERCHAR abrasiveness index for common rocks in Hong Kong, as shown in Figure 5.9. For design purposes, site-specific tests are recommended to acquire more representative results, which may better reflect the local geological conditions.

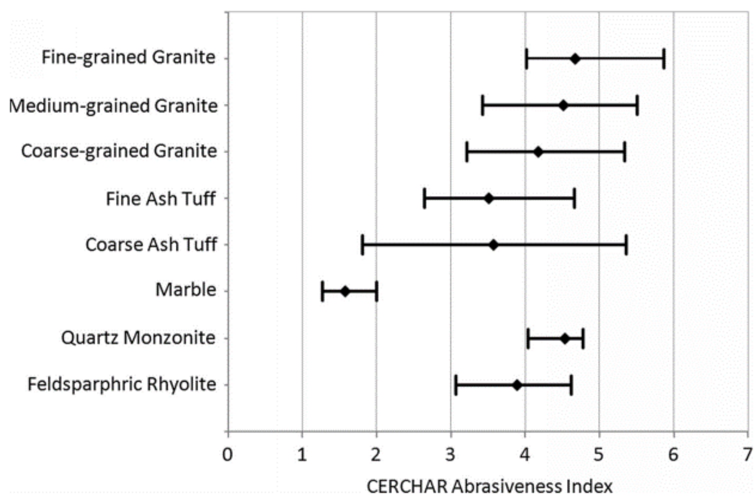



**Figure 5.9 CHERCHAR Abrasiveness Index Values for Common Types of Rocks in Hong Kong (Ho et al, 2016)**

The drilling rate index (DRI), also known as the drillability index, gives an indication of the drillability of a rock. The DRI is calculated using the $S_{20}$ value from the brittleness test and the $S$ value of the Sievers Miniature Drill Test.

The bit wear index (BWI) is a measure of expected drill bit wear. It can be determined from the DRI.

The DRI and BWI are related to the quartz content of a rock as shown in Figure 5.10. The correlation may be used as a basis for estimating the drillability of a rock. This procedure may be sufficient without the need to carry out any test other than quartz content analyses. The tests are described by Tamrock (1986) and Selmer-Olsen & Blindheim (1970).

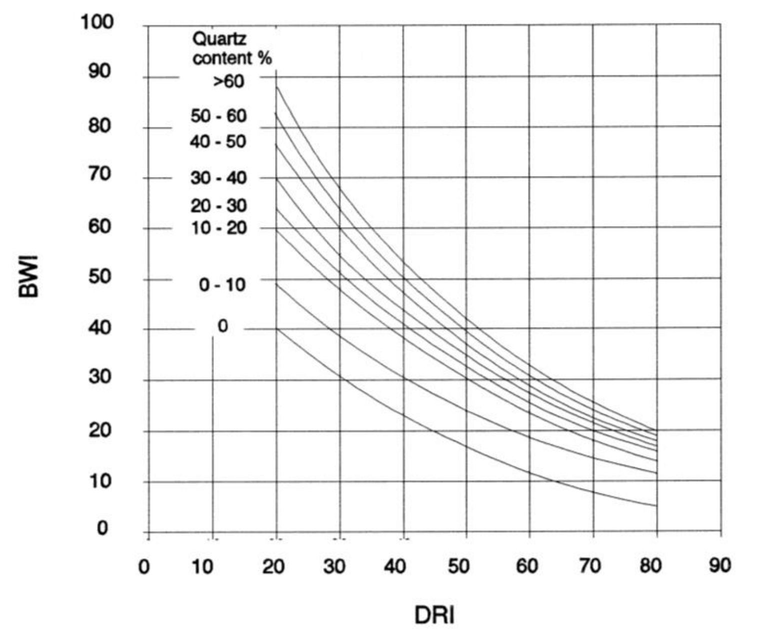



**Figure 5.10 Bit Wear Index (BWI) vs. Drilling Rate Index (DRI)**

### 5.8.12 Aggregate Testing

The effective re-use of excavated materials will improve the sustainability and economics of a cavern development project. To facilitate the optimum re-use of excavated materials, appropriate aggregate testing throughout the project, beginning at the early stages of the ground investigation would be needed. The type of aggregate testing to be carried out depends on the intended use. The types of testing that may be carried out are given in Table 6.1. In Hong Kong, aggregate is mainly used for productions of concrete and asphalt. Further information on aggregate testing is given in GSL (2001) and Construction Standard CS3:2013 (SCCT, 2013).

# 6 Cavern Design

## 6.1 Introduction

This chapter covers the recommended standard of good practice for the design of caverns. Sections 6.2 and 6.3 outline the design approach and key design considerations. Guidance on the optimisation of the cavern location, orientation, layout and shape of a cavern is given in Sections 6.4 and 6.5. Heuristic design considerations that are useful for technical feasibility and planning studies are provided. The development of ground model and design model, and their roles in cavern design are covered in Sections 6.6 and 6.7. Guidance on the design of cavern structures and associated support elements is given in Sections 6.8 and 6.9. The scope and considerations of a hydrogeological impact assessment for cavern design are detailed in Section 6.10.

The contents of this chapter are for the design of caverns for general usage. Aspects relevant to the design of caverns for special purposes (e.g. hydroelectric schemes, fuel storage, cold storage and nuclear waste storage) are not covered.

## 6.2 Design Approach

### 6.2.1 Basis of Cavern Design

The fundamental basis of cavern design includes the following:

(a) rock is a structural material, and

(b) rock mass is not a perfectly homogenous and impermeable medium.

Technical considerations, e.g. stability and groundwater inflow limit, in cavern design are originated from the above basis. However, a cavern design may be optimised based on consideration of factors such as usage, construction methods, construction procedures and rock mass conditions.

### 6.2.2 Design Sequence

A cavern design comprises a series of elements which are optimised to produce a cost-effective cavern solution. The design commonly progresses in an iterative manner as illustrated in Figure 6.1.

As part of the design process, public engagement and studies of other engineering aspects such as environmental impacts, internal uses/functions, traffic impacts, ventilation, fire engineering and evacuation are also essential to the success of a cavern development project. Guidance on these topics is not covered in this Geoguide.

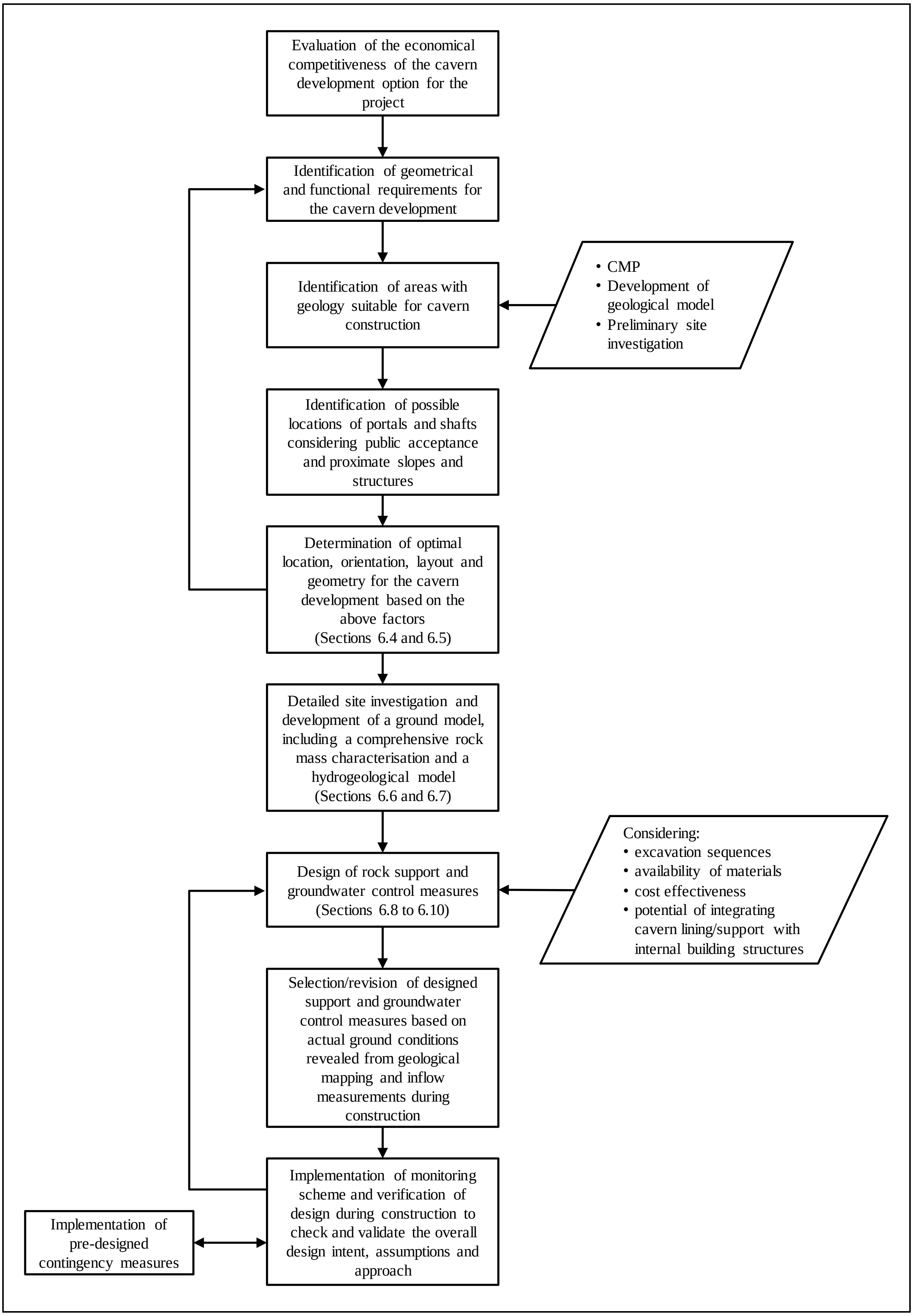



**Figure 6.1 Design Sequence for Caverns**

### 6.2.3 Rock as a Structural Material

In Hong Kong, igneous rocks of decomposition Grades I and II (i.e. fresh and slightly decomposed) typically have a uniaxial compressive strength (UCS) of 100 MPa to 200 MPa or more (see Table 5.1). Thus, strong to extremely strong rock has a compressive strength typically greater than concrete, and can be considered as a structural material in the design of underground excavations in rock. Grade III rock (i.e. moderately decomposed rock) may also be considered suitable as a structural material in some circumstances. However, a rock mass is seldom free of discontinuities or weakness zones. The essence of a cavern design lies in the recognition of weaknesses in the rock mass and the provision of adequate structural supports.

## 6.3 Design Considerations

### 6.3.1 Stability

A cavern should be designed to fulfil the fundamental requirements of stability during construction and throughout its design life. Instability in caverns, including cave-in, rockfall and failure of structural supports, can result in damage to underground facilities, entrapment, injury and death of personnel. Collapse or substantial collapse of caverns may also affect surface facilities above.

### 6.3.2 Serviceability

Serviceability pertains to service performance requirements, for example, cavern deformation, amount of groundwater seepage and movements that can be tolerated by sensitive receivers affected by a cavern construction. Creation of voids within a rock mass results in stress changes and hence deformation. The deformation limit is usually determined based on the requirements of sensitive receivers within the deformation influence zone. Sensitive receivers can either be underground facilities or facilities on the ground surface. The allowable limit of groundwater inflow is primarily governed by the water tightness requirements for the particular usage of a cavern. Assessment of the impact of groundwater drawdown caused by an inflow on environmental and ecological conditions should be made in establishing the inflow limit. Since groundwater drawdown can induce ground settlement, consideration should also be given to the movement tolerance of sensitive receivers that may be effected.

### 6.3.3 Service Life

Similar to other engineered structures, a cavern is designed and built to last for a service life during which it can be maintained in a practical and economically viable manner. Support elements should be designed to be sufficiently durable and robust to guard against local deterioration of the rock mass over time, in particular, at weakness zones. The corrosion and/or deterioration of the cavern support elements within the service life span of the cavern structure in the associated geochemical environment should be duly considered.

### 6.3.4 Other Considerations

The construction cost and time required to build a cavern depends on site accessibility and constraints, construction method and sequence, rock mass quality, groundwater inflow control and ground support requirements, amount of over-break, potential in reusing the excavated materials, tolerance of sensitive receivers affected, need for permanent structural lining, etc. These factors should be considered holistically in order to produce an optimum design solution. Considerations for portals, adits, tunnels and shafts required for access are important as part of the design process for the cavern development and are specifically addressed in Chapter 8. It is essential that the concept of safety forms an integral part of the design so that the proposed cavern is safe to construct, operate and maintain, and it will not impose long-term health hazards to the occupants. In addition, impact on the environment should be duly assessed as part of the design.

## 6.4 Cavern Location and Orientation

### 6.4.1 Factors in Optimising Cavern Location and Orientation

The choice of location for a cavern development is one of the most important decisions in the design process, as it determines the quality of the rock mass in which the cavern will be excavated. Factors other than geology can constrain the choice of cavern location. The CMP (Section 3.2.1) provides a good reference to identify potentially suitable locations for a cavern development project. It should be consulted at an early stage of the project.

The constraints on the selection of a cavern location are frequently related to the locations of portals, adits, external traffic, and the size and operation of the cavern facility. Within such constraints, the cavern installation should be optimised with respect to the topography and geology. A balance of the following factors should be made in achieving the optimisation:

(a) adequacy of rock cover,

(b) avoidance of rock mass weakness zones or the crossing of them in the shortest possible distance,

(c) avoidance of adverse orientations relative to major discontinuity sets,

(d) sufficient depth below groundwater table (for some uses),

(e) avoidance of rock mass under abnormally low stresses, giving reduced confinement pressures, and

(f) avoidance of rock mass under very high stresses (unlikely to be a critical factor for cavern developments in Hong Kong at relatively shallow depths).

The use of 3D models (e.g. building information modelling) can help to visualise the geographical relationship between a proposed cavern and the associated constraints / sensitive receivers affected and thus can facilitate the process of optimisation of cavern location and orientation.

### 6.4.2 Minimum Rock Cover

For shallow caverns, a minimum rock cover should be provided. The rock cover should be sufficient to give adequate normal stresses on the discontinuities such that the roof and walls will be, as far as possible, self-supporting. The requirement of a minimum rock cover can be a major constraint on the selection of cavern location and is determined with consideration of many factors, including:

(a) quality of geological information,

(b) rock properties,

(c) thickness of superficial deposits,

(d) depth of weathering,

(e) in-situ stresses,

(f) cavern span, and

(g) cost implications.

In feasibility studies, based on limited information, it is prudent to provide a rock cover of not less than half of the cavern span. However, a greater rock cover may be required in a variety of situations, for instance where the rock mass is of poor quality, at faults or below bodies of water.

In general, a reduced rock cover would increase the amount and cost of ground investigation, rock support, and potentially groundwater control measures. Therefore, a reduced rock cover should normally be adopted in limited areas, such as the section of cavern closest to the portal.

### 6.4.3 Weakness Zones

Weakness zones are defined as zones that are weaker than the surrounding rock. They have many origins and can be highly decomposed or weak rocks, faults, heavily fissured zones, hydrothermally altered rocks, deeply weathered zones and combinations of those. Many major difficulties in underground construction are related to such zones.

The width of a weakness zone may range from a few centimetres to several hundred metres. The material in the zone may be described in accordance with the principles given in Geoguide 3 (GEO, 2017a). Consideration should be given to dividing the weakness zone into areas of similar properties (where applicable) as recommended by USBR (1998) for improving the overall characterisation and understanding of the weakness zone. Figure 6.2 shows an example from USBR (1998).

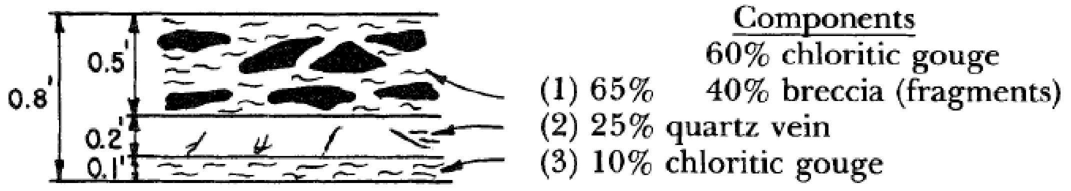
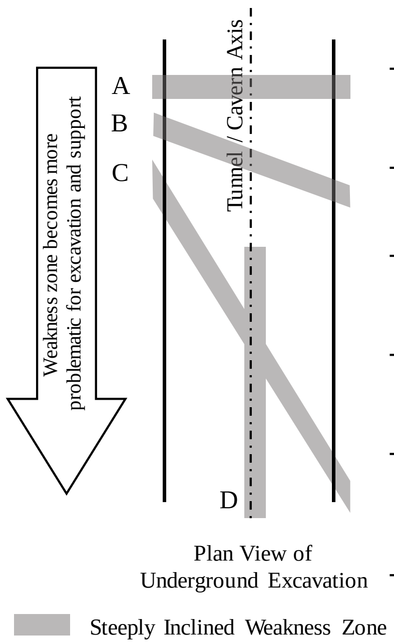



**Figure 6.2 Sample Description of a Structured Shear Zone (after USBR, 1998)**

Figure 6.3 illustrates the likely amount of rock supports required and the potential construction difficulties in different situations where weakness zones intercept a cavern at different angles. It serves to provide a general discussion for reference. The actual situation should be assessed on a case specific basis. Excavation in weakness zones should be minimised or, if possible, avoided by adoption of a different cavern location or geometry. Consideration should be given to the orientation of the weakness zones in a similar manner to discontinuities as discussed in Section 6.4.4.




| Case | Weakness Zone Orientation Relative to Tunnel/Cavern Axis | Sidewall Stability | Roof Stability | Overbreak |
|---|---|---|---|---|
| A | Perpendicular | Smaller blocks and less support needed | Localised support over short length of excavation | Likely but localised |
| B | Oblique (≥ 45°) | Medium blocks and moderate support needed | Moderate support over medium length of excavation | Likely and over wider section of excavation |
| C | Oblique (< 45°) | Larger blocks and heavier support needed | Significant support over long length of excavation | Extended with greater influence on blast hole tolerance & blockyness |
| D | Sub-parallel (Unfavourable) | If intersect sidewall then could destabilise major sections of sidewall, very heavy support needed | Major systematic support over very long length of excavation | Major influence on roof and sidewall overbreak & significant influence on blast hole tolerance |

**Figure 6.3 Rock Support and Overbreak Related to Orientation of Weakness Zones**

### 6.4.4 Discontinuities

The orientation of discontinuities (which include shear fractures, joints, bedding, foliation and cleavage) with respect to the axis of an excavation has an influence on its stability and the amount of overbreak.

When other constraints can be overcome, optimisation of the direction of the excavation axis with respect to discontinuity orientation should be achieved. To this end, it is necessary to carry out a detailed survey of the structural geological features in the rock mass. In some cases, persistence of the discontinuities can be determined in outcrops or excavations. For openings situated at shallow or intermediate depths, the longitudinal axis of caverns is ideally oriented along the bisection line of the largest intersection angle of the strikes of the two dominant sets of discontinuities, as illustrated in Figures 6.4 and 6.5. It is highly undesirable for the long axis of an excavation to be parallel to the strike of a major discontinuity set. In sedimentary rocks, the long axis should preferably be in the same direction as the dip direction of the beds. Similarly, the stability of the temporary face of an excavation should be checked taking into account the dip and dip direction of the discontinuities, to ensure adequate safety to the construction personnel.

The amount of overbreak has also been shown to increase if the angle between the excavation axis and the strike of a major discontinuity set is small, say less than 30° (Thidemann, 1976). The character of the discontinuities can have a major influence on the orientation of an excavation. For long and high walls it is important to have an angle of at least 25° to the strike of steeply dipping, smooth discontinuities or clay-filled joints and zones (Thidemann, 1976).

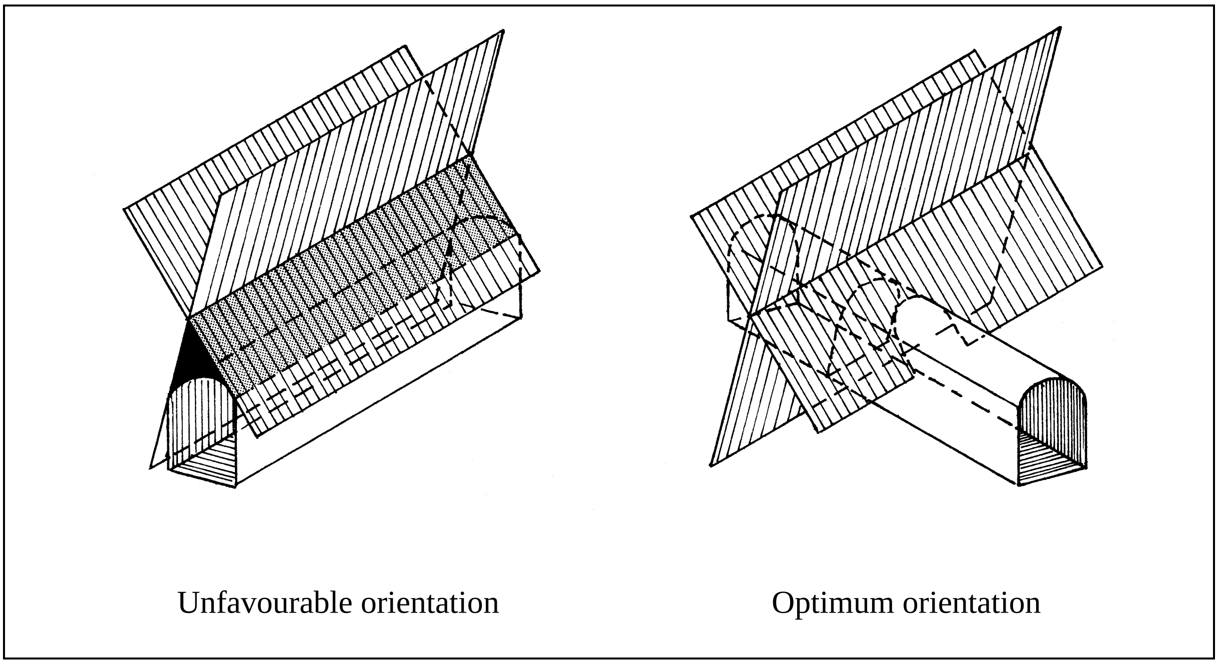



**Figure 6.4 Orientation of Cavern with Respect to Discontinuity Sets (after Hoek & Brown, 1980)**

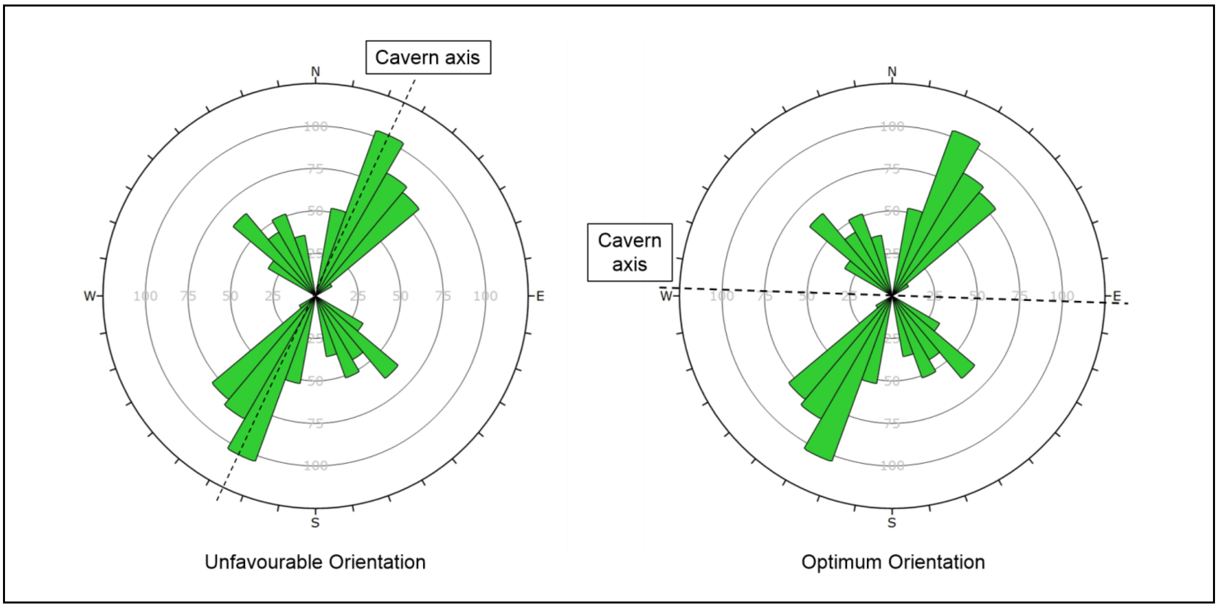



**Figure 6.5 Orientation of Cavern with Respect to Discontinuity Sets as Presented on Rosette Plots**

### 6.4.5 Groundwater

The location of the groundwater table and prediction of changes to the hydrogeological regime caused by underground openings can be critical in determining the elevation of a cavern development. Some schemes require continuous seepage towards the cavern which ensures that the groundwater will not be polluted by materials and activities in the cavern facility. For some uses, such as storage of petroleum products and fuel gas, the groundwater pressure is used to confine the products or gas and is thus a prerequisite for successful implementation. For such cases, the groundwater level and pressure around the cavern have to be maintained with surrounding water curtains. For other applications, the location of the groundwater table is less important, and the cavern development should ideally be placed above the groundwater table to obtain conditions that are as dry as possible. The need to minimise the impacts on the environment or existing facilities due to changes in the groundwater regime may be a constraint on the cavern location in some projects.

### 6.4.6 In-situ Stress Conditions

The stresses which exist in a rock mass can have one or more origins. Gravitationally induced stresses and tectonic stresses are often the major components, but residual stresses (the locked-in stresses resulting from the stress history of the rock mass) can also be significant.

Stresses within the rock mass influence the stability of an excavation. High tectonic and residual stresses at shallow depths can result in issues including rock bursts, slabbing and squeezing ground. Local experience in cavern construction suggests that this may not be relevant to the practice in Hong Kong. Kwong & Wong (2013) assessed the in-situ rock stress ratio (i.e. horizontal principal stress divided by vertical principal stress) based on the measurement data obtained from previous underground railway projects in Hong Kong. The ratio was found to range from 1.4 to 2.5 at shallow depths.

At locations where a high in-situ stress ratio is encountered, the long axis of a high gallery cavern should be arranged to be parallel to the major principal horizontal stress direction where possible, as this will pre-stress the rock blocks within the walls (Figure 6.6). On the other hand, for a large span cavern, it would be more suitable to arrange the cavern axis to be perpendicular to the major principal horizontal stress direction to aid overall stability of the roof (Figure 6.7). Free et al (2000) reported the orientation of major horizontal stresses in rock mass at some locations in Hong Kong.

A cavern development project should be planned in a way such that the flow of stresses around the chosen geometry of the excavation is smooth. Stress concentration, tensile stresses and induced slip zones should be avoided, especially in highly anisotropic stress fields.

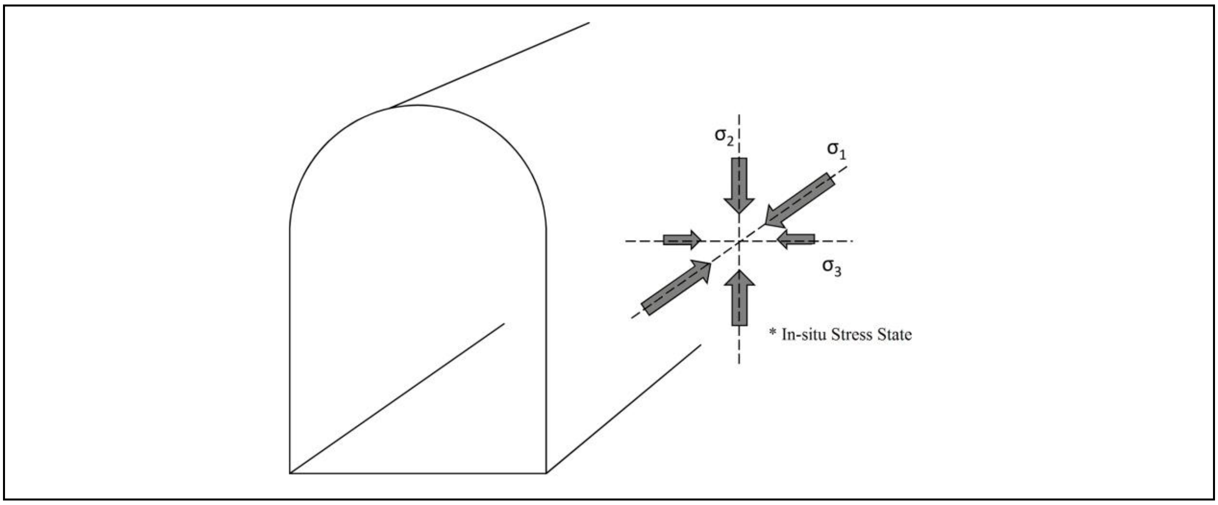



**Figure 6.6 High Gallery Cavern Needs Greater Wall Stability**

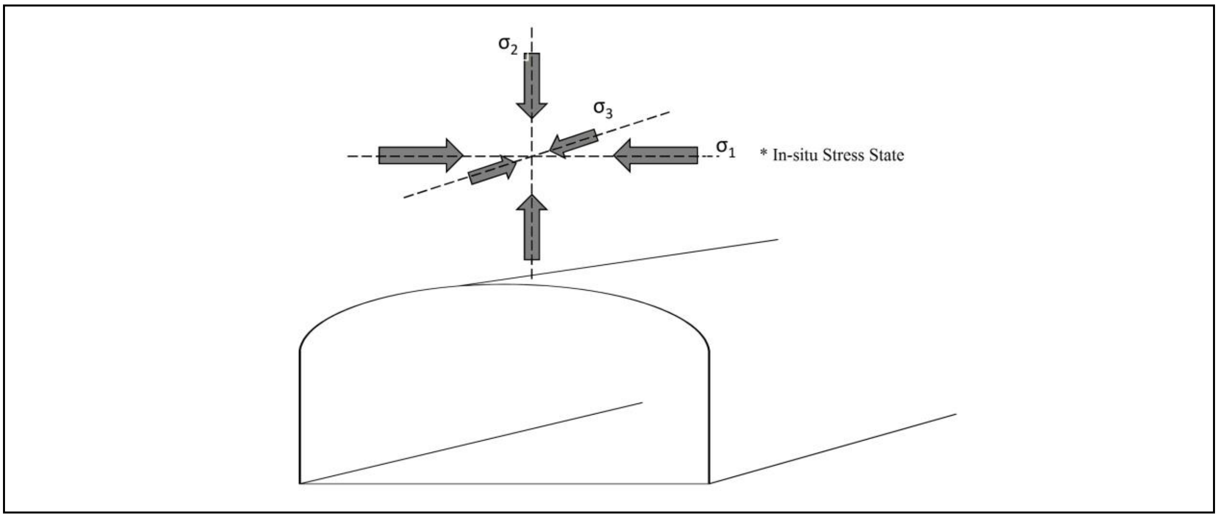



**Figure 6.7 Large Span Cavern Needs Greater Roof Stability**

## 6.5 Cavern Layout and Shape

### 6.5.1 General

The geometry and layout of a system of caverns are normally based on optimising the requirements given by the cavern usage, and on empirical guidelines for dimensioning low-cost cavern space that are based on the costs of performing various excavation and support operations. The geometry of the openings, i.e. the total height and arch shape, influences the cost of excavation and support. Figure 6.8 shows the layout of the cavern development for the Island West Transfer Station.

The main parameters to be considered are the cavern size and shape and the spacing between caverns. This section provides general guidance on these aspects.

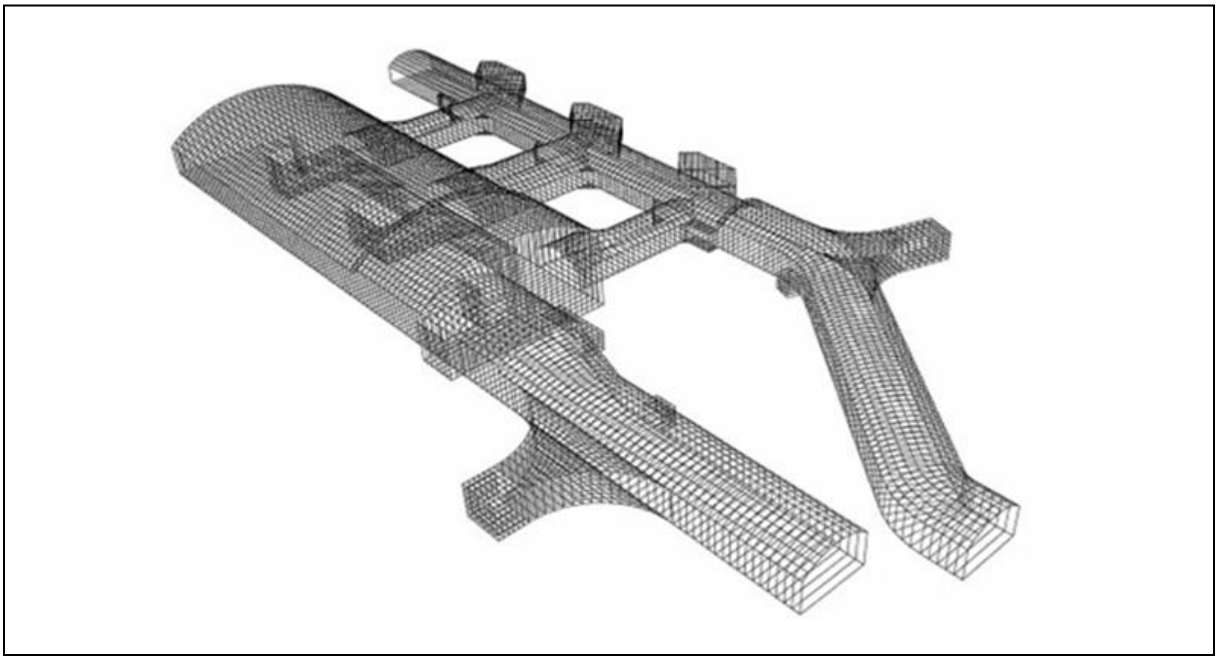


**Figure 6.8 Layout of Island West Transfer Station (courtesy of ARUP)**

### 6.5.2 Cavern Shape

(1) *General.* A rock mass is a discontinuous material. The basic design concept for a cavern is that the shape of the opening should conform to the rock structures and stress conditions. The compressive stresses in the rock mass bounding the excavation should be evenly distributed such that the span of the cavern can be self-supporting as far as possible. The best possible stress distributions are usually obtained by giving the cavern space a simple form with an arched roof to reduce the zone of tensile stresses in the crown. Intruding corners should be avoided as far as practicable, since the rock will be locally de-stressed, resulting in greater overbreak during blasting (Figure 6.9).

The cross-section of a cavern should be optimised, within given constraints, to produce the lowest combined excavation and support costs. For example, support costs increase with cavern span, excavation rates reduce with cavern height, and wall support costs increase with cavern height. The support costs typically rise more rapidly with span compared with surface structures. Therefore, costs can be a significant factor with respect to a cavern span. At the feasibility study or preliminary design stage, an optimisation study should be carried out to quantify these costs, at least relative to each other, for the prevailing rock mass conditions. Figure 6.10 shows typical curves that illustrate the relationship between unit cost and cavern dimensions.

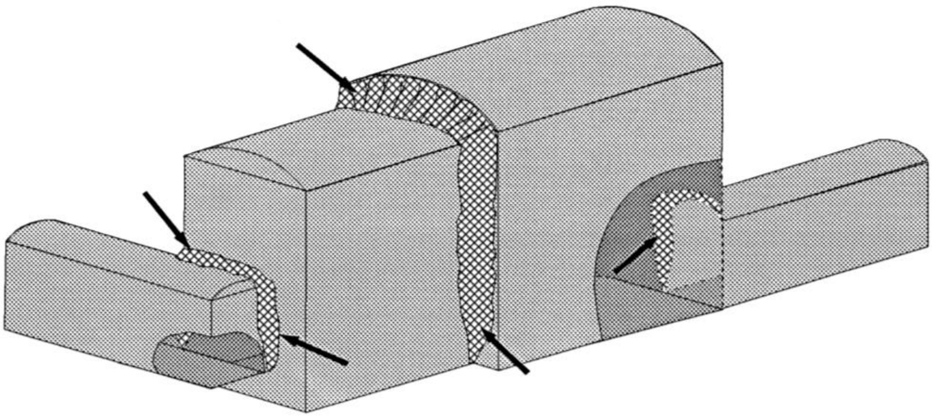
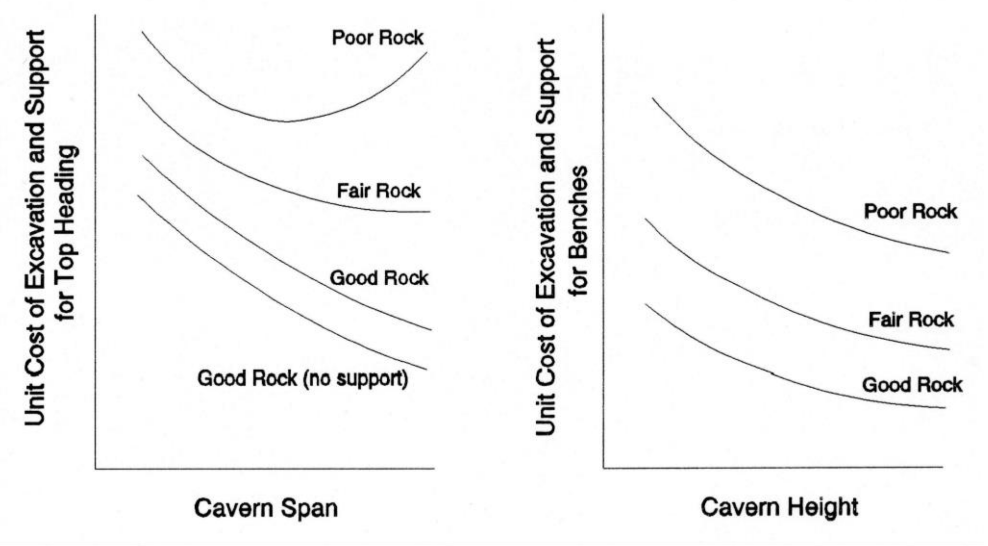


**Figure 6.9 Location of Unstable Intruding Edges and Corners (after Selmer-Olsen & Broch, 1982)**




Note: The unit cost of excavation and support refers the cost per unit volume of rock excavated.

**Figure 6.10 Typical Cost Curves for Different against Cavern Span and Height for Different Rock Conditions**

(2) *Roof Arch*. The starting point for the design of the shape of a cavern roof is the assumption of a standard roof arch height of 1/5 of the cavern span (Figure 6.11). The reduction in roof support costs due to improved stability by increasing the roof arch height is normally inadequate to offset the additional excavation cost. In contrast, reducing the roof arch height increases instability problems and rockfalls potential during blasting and may only be justified if the dominant discontinuities have a shallow dip. In Hong Kong, sheeting joints have caused parts of the cavern roofs that were intended to be arched to become flatter, with an associated increase in instability and overbreak (e.g. Taikoo and Sai Ying Pun MTR station caverns and the Western Salt Water Reservoirs). Hencher et al (2011) noted that sheeting joints are generally unlikely to be encountered below 30 m depth, although this will need to be considered on a site-by-site basis.

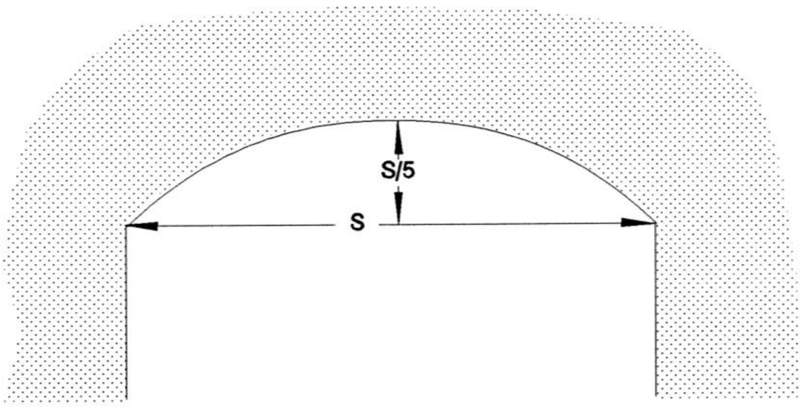



**Figure 6.11 Standard Roof Arch Shape**

(3) *Wall Height.* Cavern walls are normally vertical. This suits the method of excavation and maximises usable space. The wall stability is a function of the wall height, the in-situ stresses and the orientation and engineering properties of the dominant discontinuity sets. A flat vertical wall surface precludes any substantial arching action and high walls tend to be unstable.

Major discontinuities and seams can dominate wall stability and can affect the chosen wall height. The cost and scale of stabilising measures can increase substantially with wall height and this has to be taken into account in optimising cavern shapes.

The steeply dipping discontinuities with strikes parallel to a wall reduce stability as the horizontal stresses on the discontinuities are small. The converse of this is true for the roof arch. This is typical of underground works where conditions that favour one part of the works are detrimental to the other parts.

(4) *Effects of Anisotropic and High Stresses.* Anisotropic and high stresses may need to be taken into account in cavern design, but only in exceptional cases would the shape of the cross-sections be altered for such reasons. There are few records of caverns in rock where anisotropic stresses have been a major influence on the cavern cross-sections.

### 6.5.3 Spacing between Caverns

(1) *Pillars*. Consideration should be given to stress concentrations and failures along discontinuities in determining the acceptable width of pillars. At the preliminary design stage, pillar widths can be estimated by assuming kinematically possible sliding on unfavourable discontinuities (Figure 6.12), and from an assessment of pillar capacity on the basis of available geological data (see also Section 6.8.2 for pillar stress calculation and Section 6.8.4 for stability criteria for pillars). In-situ stresses can also affect pillar widths, especially for deep caverns. Typical pillar widths between caverns are between half and full cavern span or height, whichever is the greater. As a project progresses on the basis of improved geological data, the pillar widths should be optimised, and appropriate lateral confinements to the pillars should be provided if necessary.

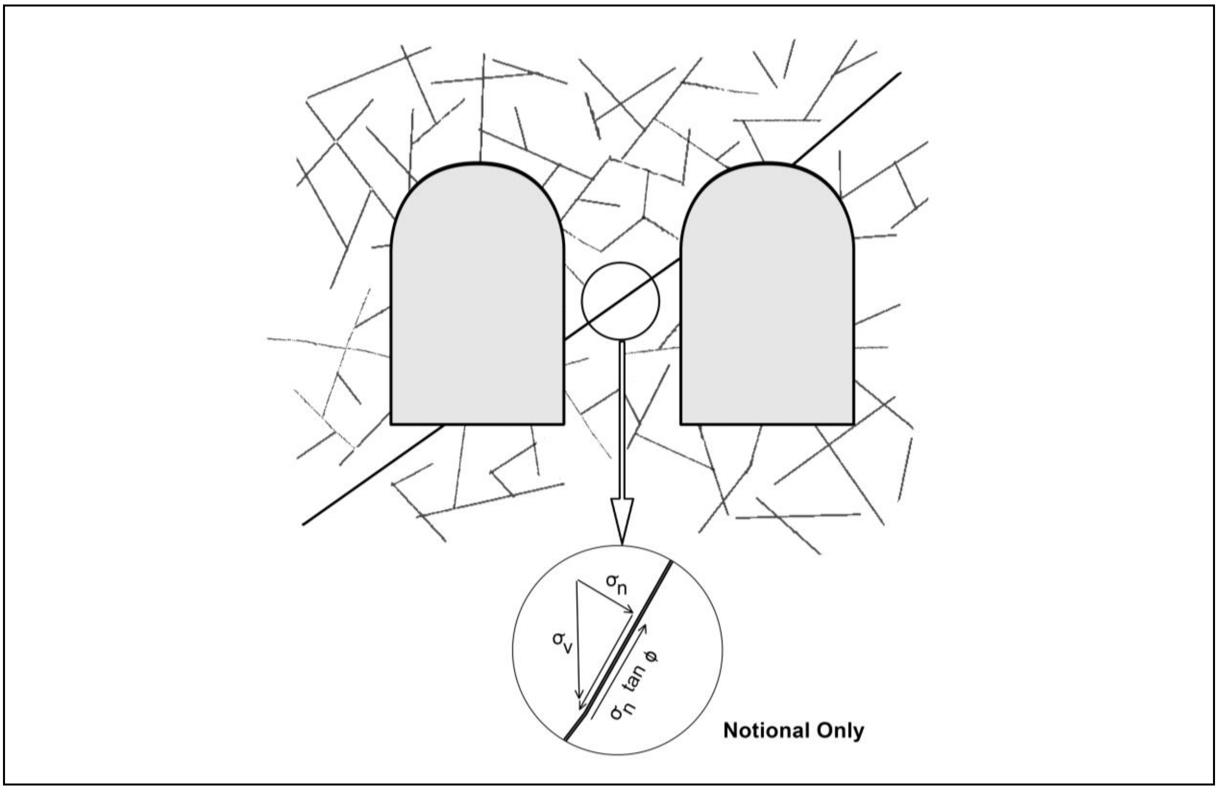



**Figure 6.12 Stability of Pillars with Potential Sliding Planes**

(2) *Vertical Separation*. There are no simple guidelines or dimensioning practice for evaluating vertical separation of parallel caverns located above each other. However, the influence of an excavated opening on the stresses in the surrounding rock mass decreases with increasing distance from the opening. If isotropic virgin stresses and a circular opening are assumed, induced stresses are insignificant at a distance from an opening of 0.5 to 1 times the diameter of the opening. For caverns located at a modest depth, which will be the common case in Hong Kong, the induced stresses will be very low compared to the rock strength. Hence, this subject has to be considered from the viewpoint of discontinuity geometry relative to caverns and practical construction.

In the mining industry, there are examples of caverns excavated one above the other with a minimum vertical separation of 10 m. For example, the Franzefoss Bruk caverns in Norway have span, height and pillar width of 13, 17 and 8.5 m respectively and a vertical separation of 8 m.

As a general guide, vertical separation should not be less than the largest span or height of the adjacent caverns. Separations of less than 20 m should be designed with support of detailed analysis. For schemes with stacked caverns, detailed analysis will be required if lesser separations are desired. Reduction in vertical separation must be justified on the basis of clear benefits in relation to cost, including the cost of data collection and analysis required to establish the safety and construction cost of the reduced separation. Design of reduced separation should take into account:

(a) increased blasting and support costs,

(b) overbreak and loosening of rock beyond the rock face in both the lower and upper caverns, and

(c) risk of rockfalls that may impair the stability of the floor of the upper cavern.

The stability of the separating rock may be improved by pre-grouting, bolting and anchors from either the upper or lower cavern.

Excavation of an upper cavern before a lower caverns is recommended. This avoids the risk of damage to the roof support installed in the lower cavern by vibrations from the heavy explosive charges used in the bottom of the upper cavern.

#### 6.5.4 Cavern Junctions and Adit Connections

Another aspect of cavern design that requires careful consideration is the junctions between caverns and their associated adits, tunnels and shafts. The junctions often result in zones of stress concentration and intruding corners which are detrimental to cavern stability. Care should be taken to deal with stressed zones around support elements which may be adversely affected by the excavation for the cavern junctions and adit connections.

## 6.6 Ground Model

### 6.6.1 Definition

Economic cavern design is not possible without the development of a comprehensive ground model. Knill (2003) considered that a ground model which builds on the geological model and embeds the range of engineering parameters and ground conditions should be considered in the design. The ground model should focus on the main engineering issues and will typically comprise plans, sections, 3D representations, descriptions and parameters, and will complement a geotechnical risk register. Guidance on ground models is given in GEO (2007).

This Geoguide focuses on the rock mass characterisation and hydrogeological model components of a ground model for cavern design purposes. These two components are the most applicable elements for assessing the engineering issues relevant to the design of cavern structures. Other components of a ground model that are of importance to cavern design are more general. For instance, the extent and engineering properties of decomposed rock around a portal for assessing the landslide hazards to the access points.

### 6.6.2 Rock Mass Characterisation

(1) *General*. Rock mass characterisation is one of the most important aspects of cavern engineering and follows logically from the recognition that rock can generally be considered to be a structural material for the purposes of many cavern developments.

Rock mass ***characterisation*** is not the same as rock mass ***classification***. Rock mass characterisation requires that the actual conditions of the rock mass are described as closely as possible and parameters are applied where possible and appropriate, allowing the behaviour of the rock mass to be assessed. Rock mass classification is the division of the rock mass into different classes on the basis of a defined scheme such as the Q, RMR or RMi system. However, good rock mass characterisation greatly assists appropriate rock mass classification. Data is lost when classification occurs and the process is not reversible, i.e. it is possible to move from characterisation to classification but it is not possible to move from classification to characterisation.

The behaviour of a rock mass during excavation and how the stress changes will affect the blocks of rock surrounding the excavation are key elements of the cavern engineering design. Rock mass characterisation facilitates the following:

(a) delineation and characterisation of areas with similar engineering properties and identification of boundaries at which changes in geotechnical conditions may occur (e.g. changes in rock type, structural domains and weathering),

(b) identification, consideration and incorporation of stability-critical or performance-critical features such as weakness zones, dykes and discontinuities,

(c) targeting of critical features for a detailed investigation,

(d) selection of parameters,

(e) sustainability and cost-effective re-use of excavated materials, and

(f) identification and management of uncertainties and risks through the use of a geotechnical risk register.

The appropriate level of details of a rock mass characterisation is dependent on the project stage. For instance, at the feasibility study stage of a project, rock mass characterisation can be quickly and inexpensively carried out by consulting published geological maps, memoirs, papers, API and the CMP. This allows consideration of the location, orientation and geometry of a cavern development to begin. The characterisation also allows key rock engineering uncertainties and risks to be identified early in the project and in a simple form that facilitates communication within the project team.

As the project progresses and more information becomes available, the characterisation should be refined provided that adequate site investigation is carried out. In the early stages of design, the available information may allow rock mass classification and analytical solutions. Later stages of the development will allow refinement of the earlier analyses and facilitate comprehensive numerical modelling. Table 6.1 lists the information that a detailed rock mass characterisation should include. Italicised text refers to essential data for a cavern design.

**Table 6.1 Data for Rock Mass Characterisation (Sheet 1 of 2)**

| | |
|---|---|
| Rock material | • *Elastic constants (e.g. Young's modulus and Poisson's ratio)*<br>• *Compressive strength (e.g. unconfined compressive strength (UCS), triaxial)*<br>• *Tensile strength (e.g. Brazilian test, point load strength)*<br>• *Density*<br>• *Porosity*<br>• *Anisotropy and inhomogeneity*<br>• *Decomposition grades* |
| Discontinuities (e.g. weakness zones, shear fractures, joints, cleavage, etc.) | • *Orientation (dip angle / dip direction)*<br>• *Description of sub-zones if applicable (e.g. of weakness zones)*<br>• *Persistence, size, shape and termination*<br>• *Spacing and frequency (including rock quality designation (RQD))*<br>• *Block size*<br>• *Wavelength and amplitude*<br>• *Roughness (including joint roughness coefficient (JRC))*<br>• *Aperture*<br>• *Infill (including description and properties)*<br>• *Discontinuity wall strength (e.g. joint compressive strength (JCS)) and weathering*<br>• *Hydraulic conductivity**<br>• *Groundwater chemistry*<br>• *Shear strength* |

**Table 6.1 Data for Rock Mass Characterisation (Sheet 2 of 2)**

| | |
|---|---|
| Rock mass properties (measured using field tests or derived from rock mass classification schemes and/or appropriate failure criterion) | • *Deformability*<br>• *Strength*<br>• *Anisotropy and inhomogeneity*<br>• *Weathering*<br>• *Permeability* |
| In-situ Stress | • *Natural state of in-situ stress* |
| Excavatability | • *Brittleness value*<br>• *Surface hardness (e.g. Vickers hardness test)*<br>• Abrasiveness (e.g. CERCHAR)<br>• *Indentation (e.g. punch-penetration tests)*<br>• *Rock cutting tests (linear and rotary)*<br>• *Drillability (Siever's J-value)*<br>• Flakiness<br>• *Petrographic analysis (thin section and X-ray diffraction (XRD) analysis)* |
| Typical aggregate properties (data to determine the suitability of excavated rock for aggregate – depending on project requirements and final use) | • Los Angeles value (LAV)<br>• Flakiness and elongation indices<br>• Aggregate abrasion value (AAV)<br>• Aggregate crushing value (ACV)<br>• Aggregate impact value (AIV)<br>• Alkali-silica Reactivity<br>• Polished stone value (PSV)<br>• Ten-percent fines value (TFV)<br>• Magnesium sulphate soundness value<br>• Drying shrinkage<br>• Chlorine content<br>• Sulphate content<br>• Resistance to fragmentation<br>• Oven-dried particle density and water absorption |

Notes: (1) Essential data for cavern design in italics.
(2) * The in-situ permeability of rock masses should be measured over portions of drillholes containing sufficient number of representative discontinuities.

(2) *Parameters.* An important part of a rock mass characterisation is the selection of parameters that accurately represent all aspects of the ground conditions and allow engineering analyses. These parameters will be a combination of the site investigation findings, engineering judgment and experience. Generally, parameters should be quoted with a range

to account for natural variability, uncertainty and error. Where applicable, the use of histograms or similar graphical and statistical methods during data collection is recommended to assist with the derivation of appropriate parameters and their ranges. In particular, this approach has been recommended by Barton & Grimstad (2014) for the assessment of input parameters for the Q-system but it is applicable to the consideration of other parameters as well. Further discussions on design parameters will be given in Section 6.7.2.

(3) *Strength and Deformation of Jointed Rock Mass*. A rock mass failure criterion is required for incorporating into numerical models or limit equilibrium analyses for predictions of the load-deformation behaviour of a jointed rock mass.

For numerical models and limit equilibrium analyses which adopt a discontinuum approach, it is recommended to use the Mohr-Coulomb criterion or the Barton-Bandis failure criterion (Barton & Bandis, 1990) to model the shear strength of rock discontinuities. The Mohr-Coulomb criterion is a simple and well-understood model. Parameters required by the criterion can be readily estimated or determined from relatively standard tests at appropriate stress levels. The Barton-Bandis failure criterion utilises a series of empirical relations for joint normal and shear behaviours based on the effects of surface roughness on discontinuity deformation behaviour and strength. It aims at scaling laboratory data on rock joint behaviour to a larger scale for numerical modelling purposes. Equation 6.1 presents the shear strength component of the Barton-Bandis failure criterion.

$$\tau = \sigma_n \tan\left[\phi_r + \mathrm{JRC}\log_{10}\left(\frac{\mathrm{JCS}}{\sigma_n}\right)\right] \qquad (6.1)$$

where $\tau$ = shear strength (in kPa)
$\sigma_n$ = normal stress (in kPa)
JRC = joint roughness coefficient
JCS = joint compressive strength
$\phi_r$ = residual friction angle of the failure surface (in °).

Barton & Choubey (1977) suggested that $\phi_r$ can be estimated from Equation 6.2:

$$\phi_r = (\phi_b - 20) + 20\left(\frac{r}{R}\right) \qquad (6.2)$$

where $r$ = Schmidt hammer rebound number on wet and weathered fracture surfaces
$R$ = Schmidt rebound number on dry unweathered sawn surfaces
$\phi_b$ = basic friction angle of the failure surface (in °).

Equations 6.1 and 6.2 are part of the Barton-Bandis criterion for rock joint strength and deformability (Barton & Bandis, 1990). The shear strength of discontinuities infilled by soft materials can be reduced drastically. A comprehensive review of the shear strength of infilled discontinuities was given by Barton (1974). The shear strength of the infill material should be investigated if it is critical to the stability of a rock mass.

Barton (2014) provides guidance on establishing the values of JRC and JCS. The guidance covers the measurement or estimation of these parameters, and the necessary adjustments for scale effects, i.e. the down-scaling of JRC and JCS to cross-joint spacing (block size) in relation to the reference laboratory-scale joint sample length. Hoek (2007) provides further information on the shear strength of discontinuities including a discussion of the Barton-Bandis failure criterion parameters.

The generalised Hoek-Brown failure criterion (Equation 6.3) is recommended for use in continuum analyses. The Hoek-Brown failure criterion assumes isotropic rock and rock mass behaviour, and is applicable to rock masses characterised by discontinuities with similar joint properties. When the structure being analysed is large relative to the block size, the rock mass can be treated as a Hoek-Brown material.

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_{\mathrm{ci}} \left( m_{\mathrm{b}} \frac{\sigma_3^{'}}{\sigma_{\mathrm{ci}}} + s \right)^a \quad \text{..........................................} \quad (6.3)$$

where $\sigma_1^{'}$ & $\sigma_3^{'}$ = maximum and minimum effective principal stresses at failure (in kPa)

$\sigma_{\mathrm{ci}}$ = uniaxial compressive strength of the intact rock pieces (in kPa)

$m_{\mathrm{b}}$ = value of the Hoek-Brown constant of the rock mass

$s$ & $a$ = constants which depend upon the rock mass characteristics.

An important component of the Hoek-Brown failure criterion is the Geological Strength Index (GSI), proposed by Marinos & Hoek (2000). The GSI provides a means to characterising rock masses and selecting input parameters for the Hoek-Brown failure criterion. The GSI ranges from 10 (for an extremely poor rock mass) to 100 (for intact rock). As noted by Palmström & Stille (2010), the use of the GSI for rock mass characterisation is straightforward and based on the visual appearance of rock structures in terms of blockiness, and the surface condition of discontinuities as indicated by discontinuity roughness and alteration. The combination of these two parameters provides a practical means to describe a wide range of rock mass types. It should be noted that there is no input for the strength of rock material in the GSI system. The GSI comprises two charts, a general chart for "jointed rocks" and one for "heterogeneous rock masses such as flysch" (see Tables 3 and 12 of Marinos & Hoek (2000) respectively). The latter chart is considered not applicable to Hong Kong.

It is also possible to use the RMi system instead of the GSI with the Hoek-Brown failure criterion and this is described in Palmström & Stille (2010). Prior to the introduction of the GSI, the RMR value was used as the rock mass parameter input to the Hoek-Brown failure criterion, but this approach has been superseded by the introduction of the GSI.

Hoek & Diederichs (2006) proposed to reduce the overall modulus of rock mass to account for the disturbance in a rock excavation process. However, generally in Hong Kong, with good blasting practice and control of an excavation process, the disturbance should generally be low, but each case should be considered on its own merits.

(4) *Properties of Discontinuities*. Sufficient information on the discontinuities in a rock mass should be collected for characterisation. This is best achieved through a combination of engineering geological mapping (including API), detailed discontinuity logging of oriented cores and the use of televiewer data.

The rock mass should be split into different structural domains as appropriate. A commonly seen error is where all of the discontinuity data for a project is placed on a single stereonet and this is then used as the structural geological interpretation for the entire project, often incorrectly lumping together several different structural domains. The converse problem is also seen, where discontinuities are considered on a "televiewer-by-televiewer" basis. The relative data deficiency at a single drillhole, combined with the directional bias of the televiewer, leads to multiple differing structural geological interpretations across the site. An assessment based on a structural geological model of the site would lead to a different and more accurate interpretation of the discontinuity data.

An example of the characterisation of discontinuity sets is shown in Figure 6.13 (Atkins Arup JV, 2009a), which is the characterisation of subvertical discontinuities based on a core inspection.

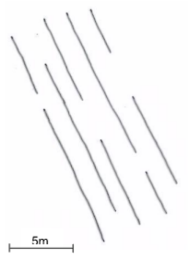
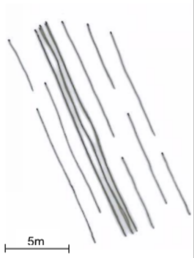
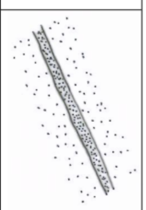
| Ref | Illustration | | Name | Aperture / Typical Thickness / infill | Joint wall Strength | Roughness (JRC) (From core only) | Persistence / Spacing | Key features |
|---|---|---|---|---|---|---|---|---|
| | Single Feature | Typical Occurrence | | | | | | |
| SV1 |  |   | Discontinuity | <1mm | 100 MPa | 6 – 8 | > 5 to 20m / 1 to 2/m | Discontinuities of limited persistence. Typically not mineral coated<br>Degree of weathering varies with weathering zone (prominent weathering of sidewalls has been noted using suffix W). |
| SV 2 |  |   | Major Discontinuity | Typically <1mm | 100 MPa (reducing to 50MPa in Zone B and C) | 5 – 7 (and N/A) | > 20m (no data ?) | Typically mineral coated, occasional slickensides.<br>Degree of weathering varies with weathering zone (prominent weathering of sidewalls has been noted using suffix W). |
| SV 3 |  |   | Major discontinuity typically with alteration | 10 to 50m wide zone (?)<br>Individual joints clay (kaolin) infilled (1 to 3mm) | 50 MPa (effect of alteration to be assessed) | 2 – 6 (and N/A) | > 20m | Closely spaced swarming of joints and incipient fractures. Discontinuities lined with kaolinite and with altered zone 100mm wide.<br>May be associated with an SV4 feature |
| SV 4 |  | | Major discontinuity (or possibly small fault) with completely altered zone | ? <1m | < 1 MPa | N/A | > 30m / ? | Structure identified at NE end of SYP in D211as single 0.7m thick feature within a wide SV3 zone. |
| SV5 | | | Tectonic fault structure - as possibly inferred by photolineaments<br>Not yet encountered in any borehole (other than possibly at UNV in D058) | | | | | |

**Figure 6.13 Example of Discontinuity Characterisation (Atkins Arup JV, 2009a)**

Figure 6.14 (Palmström, 1995) shows the common ranges of persistence of different types of discontinuities according to observations from field mapping.

The persistence ranges of joints, shear zones and faults shown in Figure 6.14 are consistent with those lengths found in the igneous rocks in Hong Kong. It is important to recognise the significance of the joint persistence in the numerical modelling approaches and to ensure that excessively conservative models of persistent multiple joint sets are not assumed.

Sheeting joints formed by exhumation and stress relief in a rock mass can be found down to a depth of around 30 m. These features can be very persistent, curved and undulating. Other low angle joints are commonly found in many tunnels and excavations, and they can be persistent. When excavations are close to faults or major shear zones, jointing typically becomes more persistent and orientated to the same strike as the faults or shear zones. Professional judgment should be applied when considering which joint sets may be more dominant than others within a proposed excavation. The nature of joints should be considered as well to cater for any variation of the same joint set, e.g. curvature in a persistent sheeting joint.

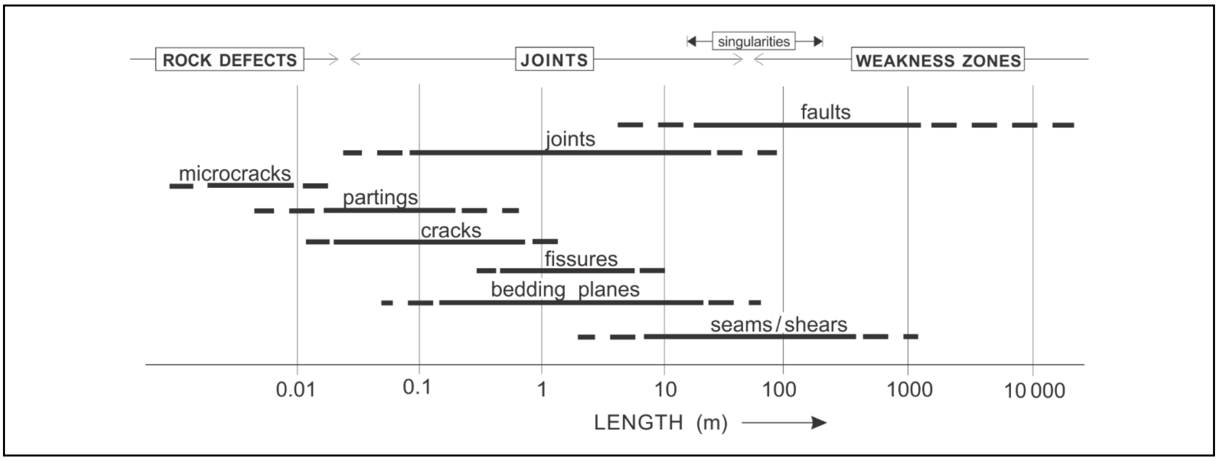



**Figure 6.14 The Main Types of Discontinuities According to Size or Persistence (adapted from Palmström, 1995)**

(5) *Block Size*. An assessment of the likely block sizes around a cavern development should be undertaken. Palmström & Stille (2010) presented a method for this purpose. Block size has particular importance with respect to characterising a rock mass, considering scale effects, selecting methods of analysis and designing appropriate rock support. The first quotient of the Q-system (viz. RQD/$J_n$) are sometimes used as a crude measure of block size in centimetres. However, this approach has its limitation as the maximum possible block dimension that can be calculated is 200 cm (which is determined based on the maximum RQD value of 100 and the minimum $J_n$ value of 0.5). In reality, blocks can be larger than 200 cm.

(6) *Qualitative Stability Assessment of Cavern Structures.* Palmström & Stille (2010) presented a method of classifying a rock mass to allow qualitative predictions of initial and

long-term behaviours following an excavation and assuming appropriate support has not been installed. The method comprises a number of figures with associated tables depicting different types of rock mass. These are particularly useful during the preliminary feasibility study and planning stages of a project. They can be used in conjunction with a simple rock mass characterisation to predict ground problems that may arise during construction if appropriate design measures are taken. The predictions can be considered in a risk register and used later during design as a reference point when evaluating the results of analysis.

Following the considerations suggested by Palmström & Stille (2010), it can be noticed that generally the main problems in Hong Kong that need to be designed against would be block falls. When weakness zones are present, further problems may include large-scale structurally controlled failures. Groundwater inflow will generally increase the chance of instability and may potentially result in water inburst and ravelling, particularly when faults are hydraulically connected to a body of water near the ground surface.

Another simple approach for considering the behaviour of a rock mass around an excavation is illustrated by the sketches in Figure 6.15, which show how failure modes may change with increasing discontinuity frequency, caused in this example by decreasing discontinuity spacing or increasing excavation size (Atkins Arup JV, 2009b).

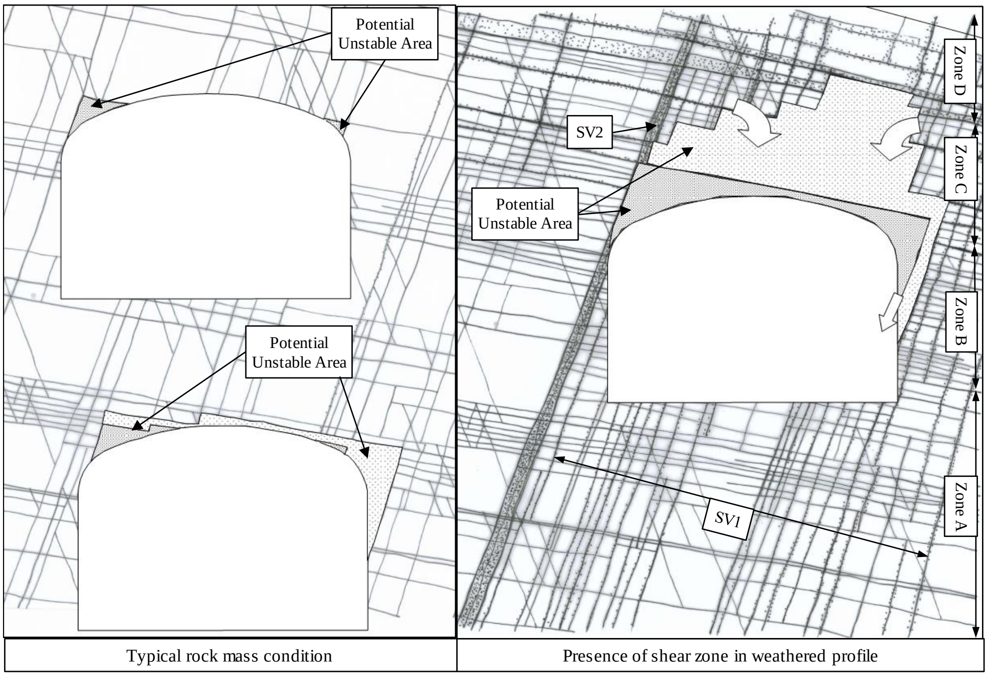
Figure 6.16 shows examples of comprehensive rock mass characterisations. The potential applications of such characterisations to rock engineering analysis are also presented.

### 6.6.3 Hydrogeological Model

The development of a hydrogeological model is analogous to rock mass characterisation. There is significant overlap between the two, as knowledge of a rock mass is essential to the understanding of the hydrogeological regime and vice versa. Therefore, both should be developed in parallel, drawing on many of the same sources of information and each informing the development of the other.

It is necessary to define the seasonal variations in the groundwater levels, in particular knowledge of the lowest levels. The large seasonal differences in rainfall and runoff from the hillsides in Hong Kong may result in some large seasonal variations in groundwater tables. Quite often in these areas the groundwater table during the dry season lies within bedrock and not in the overlying soil. Perched groundwater which exists locally within the soil would not normally be affected by cavern construction below rockhead. Where large groundwater drawdowns are expected in the soil, it is necessary to carry out detailed assessments of the potential impacts and whether excessive ground settlements are likely to occur as a result of groundwater drawdown.

The long-term performance of a tunnel or cavern also needs to be considered in terms of hydrogeological impacts. For instance, the selection of drained or undrained tunnel linings needs to be considered based on the analysis of changes to the groundwater regime due to the underground facility. Other considerations may include the effects on water catchments and water supply and whether recharge wells are required to mitigate the effects of ground settlement.

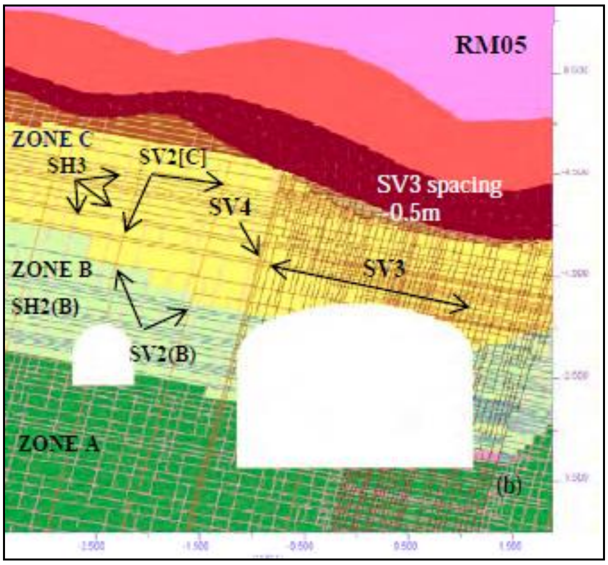



Notes:

(1) Zones A to D represents a graduation of the Weathering Zone with the best to the worst rock mass condition. Weathering Zones are classified based on rock mass condition determined from drillholes according to weathering grades, Total Core Recovery (TCR), Rock Quality Designation (RQD) and nature of subhorizontal joints.

(2) SV1 denotes the subvertical structure type comprising closely spaced, infilled with alteration, subvertical joints. SV2 denotes the subvertical structure type comprising discrete subvertical zone (shear zone) with alteration.

**Figure 6.15 Example Rock Mass Behaviour Models (modified after Atkins Arup JV, 2009b)**




| Description of rock mass | | Analysis | | Classification |
|---|---|---|---|---|
| | Intact massive s>2m D/s < 5 | Continuum | FEM, BEM | Q >1000 RMR >80 GSI >80 |
| | Jointed s=0.5-2.0m D/s=5-20 | Discontinuum | DEM | Q = >10 - 100 RMR = 61-80 GSI =>40-80 (applicable?) |
| | Bedded s=0.1-0.5m D/s=20-100 | Discontinuum | DEM | Q = >1 - 10 RMR =41-60 GSI=>30-40 (applicable?) |
| | Very densely jointed s=0.05-0.1m D/s=100-1000 | Discontinuum / Equiv. Contin. | DEM | Q = 0.1-1 RMR=21-40 GSI =>20-30 (applicable?) |
| | Tectonically disturbed s<0.1m D/s>500 | Equiv. Contin. | FEM, FDM | Q < 0.1 RMR<21 GSI<20 |
| | Rockfill or talus material d=0.01-0.5m D/d >10000 | Continuum | FEM, FDM | Q <0.01 RMR, GSI not applicable |

**Figure 6.16 Example Components of a Rock Mass Characterisation (after Bandis et al, 2011)**

The development of a hydrogeological model requires a well planned and effectively implemented site investigation, in-situ testing and long-term monitoring. In addition, the following information is required:

(a) rainfall characteristics,

(b) drainage catchment characteristics,

(c) existing groundwater levels and seasonal variations,

(d) geology and ground characteristics including permeability and groundwater chemistry,

(e) aquifer orientation and spatial distributions,

(f) subsurface distribution and stratigraphy, and

(g) interaction of different soil and/or weathered rock layers (e.g. groundwater pressure gradients).

Appropriate hydrogeological boundaries should be adopted in a hydrogeological model. Qualitative predictions of the possible groundwater response due to a proposed works can be made in light of the model. If more detailed hydrogeological analyses are required, the hydrogeological model can form the basis of the analyses. In addition, the hydrogeological model provides inputs into rock mass classification systems, analytical solutions and numerical analyses for stability assessment and design of ground support and groundwater control measures.

GEO (2007) provides further guidance on how to assess the hydrogeological impacts of a project. Figure 6.17 shows an example of a hydrogeological model.

### 6.6.4 Uncertainties and Risk

No matter how comprehensive a rock mass characterisation is, uncertainties and risks always remain. Limitations of a site investigation also result in uncertainties and risks. For example, due to scale effects, the results of laboratory shear strength testing of discontinuities may not be directly applicable to the assessment of the overall rock mass behaviour around an excavation (although they may be of most relevance to the assessment of individual block falls). Nevertheless, sufficient resources and time should be allowed for a site investigation, including laboratory tests and field tests, to attain an acceptable level of residual risks.

The residual risks should be managed using a geotechnical risk register. In the early stages of a project, these risks can be used to guide and focus the site investigation. Later, during design, they can be used to identify potential issues that must be designed out. At the construction stage, workers should be briefed on the risks that may endanger them or the project. The remaining minor uncertainties and residual risks accepted during the implementation of the project can be taken over by the client and the operator so that they can be managed and reviewed through the operational phase and the lifespan of the facility.

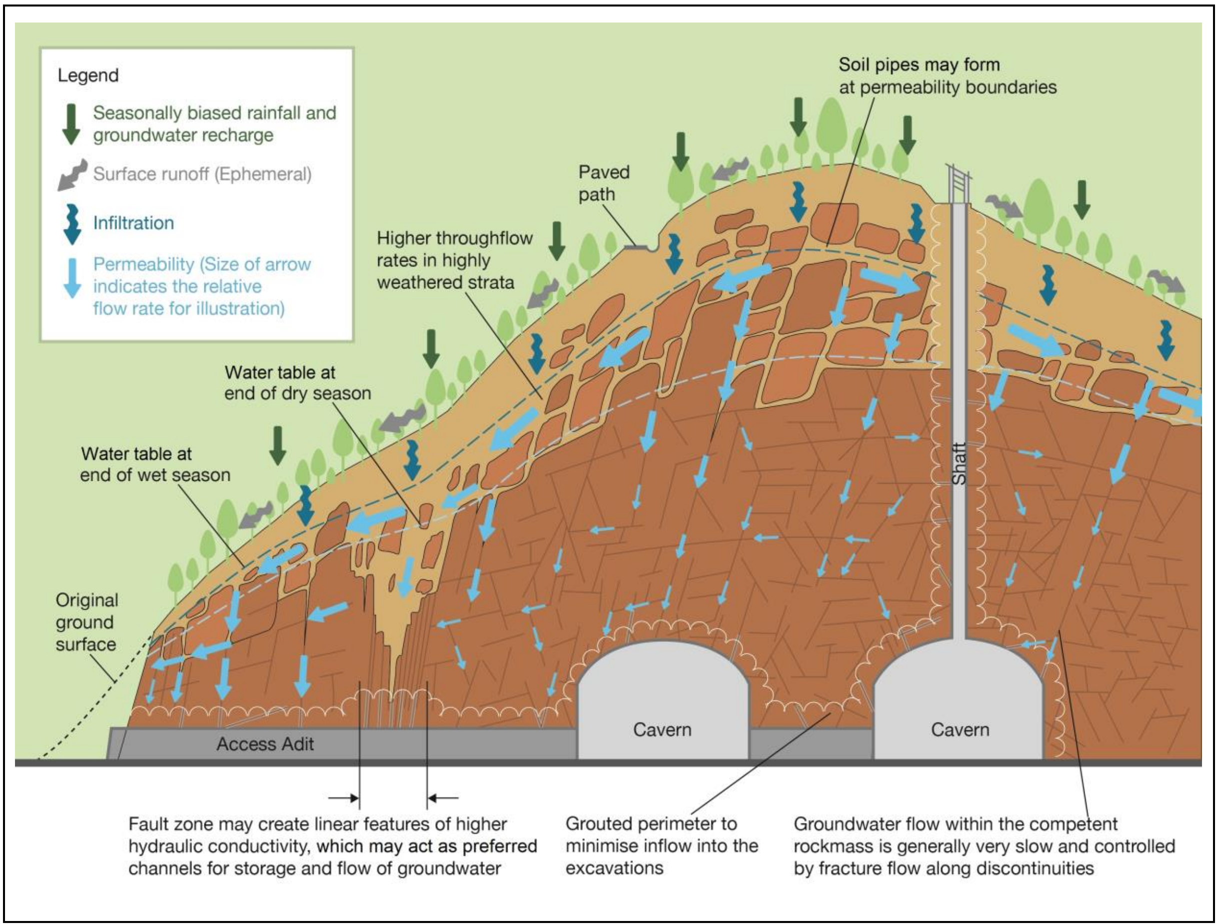



**Figure 6.17 Example of a Hydrogeological Model**

## 6.7 Design Models

### 6.7.1 General

A cavern development may require many different design models as, unlike geological and ground models, design models are intimately linked with the method of analysis that is used. A design model is concerned primarily with the assessment of the response of the ground to the proposed works and vice versa for use in a geotechnical assessment or engineering design. Design models for empirical, prescriptive and quantitative designs depend on the engineering application, degree of conservatism in the empirical/prescriptive models and the level of geotechnical risk. The development of design models is iterative and the models will evolve as a project progresses. In all cases design models require a ground model to be appropriately simplified to allow for analyses. The nature of a particular design model and the degree of simplification required will depend on a variety of factors including the nature of the engineering problem, the project stage, the data available and the type of analysis that is to be employed. The design models for most rock mass classifications and analytical solutions are relatively simple. Conversely, the design models for numerical analysis can become very detailed and sophisticated. Barbour & Krahn (2004) provided useful guidance on the development of design models and the following good practice:

(a) It is essential that design models are based on a conceptual model of the ground and an understanding of the material properties and the likely ground behaviour (e.g. a ground model).

(b) Although design models are simplified abstractions of reality, the theoretical basis of the models must be understood.

(c) The modelling process should always begin with the simplest possible conceptual model. Complexity should be added incrementally until the level of complexity is sufficient to represent the material behaviour of interest. This approach has a number of advantages:

   (i) it is easier to spot errors,

   (ii) it is often possible to make initial checks using simple hand calculations or analytical solutions, and

   (iii) it assists with the development of an intuitive understanding of the design model and its sensitivity.

(d) All models are data deficient. Dealing with data deficiency means the complexity of the theories and models must never be increased beyond the level of the data sufficiency. Therefore, complicated design models should not be developed on the basis of very uncertain ground models with limited data.

(e) It is essential that the inputs and outputs of the design model are comprehensively verified, including the use of sensitivity analysis.

(f) All design models should only be considered as conditionally valid. In particular, the results of computer modelling software should not be accepted blindly. The solution provided is only one step in the design process. The design should only be based on the clear thinking and judgment of the designer.

(g) Engineering experience and judgment are essential and should be exercised in the development of the design models. Attention should be paid to any model result which contradicts sound engineering intuition.

(h) A record of the simplifications and assumptions that have been made in the development of the design models should be kept. This record should be included in the design reports and the geotechnical risk register as appropriate.

Further information on design models is given in GEO (2007). It should be noted that the above points are equally applicable to the numerical modelling of groundwater response. Other guidance on development of design models is provided below.

### 6.7.2 Design Methods and Parameters

Designers should use relevant structural codes for structural design of cavern support elements. The loadings adopted in a design should be evaluated in accordance with the principles given in this Geoguide. Generally, unfactored parameters should be used in calculations, but this would depend on the requirements of the structural code adopted. In the commonly-used limit state structural design codes, ultimate limit state loads are evaluated based on unfactored parameters. These are then multiplied by appropriate partial load factors in the checking of limit states.

For geotechnical stability assessment and design of caverns, the global factor of safety method is commonly adopted (see Section 6.8.4). The limit state partial factor method may also be used. For such case, reference should be made to Geoguide 1 (GEO, 2017d) for guidance on partial factors, the principles of selection of geotechnical parameters and the approach to obtain selected values for geotechnical assessment and design. It should, however, be noted that use of factored values of geotechnical parameters is not appropriate in empirical methods of design based on rock mass classification systems such as the Q-system, RMR and RMi, which already have safety margins implicitly incorporated.

### 6.7.3 Scale Effect

Goodman (1989) noted that it is known from experience that a small tunnel is more stable than a large one, everything else being equal. It is also generally known that strength tends to decrease with increasing sample size. This is because of the scale (or size) effect and the related concept of the representative elemental volume (REV), which is discussed by Hudson & Harrison (1997). The scale effect can be introduced in real tunnels not only by including the additional load due to gravity acting on the rock near tunnels, but also by introducing scale effects in material strength and rock mass behaviour. As a greater number of discontinuities are contained within a sample of rock, its strength must decrease. Accordingly, when the span of an opening is many times greater than the average spacing between discontinuities, the opening cannot be expected to stand without external support. The scale effect and REV also have significant implications for the investigation and assessment of in-situ stresses as discussed by Hudson & Harrison (1997).

The implications of increasing joint density are illustrated in Figure 6.18, which specifically relate to the applicability of the Hoek-Brown failure criterion at different scales, and are relevant more generally to the selection of appropriate analysis techniques, particularly numerical analyses. The largest circle shown in Figure 6.18 can be taken to illustrate the scale at which a rock mass may be analysed as a continuum, the middle circles indicate scales at which the rock may be analysed as a discontinuum and the smallest circles represent individual blocks and intact rock where analytical methods such as kinematic analysis and key block theory are more appropriate.

The scale effect concept is also directly relevant to the considerations of groundwater flow through a rock mass as discussed further by Hudson & Harrison (1997) in terms of the REV. They noted that, based on considerations of scale effects and discontinuity distributions, for almost all rock tunnels there will be lengths where there will be little inflow, and lengths where there may be high inflow. In general, it will not be possible to predict the specific local water inflow. Additionally, it will generally be unknown how such a network is connected to the regional hydrogeological regime. Therefore, a design should include defensive strategies against local high water inflows on the basis that the precise location of these flows cannot be predicted. Hudson & Harrison (1997) noted that there is no simple procedure for establishing the permeability of a rock mass. An assessment should be carried out to evaluate the sensitivity of the results of a hydrogeological analysis based on a reasonable range of permeability.

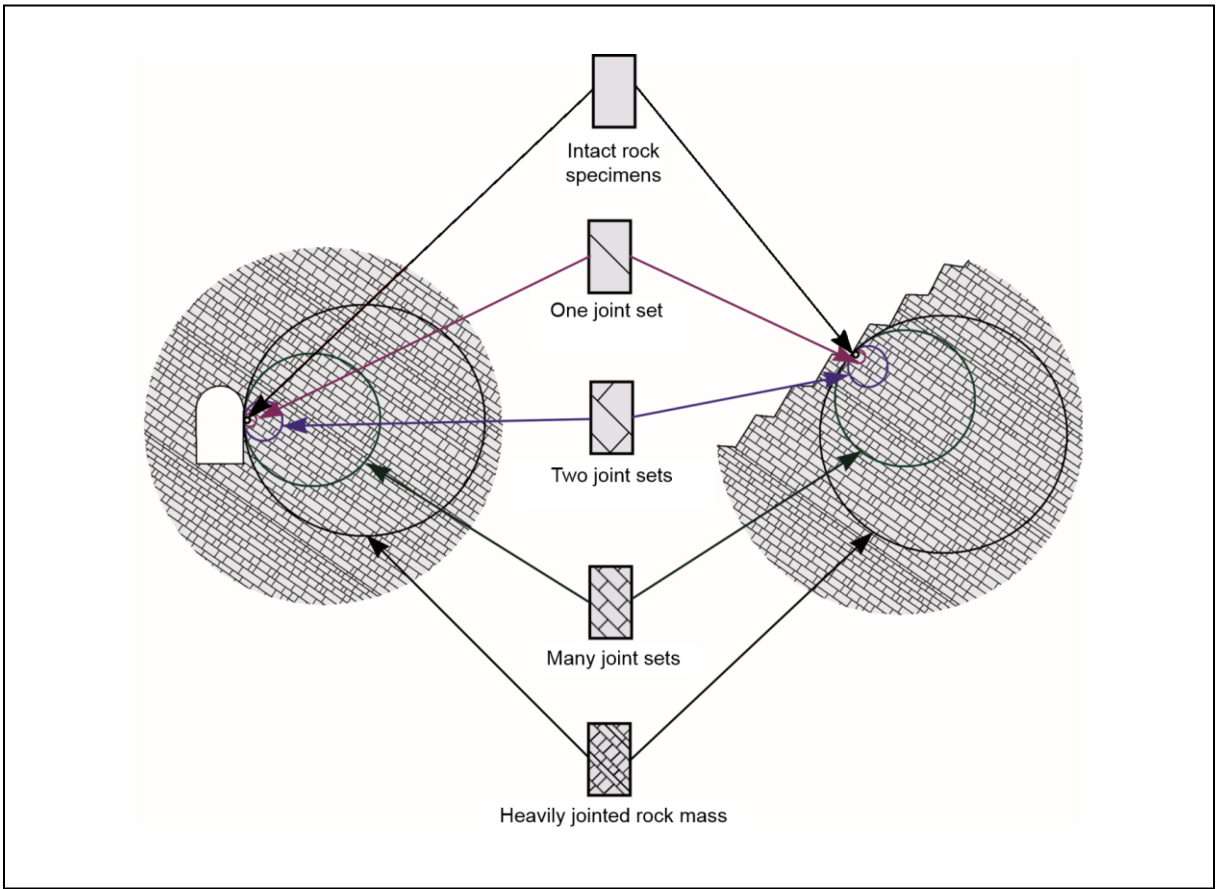



**Figure 6.18 Idealised Diagram Showing the Transition from Intact to a Heavily Jointed Rock Mass with Increasing Sample Size (after Hoek, 2007)**

### 6.7.4 In-situ Stress

Different methods of analysis consider and incorporate in-situ stresses in different ways. It is essential that the effects of in-situ stresses, if any, should be considered either explicitly or implicitly in various analyses for a cavern design.

## 6.8 Design for Cavern Structures

### 6.8.1 Failure Modes

(1) *General*. Failure modes in caverns depend on the characteristics of a rock mass, which is dependent on the strength of the intact rock and discontinuities, as well as the in-situ stresses. Two types of failure modes, viz. structurally controlled failure and stress-induced failure, should be considered in a cavern design.

(2) *Structurally Controlled Failure*. This type of failure involves kinematically permissible sliding on adverse joints or block falls. Examples are detachments of rock blocks, wedges or fragments, formed by intersection of pre-existing structural features, from free faces formed after excavation, including roof, sidewalls and heading face. For weakness zones or fractured rocks in discontinuities, detachment of large volume of rock fragments or partially weathered materials can also occur. Structurally controlled failures are often the dominating type of instability for caverns at modest depth in Hong Kong where the stress level relative to the rock mass strength is generally low.

(3) *Stress-induced Failure*. Stress-induced failures such as slabbing, rock bursting and squeezing will occur when rock materials are subject to a high external or in-situ stress. Portions of a cavern affected by stress concentration at localised areas (e.g. pillars, intersections, and areas affected by high external loads, e.g. load from foundations in the vicinity of the cavern) are also vulnerable to stress-induced failures and their stability should be duly assessed.

Figure 6.19, based on Martin et al (1999), illustrates the possible failure modes according to the level of in-situ stresses and the fracture conditions of a rock mass.

### 6.8.2 Sources of Loading

(1) *General*. Cavern structures, including any structural support elements, should be designed to meet the requirements for stability and serviceability under the actions of various loads. Common sources of loading include, but are not limited to, the following:

(a) groundwater pressures,

(b) surcharge,

(c) ground loads,

(d) stresses induced by adjacent openings,

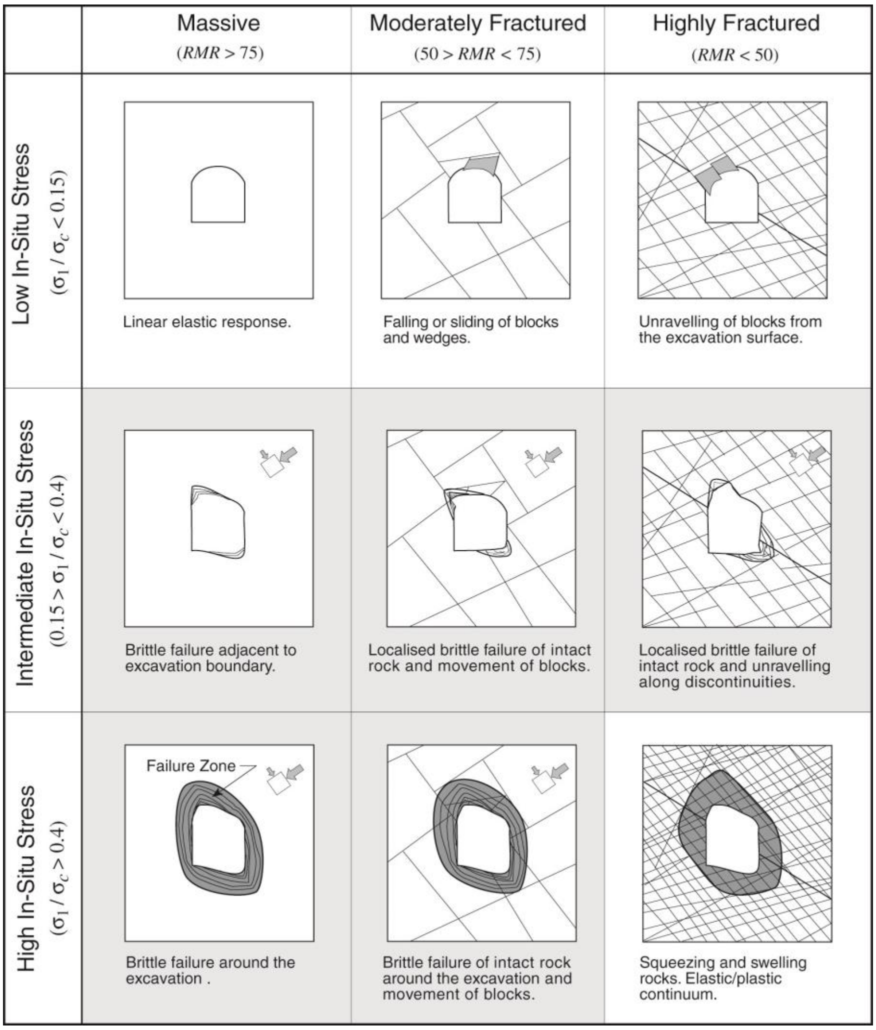



**Figure 6.19 Illustration of Failure Modes (modified from Martin et al, 1999)**

(e) seismic loads,

(f) dynamic loads during operation of the facilities in the cavern or in the vicinity (e.g. machines), and

(g) other loads, including shrinkage and thermal loads, fire load, grouting pressures, future development loads, loads due to installations and utility services supported by the cavern structures, etc.

It is important for designers to appreciate the relevant sources of loading on the subject cavern. A systematic identification of sources of loading pertaining to the design of the cavern structures should be carried out based on the ground conditions, presence of adjacent facilities and structures, construction methods, etc.

The sources of loading should be either accounted for by an empirical method or an analytical/numerical assessment. Some of the loadings may fall beyond the capability of some empirical methods. For example, the Q-system does not account for stress concentrations at intersections of caverns, and the loads from foundations close to caverns. In such cases, an appropriate method or combination of methods should be adopted.

A detailed discussion on the sources of loading is provided below.

(2) *Groundwater Pressures*. Effects of groundwater pressure, including cleft pressure on rock joints and porewater pressures in soils, should be duly considered in the design.

(3) *Surcharges*. Surcharge loads can be loadings from foundations of adjacent structures. If a cavern is located at a shallow depth, appropriate loadings on the ground surface should also be included.

(4) *Ground Loads.* Ground loads include overburden from soils or weakness zones and loading from the rock mass. Design of adequate supports to overburden from soils is not uncommon for cavern structures with a shallow rock cover. The overburden should be determined with consideration of the resistance along the plausible failure surfaces, if relevant. Depending on the width and extent of a weakness zone, some empirical methods (e.g. the Q-system) can cover the design of support measures to the weakness zone.

Loading from a rock mass is controlled by its failure modes discussed in Section 6.8.1. The loading will be pertinent to a potential rock wedge or rock block, when a structurally controlled failure is of concern. On the other hand, when carrying out excavation in a rock mass with high in-situ stresses, squeezing load from the rock mass is relevant.

(5) *Stresses Induced by Adjacent Openings*. The alteration of stress fields due to other openings adjacent to a subject cavern should be duly considered in a design. For example, additional stresses will be induced at cavern intersection areas or rib pillars. For pillars constructed in a regular pattern, the average stress in the pillars can be estimated by a tributary area method (Hoek & Brown, 1980) as shown in Figure 6.20.

The tributary method is not valid for irregularly spaced pillars. Numerical methods should be adopted for assessment in this case.

(6) *Seismic Loads*. Underground structures perform better during seismic events than above ground structures. Hong Kong is situated in a region of low to moderate seismicity. Seismic loads are generally not critical and therefore need not be included in the geotechnical design of cavern works. However, for caverns housing high consequence facilities and major lifelines (e.g. LPG storage plants, power plants, etc.), it is recommended that ground distortion is considered in a geotechnical design and the stability of a structure to cope with the resulting displacements is checked. Where caverns and significant elements of excavation support cross contrasting geological boundaries, significant changes in imposed shear strain may occur resulting in more severe local distortions in a cavern structure. Contrasting boundaries may be soil to rock interface, major fault zone, etc. The design magnitude of an earthquake should be determined based on a site-specific seismic hazard assessment with consideration given to the level of the cavern.

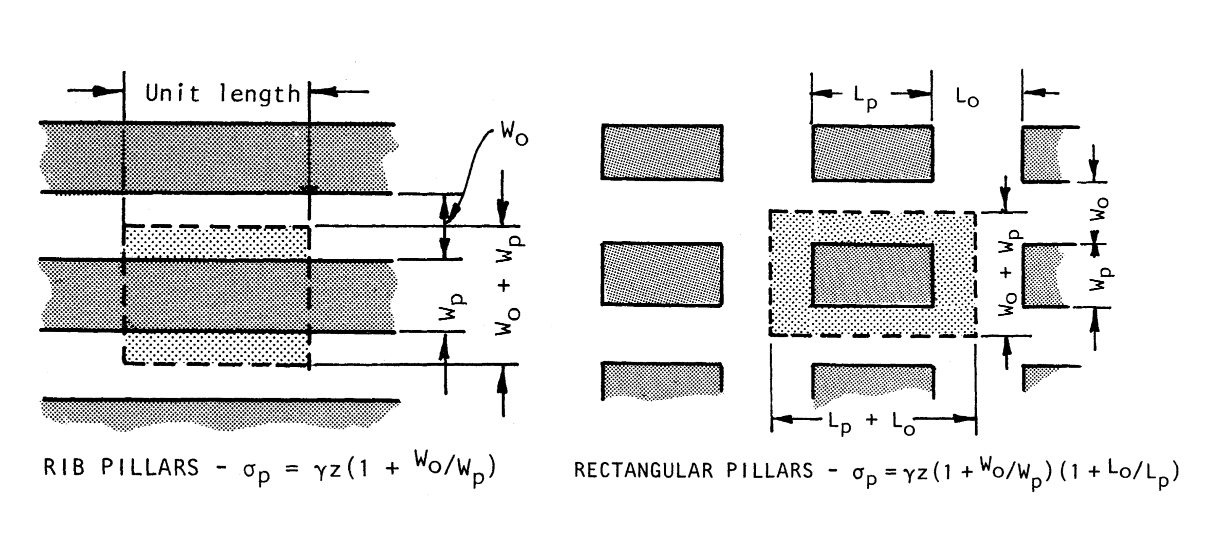



**Figure 6.20 Calculation of the Average Pillar Stress Using Tributary Method (after Hoek & Brown, 1980)**

(7) *Dynamic Loads during Operation of the Facilities in the Cavern or in the Vicinity*. This type of loading is applied to the design of permanent support design only, and is not relevant to the temporary stability and temporary support design of caverns.

(8) *Other Loads*. Other loads, including shrinkage and thermal loads, fire load, grouting pressures, utility loading, etc., should be considered as appropriate. Inclusion of grouting pressures into the design of cavern lining should not be overlooked when post-excavation grouting for groundwater inflow control purposes is carried out after construction of the permanent lining.

The drill and blast construction method has conventionally been used for cavern excavation. Local experience has indicated that with appropriate blast control to limit the peak particle velocity (PPV), consideration of loading on a cavern designed and built to current safety standard due to blasting is normally not required.

### 6.8.3 Temporary and Permanent Supports

Temporary support is required to control deformations and secure safe working conditions for construction crews, and should be installed shortly after excavation and removal of spoil are completed. It normally comprises structural elements and may be used in conjunction with ground improvement works (e.g. grouting). Key dimensions of an excavation (e.g. the design length of advance, sequence and extent of staged excavations if applicable, inclination of excavation faces, etc.) pertinent to the design of temporary support should also be specified for construction.

Permanent support is required to maintain stable rock conditions in an excavation during the service life of a cavern. The permanent support should also be designed to meet the serviceability, design life and durability requirements specified by the client, with reference made to the relevant structural code (e.g. BD, 2011b & 2013; BSI, 2004, 2005a, 2005b & 2008). Unless the design life and durability requirements of the temporary and permanent supports are compatible, the temporary support should not be taken as contributing to the capacity of the permanent support. When permanent structural elements, e.g. walls and slabs, are used as temporary support to an excavation, the design of such permanent structural elements should be compatible with the method of construction.

### 6.8.4 Stability Criteria

In cavern design, where analytical and numerical solutions are used for structurally controlled failures, the global factor of safety method is commonly adopted. The global factor of safety is often defined as the ratio between the capacity (strength or resisting force) and the demand (stress or disturbing force) (Hoek et al, 1995; Hoek, 2007). A minimum global factor of safety of 1.5 is recommended for design and estimating the required working loads of supports against structurally controlled failures, including failure of isolated blocks or blocks with discontinuities with an adverse geometry.

For pillar stability, in addition to the necessary stability check against structurally controlled failures (see Figure 6.12), an assessment should be carried out to confirm that the load-carrying capacity of the pillar exceeds the pillar stress by an adequate factor of safety. The load-carrying capacity of pillars should be established using the Hoek-Brown failure criterion, empirical methods and/or numerical methods for jointed rock masses as appropriate with consideration given to the height-width ratio of pillars and potential brittle behaviour. Martin & Maybee (2000) provided a review of case studies for hard rock mines and the results of finite element analyses. With reference made to Martin & Maybee (2000) and experience from local practice, a minimum global factor of safety of 1.5 is recommended.

Cavern structures designed using well-established empirical methods (Section 6.8.6) are deemed to satisfy the stability criteria and possess an adequate factor of safety provided that such methods are applied carefully within their intended scopes. In circumstances where empirical methods are not applicable, the stability of the structures should be assessed separately using suitable design tools.

For design of cavern structures where the induced stress may exceed the strength of the rock mass (e.g. caverns with large span or complex geometry or involving multiple openings, excavations in an extensive weakness zone, excavations in a rock mass with high in-situ stresses, and construction involving staged excavation, etc.), an appropriate stress analysis should be undertaken. The cavern stability should be assessed based on the calculated stress conditions including the extent of yielded zones and the presence of tension zones. Yielded zones and tension in rock close to the excavated surface indicate potential for rock spalling and rockfalls respectively. Appropriate supports should be provided in these areas to ensure cavern stability.

### 6.8.5 Methods of Analysis

Rock mass classification systems are typically used as the starting point in a design process, beginning at the feasibility study stage and continuing through the project planning and preliminary design stages. They can also form the basis for determining rock support, especially the initial support, during construction for situations falling within the empirical database upon which the system is based.

For more complicated settings, such as large span caverns in poor quality rock masses or where there are multiple junctions, rock mass classification systems should be used in combination with other appropriate design tools, such as analytical solutions and numerical analyses.

It is essential that the designer is competent in the use of all of the methods of analysis adopted. Competence comprises knowledge of the theoretical basis, experience in the use of the methods, appreciation of the appropriate applications and knowledge of the assumptions, limitations, advantages and disadvantages of the methods. In general, a detailed design should preferably be based on at least two methods of analysis to improve confidence and facilitate optimisation in the design.

### 6.8.6 Empirical Methods

(1) *General*. There are a number of empirically based rock mass classification systems available. They can be applied to cavern design in appropriate circumstances. The empirical methods are established based on the type and quantity of support that have been applied previously in rock masses of similar qualities and under similar circumstances. They provide guidance on rock support decisions and documentation for rock mass quality. The assessment of their input parameters is relatively uncomplicated. It places a focus on the rock engineering and aims to obtain a careful and thorough scrutiny of a rock mass.

This Geoguide discusses the most commonly used rock mass classification systems, namely, the Q-system, the Rock Mass Rating (RMR) and the more recently developed Rock Mass Index (RMi). Other rock mass classifications are also available in the literature, such as Hoek (2007) and Singh & Goel (1999).

The use of more than one of the empirical systems together, as described by Palmström (2009), allows a better understanding of the required ground support to be developed. Such an approach is particularly beneficial for complex and difficult ground conditions.

Parameters selected for use in rock mass classification systems should be representative of the ground. One method to assist with the selection of appropriate parameters for use in rock mass classifications is to use histograms during data collection (Barton et al, 1994; Barton, 2002; NGI, 2015). However, these empirical methods are subject to a number of key limitations in general. As for other empirical methods for design of other engineering structures, rock mass classification systems are only applicable when used within the scope of their empirical databases. They are most suitable for assessing structurally controlled failures driven by gravity. Other rock mass failure modes and additional loading induced by adjacent facilities, e.g. foundations, shallow rock cover and ground-support interaction, should be adequately considered in the design.

Classification systems tend to promote generalisations that are in some cases inadequate to describe the full range of specifics of rock masses. Their simplicity of use can mask the critical requirement for engineering judgment and experience. In cases where the dominant potential failure mechanism is not identified in a classification system, such as a persistent major sub-parallel shear or fault zone, the use of the empirical approach by itself may not be sufficient. The considerations of complex cavern geometries, multi-excavation interactions, seismic or dynamic loading, and durability and serviceability requirements are also not given. More detailed discussions on the general advantages and limitations of using empirical methods in rock engineering can be found in Palmström & Stille (2010), Hoek (2007), and Barton & Grismtad (2014). Other specific limitations of individual rock mass classification systems are further discussed in the corresponding sections.

Rock mass classification schemes are used at different stages in a project. In the project planning and design phases, these schemes can be used for the evaluation of design feasibility, required cavern support, and required construction time and cost. They are intended to reflect the designer's overall impression of the rock mass and to serve as an aid in assessing and comparing rock masses rapidly for later, more rigorous analysis. For the construction phase, rock mass classification schemes are useful for general verification of rock support level and for recording of engineering geological conditions.

(2) *Q-system*. It was developed by the Norwegian Geotechnical Institute (NGI) and the original version is described by Barton et al (1974). Since then, the system has been continuously updated to include more case histories and take into account the advancement in technology (e.g. Grimstad & Barton, 1993; Barton & Grimstad, 2014; NGI, 2015). This empirical method is based on the rock quality designation, RQD (Deere, 1963) and five additional parameters, which account for the number of discontinuity sets, the discontinuity roughness and alteration (infilling), the amount of water and various adverse features associated with loosening, high stress, squeezing and swelling. This classification system is based on the supports installed for more than 2,000 case histories. The Q-system also takes into account the span and the intended use of an excavated space. It is generally considered to work best in ground conditions where block falls is the most likely failure mode.

The Q-value is expressed by the formula:

$$Q = \frac{\mathrm{RQD}}{J_\mathrm{n}} \times \frac{J_\mathrm{r}}{J_\mathrm{a}} \times \frac{J_\mathrm{w}}{\mathrm{SRF}} \quad \text{..............................................} \quad (6.4)$$

where RQD = rock quality designation
$J_\mathrm{n}$ = joint set number
$J_\mathrm{r}$ = joint roughness number
$J_\mathrm{a}$ = joint alteration number
$J_\mathrm{w}$ = joint water reduction number
SRF = stress reduction factor.

The numerical value of $Q$ ranges from 0.001 (for exceptionally poor quality squeezing ground) to 1,000 (for exceptionally good quality and massive rock). The six parameters can be estimated from site investigation results and verified during excavation. The parameters are grouped into the following three quotients:

(a) RQD/$J_n$, which represents the degree of jointing or block size,

(b) $J_r/J_a$, which represents the inter-block shear strength, and

(c) $J_w$/SRF, which consists of two stress parameters representing an empirical factor describing the "active stresses".

The guidance on the application of the Q-system in Hong Kong is provided in Appendix D. The use of the Q-system requires detailed engineering geological mapping and analysis of all the geological features encountered. The rock support evaluated from the Q-value and the corresponding tables give only probable amounts and support types to be used. During construction of an underground opening, the rock support types and quantities should be adapted to the observed rock conditions. The heterogeneous nature of rock masses precludes the design of definitive and cost-effective support systems prior to excavation.

The Q-system is extensively used throughout the design and construction process in Hong Kong and has been found to provide good design estimates of the required support installed during construction when used by competent site staff. Its applications in Hong Kong have been discussed by Barton (1989).

As there have been a number of updates to the Q-system, it is important that the most recent versions of the Q-system parameters and support charts are considered by the designer and used as appropriate. NGI (2015) advised that if $Q < 1$, then deformation measurement and numerical analysis should be carried out in addition to the use of the Q-system. The document also noted that the support proposed by the Q-system is considered to be conservative for good quality rock masses.

Empirical relations have been proposed to relate $Q$ to various rock mass properties such as rock mass deformation properties (Barton et al, 1980 & 1985; Grimstad & Barton, 1993; Barton, 2002 & 2007). The Q-value may also be used as a way to estimate the $m$ and $s$ factors in the Hoek-Brown failure criterion (Hoek et al, 2002, Hoek & Diederichs, 2006), as follows:

$$\mathrm{GSI} = 9 \ln Q' + 44 \quad \text{.................................................} \quad (6.5)$$

$$m_\mathrm{b} = m_\mathrm{i} \mathrm{e}^{\left(\frac{\mathrm{GSI}-100}{28-14D}\right)} \quad \text{...................................................} \quad (6.6)$$

$$s = \mathrm{e}^{\left(\frac{\mathrm{GSI}-100}{9-3D}\right)} \quad \text{.....................................................} \quad (6.7)$$

$$E_\mathrm{m} = E_\mathrm{i} \left( 0.02 + \frac{1 - \frac{D}{2}}{1 + \mathrm{e}^{\left(\frac{60+15D-\mathrm{GSI}}{11}\right)}} \right) \quad \text{.....................................} \quad (6.8)$$

where GSI = Geological Strength Index
$Q'$ = RQD/$J_n \times J_r/J_a$
$m_i$ = material constant
$m_b$ = reduced value of material constant
$s$ = rock mass constant
$D$ = disturbance factor
$E_m$ = rock mass modulus (in GPa)
$E_i$ = intact rock modulus (in GPa).

Extensions of the Q-system have been proposed for predicting the occurrence of squeezing ground conditions (Singh et al, 1992; Singh, 1993). However, these conditions are generally not applicable to caverns in Hong Kong. In addition, similar to all other empirical equations, such relationships and extensions to the Q-system are only valid under particular circumstances and should be used with caution.

(3) *Rock Mass Rating (RMR) System*. This is a geomechanics classification (Bieniawski, 1976 & 1989) which provides a general rating for classification of a rock mass with values ranging from 0 to 100 based on the following six parameters:

(a) uniaxial compressive strength (UCS) of the intact rock,

(b) rock quality designation (RQD),

(c) discontinuity spacing,

(d) condition of the discontinuity surfaces,

(e) groundwater conditions, and

(f) orientation of the discontinuities relative to the engineered structure.

The RMR system was developed originally based on South African tunnelling and mining experience, so its applicability to projects in Hong Kong is limited.

Some elements of the system, particularly the assessment of favourable excavation orientation, appear to be most applicable to rock masses with one dominant discontinuity set, such as sedimentary rocks. These rock masses are not commonly encountered in Hong Kong underground works.

The RMR support chart is based on 10 m-wide horseshoe-shaped tunnels where the vertical stress is below 25 MPa. The 10 m span is a particular limitation for cavern design. Furthermore, the support recommendations have not been updated to cover modern support measures such as fibre reinforced shotcrete.

On the basis of the above considerations, it is suggested that the RMR system support chart offers no advantages over the use of the Q-system support chart. However, it may be used as a point of reference for adit and tunnel design where stability is controlled by block falls and in combination with the Q and RMi systems as described by Palmström (2009).

Bieniawski (1984) proposed an approximate relationship between Q-value and RMR. The Q and RMR systems include different parameters and therefore cannot be strictly correlated. As noted by Palmström & Stille (2010), the relationship has an inaccuracy of ± 50% or more. Therefore, the use of this relationship is not recommended. It is suggested to evaluate RMR based on first principles instead.

The RMR has been related to stand-up time against unsupported span (Bieniawski, 1976). While this may be useful for some of the failure modes in weak rocks such as spalling, the relationship is of limited use for caverns constructed in Hong Kong igneous and volcanic rocks which are dominated by structurally controlled failures. In such cases, the unstable blocks will fail anytime after excavation and support is required to be provided as early as possible for safety. However, the stand-up time chart may be relevant to weakness zones.

Other applications of the RMR system include derivation of the rock mass deformation modulus, the *m* and *s* factors in the Hoek-Brown failure criterion and GSI parameters for use with the Hoek-Brown failure criterion (Hoek & Brown, 1988):

$$\frac{m_\mathrm{b}}{m_\mathrm{i}} = \mathrm{e}^{\left(\frac{\mathrm{RMR}-100}{28}\right)} \qquad (6.9)$$

$$s = \mathrm{e}^{\left(\frac{\mathrm{RMR}-100}{9}\right)} \qquad (6.10)$$

$$E = 10^{\left(\frac{\mathrm{RMR}-10}{40}\right)} \qquad (6.11)$$

where RMR = Bieniawski's rock mass rating
$m_\mathrm{i}$ = material constant
$m_\mathrm{b}$ = reduced value of material constant
$s$ = rock mass constant
$E$ = Young's modulus of the rock mass (in GPa).

Equations 6.9 and 6.10 are only applicable for undisturbed (or interlocking) rock masses. In the case of disturbed rock masses, the value 28 should be replaced by 14 in Equation 6.9 and the value 9 should be replaced by 6 in Equation 6.10 (Hoek & Brown, 1988).

(4) *RMi System*. The rock mass index (RMi) is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. The RMi system was formulated by Palmström (1995) and has since been further developed. It makes use of the UCS of an intact rock and the strength reduction effect of the discontinuities penetrating the rock.

The RMi value can be applied as input to other rock engineering methods, such as numerical modelling and the Hoek-Brown failure criterion for rock masses (Hoek & Brown, 1980), and to estimate the deformation modulus for rock masses (Palmström & Singh, 2001). It can also be used for estimating rock support using a chart by Palmström & Stille (2010).

The RMi system requires more calculations than the RMR and Q-system and is less convenient for use during fieldwork, core logging and tunnel mapping. Similar to the Q-system, the RMi system is best suited to massive and discontinuous rock masses and is better at identifying the general block size of a rock mass and the rock bolt length needed to support the blocks. Its use is more limited for weakness zones, but it can be used to provide a preliminary support estimate. The adoption of the system has been limited in Hong Kong and the database of examples is less comprehensive compared with other systems. Barton (2002) discussed other limitations of the system.

The RMi system does take into account factors that the other systems do not and it may be used in conjunction with the Q and RMR systems as described by Palmström (2009).

### 6.8.7 Analytical Methods

(1) *General*. Analytical solutions can be useful for developing an understanding of rock mass stability and assessing support requirements. The use of analytical solutions is particularly suitable for relatively simple scenarios. Analytical solutions can also be used to lend confidence to the findings of empirical methods and validate the results of more sophisticated design methods, such as numerical analyses. These methods can also be used during construction to back up the judgment of engineering geologists and tunnel engineers.

Caverns in Hong Kong are typically situated at modest depths in hard rock where the stability is governed by weakness zones and intersections of discontinuities. The main analytical solutions used for cavern developments in current Hong Kong practice are described below.

(2) *Limit Equilibrium Method*. This method in various forms is often used to determine the stability of blocks and wedges at excavated rock surfaces and the support required to achieve stability, as discussed by Hoek & Brown (1980). Similar analyses can be used for determining pillar stability against structurally controlled failures and their support requirements.

(3) *Block Theory*. Also referred to as the key block analysis, the theory can be used to determine which blocks in a cavern roof or wall control stability. It can be applied to predict the likely location and appearance of key blocks using statistical discontinuity data or discontinuity maps taken from an excavation. The method can assist in the recognition of key blocks in an excavation based on the observed discontinuity patterns on the excavation faces. It can help the designer to determine the locations of support to rock blocks. Details of the theory are described by Goodman & Shi (1985). In practice, a key block analysis is usually carried out using stereographic projections. The analysis has been incorporated into a computer program. This method may serve as an aid to, but not a substitute for, the judgment of a competent engineering geologist or tunnel engineer.

### 6.8.8 Numerical Modelling

Numerical modelling can be a powerful tool in the design of cavern structures.

Situations where numerical methods are required cover, but are not limited to, the following:

(a) caverns with large spans,

(b) caverns constructed under a thin rock cover, in mixed ground conditions or intersected by weakness zones,

(c) caverns of complex geometry,

(d) multiple underground openings in the vicinity,

(e) irregular pillars or pillar ribs or intersection,

(f) presence of external imposed loads (e.g. foundation loads of existing structures) or other asymmetrical loading acting on the cavern, and

(g) staged excavations.

Numerical models in cavern engineering are classified as continuum and discontinuum types. For continuum models based on the boundary element method, free surfaces are divided into elements and the interior of a rock mass is treated as an infinite continuum. Stresses and strains applied to an element have a calculable effect on other surface elements and throughout the medium. Thus changes at one surface element will affect all other elements. The boundary element method has the advantage that only boundaries have to be divided into elements, and far-field stresses are not influenced by the creation of an excavation. Although discontinuities can be modelled by means of the displacement discontinuity approach, the boundary element method is not suitable for problems requiring explicit consideration of several joints or sophisticated modelling of discontinuity behaviour. Also, in general, this method is not capable of incorporating variable material properties or modelling interaction between rock and support. Other numerical methods are more suitable for problems involving these considerations.

Continuum models based on the finite element and finite difference methods relate the conditions at nodal points to the state within elements. The physical problem is modelled numerically by discretising the problem region. These methods have the advantage of being able to handle material heterogeneity and non-linearity, but they handle infinite boundaries poorly. Discontinuities can be represented explicitly by means of specific discontinuity elements or by modifying the overall material properties to implicitly incorporate pseudo-discontinuities.

Continuum models are suitable when a rock is heavily jointed and appears to have soil-like characteristics relative to the size of an excavation. In a similar manner where a rock mass is massive, then continuum models may be appropriate, as the intact rock properties tend to dominate the response of the ground to an excavation. A number of continuum models allow the incorporation of discontinuities and are more appropriate for medium-scale rock engineering projects including cavern developments.

Discontinuum models simulate jointed rock mass as a series of blocks, each of which is considered a unique free body and can be discretised into deformable zones. The blocks can rotate, separate and slide according to Newton's second law of motion. These types of numerical model are particularly appropriate for medium-scale, nonlinear problems in rock mechanics (e.g. an analysis of the rock mass surrounding a cavern) and where rock mass stability is dominated by discontinuities. They are useful for understanding stress changes and calculation of the displacements of rock blocks subject to changes in stress around an excavation. In addition, they can serve as a tool for examining rock bolt lengths and support requirements for large span excavations where empirical support charts are not sufficient. Local case histories of using discontinuum models are reported by Bandis et al (2011). The number of joint sets and their persistence should not be overly pessimistically modelled in a discontinuum model (see also Section 6.6.2).

Different computer models may have respective strengths and weaknesses making them suitable for different geotechnical design problems. The results of numerical modelling are only indicative of typical expected rock mass behaviour and must be subject to proper interpretation before they are incorporated into a design.

With respect to input parameters for numerical analysis, a particular limiting factor is to represent the heterogeneous nature of a rock mass realistically in a model. Although this means that numerical models will not yield exact solutions, the simulated results will form a rational basis for examining the adequacy of the empirical methods for support design. Also, numerical models provide a means of sensitivity analyses by which the relative impact of different parameters can be assessed.

Starfield & Cundall (1988), Barbour & Krahn (2004), Hoek (2007) and Hoek et al (2008) discussed the methodologies for modelling rock mechanics problems and recommended good practice of numerical modelling. Salient points pertaining to the use of numerical modelling methods are given below:

(a) An appropriate initial state of stress in a rock mass should be established in numerical models.

(b) Realistic rock mass parameters should be used.

(c) The amount of stress relaxation that takes place prior to the installation of support may be estimated using the guidance of Hoek et al (2008).

Numerical analyses should model correctly the sequence of construction and, if needed, stressed zones around support elements. Other potential areas of application of numerical modelling include thermal analyses, analyses of cavern installations subject to vibrations, analyses of pressurised bulkheads, etc. Numerical modelling can also facilitate design of rock support or rock reinforcement with consideration of an allowable cavern convergence.

## 6.9 Design of Support Elements

### 6.9.1 General

Support elements for an excavation designed based on empirical, analytical or numerical methods should be selected with consideration given to the geological conditions inferred by pre-excavation probing (if any), results of geological mapping of the excavated face, observation of the stability of the rock mass and monitoring data. Where necessary, timely and appropriate adjustments should be made to the empirical design of the excavation support, based on judgement augmented by analytical or numerical analysis.

The most common method of roof support used internationally for large caverns in fair to good qualities rock masses is systematic bolting combined with shotcreting as permanent support. In addition to rock mass conditions, selection of permanent support should fulfil serviceability and durability requirements. The amount of temporary support depends on rock mass stability and method of construction. Temporary and permanent supports may be combined on the basis of cost-effectiveness considerations (see also Section 6.8.3).

A design carried out using well-established empirical methods is deemed to provide an adequate factor of safety provided that the methods are applied carefully within their intended scope. The required safety margin allowed for spot rock bolts and rock anchors are presented in Sections 6.9.2 and 6.9.3. For design of other support elements (except for prescriptive measures, e.g. spiles), design forces in support elements should be established using a rational method, including analytical solutions and numerical modelling, and all designs should fulfil the requirements of relevant structural codes (e.g. BD, 2011b & 2013; BSI, 2004, 2005a, 2005b & 2008).

### 6.9.2 Rock Bolts

(1) *General.* Rock bolting, the most common method for rock support, is convenient and flexible to use. Rock bolts may be used for temporary support at working faces and for permanent support. Their common length ranges from 2 to 6 m.

In rock tunnelling and cavern construction, the term “bolt” normally refers to both tensioned bolts and fully grouted untensioned bolts. The mode of action is dominantly in tension in both cases. In Hong Kong practice, manually tensioned bolts are usually not used, mainly to avoid the more onerous installation issues and bolt specifications and to reduce the requirement for maintenance (i.e. re-tensioning).

Stress is induced in untensioned bolts when movement occurs along the discontinuity being stabilised. In a jointed rock mass, dilation occurs as movement along a discontinuity causes riding-up on the asperities. Provided that the bonded length is adequate on either side of the discontinuity, tension and shear are induced in the bolt. The magnitude of these forces is dependent on the roughness of the discontinuity, properties of infillings (if any), the orientation of the bolt to the discontinuity and the relative movement necessary to develop peak shear strength and peak dilation. A simplified design method is given in Bjurstrom (1974).

Rock bolts, serving as roof and wall supports, are normally applied in two ways: spot bolting to secure isolated loose blocks, and systematic pattern bolting to achieve a general increase in stability (Figures 6.21 and 6.22).

(2) *Spot Bolting*. Design of spot bolting is usually carried out as an excavation proceeds. Spot bolting is used to secure individual blocks of rock. The size of a block can be estimated from the observation of the discontinuities that define the block. The bolts should be long enough to obtain adequate anchorage in a stable rock mass beyond the block. Block sizes are estimated from discontinuity directions and spacings relative to an excavation. The length of bolts should reflect the uncertainty of the block size estimate. The bolts should be grouted or bonded in a stable rock mass. The factor that governs the length *d* (see Figure 6.21) is primarily the rock mass quality. Normally, *d* should not be less than 1.5 m. A minimum bolt diameter of 20 mm is recommended. Spot bolting used as an initial support near an excavation face is usually determined on the basis of experience as the time available precludes any analysis. Some bolt types are pre-tensioned to activate their anchorages and to ensure they are effective. The pre-stressing of the rock is essentially incidental.

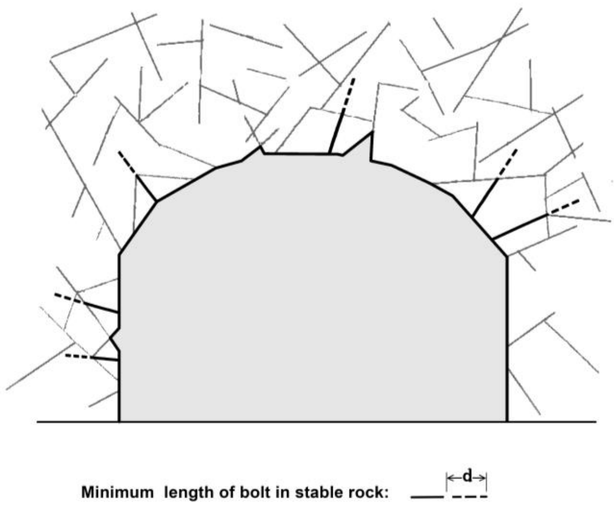



**Figure 6.21 Spot-bolting of Isolated Blocks**

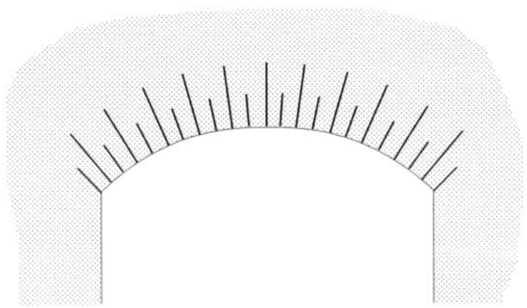


**Figure 6.22 Typical Roof Support with Pattern Bolting**

Where a kinematic permissible failure mechanism can be identified and properly defined, a limit equilibrium analysis taking into account the weight of rock block, cleft pressure and frictional resistance along rock joints should be carried out to determine the design working load of bolts with a global factor of safety of 1.5 against such a failure mechanism (see also Section 6.8.4).

For bolts grouted into a rock mass, in the absence of a detailed investigation, a presumed value of allowable rock-grout bond strength of 0.35 MPa can be used for determining the pullout capacity if the bolt is socketed into a partially weathered rock mass of PW90/100 or better rock zone. A higher value of design bond strength may be assumed if it is justified by the designer through a detailed ground investigation, site specific testing or an appropriate analysis. A minimum factor of safety of 2.0 on the ultimate rock-grout bond strength against pullout is recommended (see also Section 7.5).

For a grouted rock bolt which comprises a reinforcement, the allowable tensile strength of the reinforcement and the allowable bond strength between the grout and the reinforcement should be established with a minimum factor of safety 2.0 on the ultimate strengths. For proprietary rock bolts, design against internal failures should follow the suppliers' guidance.

Factors of safety lower than those given above on the capacity of a rock bolt (i.e. factors of safety on the ultimate rock-grout bond strength, reinforcement tensile strength and grout-reinforcement bond strength) may be applied for bolts that are used only as support during the construction stage when conditions are considered appropriate. In many recent local railway projects, a factor of safety of 1.6 has been adopted.

(3) *Systematic Bolting.* Systematic bolting is used to achieve a general increase in stability by holding a rock mass together and allowing it to form a natural arch. Rock bolts installed as part of a systematic bolting are usually not subject to the design methods applied to stabilisation of rock slopes, as sliding failure mechanisms cannot be defined. Bolt spacings and lengths for caverns are designed using empirical design rules (i.e. Q-system or RMR system) with review and modification as necessary on the basis of analytical solutions or numerical analysis. The empirical design rules are based on the use of high yield steel bolts. The most common diameter is 20 mm but larger diameters are also employed.

Bolts are normally installed in a pattern but with necessary adjustments based on the actual ground conditions observed on-site. Review of the specified bolt spacing and length should be undertaken on-site. The observed block size is the principal factor to be considered. A rule of thumb is that bolt spacing should be half the length. More rigorously, rock bolt spacing for caverns can be estimated using the Q-system, RMR and RMi classifications, provided that the classification systems employed are applicable to the design scenario. Bolt spacing of 1 to 3 m is adopted in most situations depending on the rock mass quality. Bolt spacing of less than 1 m is not normally considered practicable, and alternatives such as straps and shotcrete should be considered in conjunction with bolting.

The following formula proposed by Barton et al (1977) should be used for an initial assessment of rock bolt length in Hong Kong. In the equation, the length of rock bolts, $L$, can be estimated from the excavation width, $B$, and the excavation support ratio (ESR) as follows (for wall support, $B$ should be replaced by the cavern height, $H$, in the equation):

$$L = 2 + \frac{0.15B}{\mathrm{ESR}} \quad \text{.................................................} \quad (6.12)$$

where $L$ = bolt length (in m)
$B$ = cavern span for roof support (use cavern height, $H$, for wall support) (in m)
ESR = excavation support ratio, representing the safety requirement for the use of the cavern space (Barton & Grimstad, 2014; NGI, 2015).

Bolt length should take into account block size. Shorter bolts may be used towards cavern walls, but should not be less than 2 m. Figure 6.22 shows a typical pattern for roof stabilisation in section. In some cases, it may be cost-effective to use two lengths of bolt alternately.

Rock bolts placed systematically are in general located normal to the theoretical excavation line. Occasionally a case may be made for angling bolts to take into account discontinuity directions, but the engineer must take into account the added complication in installation and control. Design review using numerical or analytical method should be carried out.

Rock bolt design for major zones of instability created by seams or persistent smooth discontinuities should be established using numerical analysis.

(4) *Durability*. Rock bolts should meet the durability requirements (Sections 6.3.3 and 7.5.1). Appropriate corrosion protection measures should be implemented.

The effects of excavation on installed bolts should be considered in the design of permanent bolts. The possibility of loosening due to excavation and damage by blast vibration should be assessed. If the effects are deemed to be critical to the performance of the bolts, appropriate monitoring and replacement (if necessary) should be recommended.

### 6.9.3 Rock Anchors

The use of rock anchors (also known as cable bolts) may be required in special cases, such as for permanent stabilisation of caverns with large spans or where rock mass quality is poor. Anchors are most commonly used to address unexpected stability problems. The approach for design against pullout is the same as that for rock bolts (see Section 6.9.2).

Typical lengths of anchors vary between 10 and 30 m. The anchors may have grouted fixed lengths or may be fixed at their far ends. An advantage of rock anchors is that they can be pre-stressed to a high tension which is not the case for rock bolts. Permanent anchors should have adequate corrosion protection and long-term monitoring should be provided in accordance with Geospec 1 (GCO, 1989).

### 6.9.4 Spiling

Spiling (or forepoling) is a means of reinforcing a rock mass ahead of an excavation face with long bolts (spiles or forepoles). This support system is a prescriptive measure particularly applicable to poor quality rock masses, such as weakness zones, to restrict deformation of surrounding rock and to prevent rockfalls following excavation.

The design of spiles requires the determination of the required length and size of spiles, installation angle, spacing between spiles and overlapping length. Practical considerations for the construction of spiles are given in Section 7.5.3.

### 6.9.5 Canopy Tubes

Canopy tubes are adopted as pre-support to enhance roof and face stability during excavation in poor ground and normally consist of perforated steel pipes (75-170 mm) that can be joined together and drilled into the ground in the longitudinal direction of an excavation. To reduce overbreak, they are installed at shallow angles, as a single row of tubes, although sometimes a double row in a staggered pattern is adopted.

Canopy tubes are normally used in conjunction with lattice arches or steel ribs. They act as beams bridging unsupported ground in the longitudinal direction between a transverse support (e.g. lining, steel ribs, or lattice arches) and an excavation face or between two transverse supports. The design of canopy tubes requires the determination of the required length, size and stiffness of tubes, installation angle, spacing between tubes, overlapping length, and grouting pressure (Shin et al, 2008). The vertical pressure acting on a support system can be estimated by methods suggested by John & Mattle (2002). Equation 6.13, developed based on the silo theory by John & Mattle (2002), has been used for canopy tubes design in a number of underground excavation projects. The equation was developed for a single soil layer and conditions that soil arching within a silo can be established. Numerical modelling should be carried out to validate a design where needed.

$$p_1 = \left(\gamma - \frac{2c'}{R_m}\right)\frac{R_m}{2\lambda\tan\phi'}\left(1 - e^{-2\lambda\tan\phi'\frac{H}{R_m}}\right) + p\, e^{-\lambda\tan\phi'\frac{H}{R_m}} \quad \text{.................} \quad (6.13)$$

where $p_1$ = silo pressure (in kPa) at depth, $z$ (in m)
$\gamma$ = unit weight of soil (in kN/m$^3$)
$c'$ = cohesion of soil (in kPa)
$\lambda$ = horizontal pressure coefficient, can be assumed to be 1 - sin $\phi'$
$\phi'$ = friction angle of the soil (in °)
$p$ = surface surcharge (in kPa)
$H$ = overburden (in m)
$R_m$ = mean radius of the silo (in m) (see Equation 6.14).

$$R_m = 0.5 \times \sqrt{w \times s} \quad \text{................................................} \quad (6.14)$$

where $w$ = width of the tunnel (or tunnel diameter)
$s$ = $n$ × unsupported length with $n$ = 1.5 for soft ground.

Silo pressure calculated can be used to establish the design structural forces of the canopy tubes with consideration of the advance length and spacing of the transverse supports.

### 6.9.6 Steel Ribs and Lattice Arches

Steel ribs (or ribs and lagging) or lattice arches in conjunction with shotcrete are used as immediate support measures where poor ground conditions are encountered. Steel ribs are usually made of straight or bent I- or H-beams, bolted together to form a circular or pitched arch with vertical side supports (legs). Lattice arches (or lattice girders) are lightweight support members comprising steel reinforcement bars that are usually laced together, typically in a triangular configuration, serving similar function as steel ribs.

Shotcrete is used to fill the space between steel ribs/lattice arches and an excavation face to ensure good contact. Alternatively, blocking which consists of concrete blocks or steel shim spacers may be installed to fill the gaps.

Structural forces in steel ribs should be estimated in accordance with the method proposed by Proctor & White (1946) or a suitable numerical analysis which explicitly considers rock-support interaction. In a numerical analysis, the whole roof support system (i.e. the steel ribs or the lattice arches) and the vertical side support (legs) should be considered. Appropriate restraint conditions at the leg foundations should be specified in the analysis. Where significant side pressures acting on the legs are anticipated, adequate lateral restraints (e.g. invert struts) to the legs must be provided to prevent kicking in. The leg foundations should be duly designed to take into account the presence of weak materials, if any.

Steel ribs and lattice arches which are designed to fulfil the fire resistance and durability requirements can be applied as permanent support.

### 6.9.7 Shotcrete Support and Reinforced Ribs of Shotcrete

Shotcrete creates a semi-stiff lining on excavated faces. The shotcrete should have adequate shear and moment carrying capacity to prevent collapse of excavated surfaces. Reinforcement can be provided to the shotcrete to enhance its structural capacity.

Reinforced ribs of shotcrete (RRS) can be usefully employed in adverse rock mass conditions, e.g. support categories 6 to 8 in the Q-system. Zare (2007) showed with numerical modelling that RRS can be more economical compared to traditional cast in-situ concrete in ground with a Q-value between 0.001 and 1. The design of RRS for support should follow the recommendations of NGI (2015) or Barton & Grimstad (2014) (see also Appendix D).

Analytical design of shotcrete should be undertaken where needed. The design should check against the failure modes presented in Figure 6.23 to meet the requirements of relevant structural codes, where considered appropriate. A description of this analytical approach which considers the potential failure modes shown in Figure 6.23 is given by Barrett & McCreath (1995). Recent research by Bryne et al (2014) provided data on early-age shotcrete bond strength in hard rock. These publications suggested that typical values for such early-age shotcrete bond strength range from 0.5 MPa to 1 MPa.

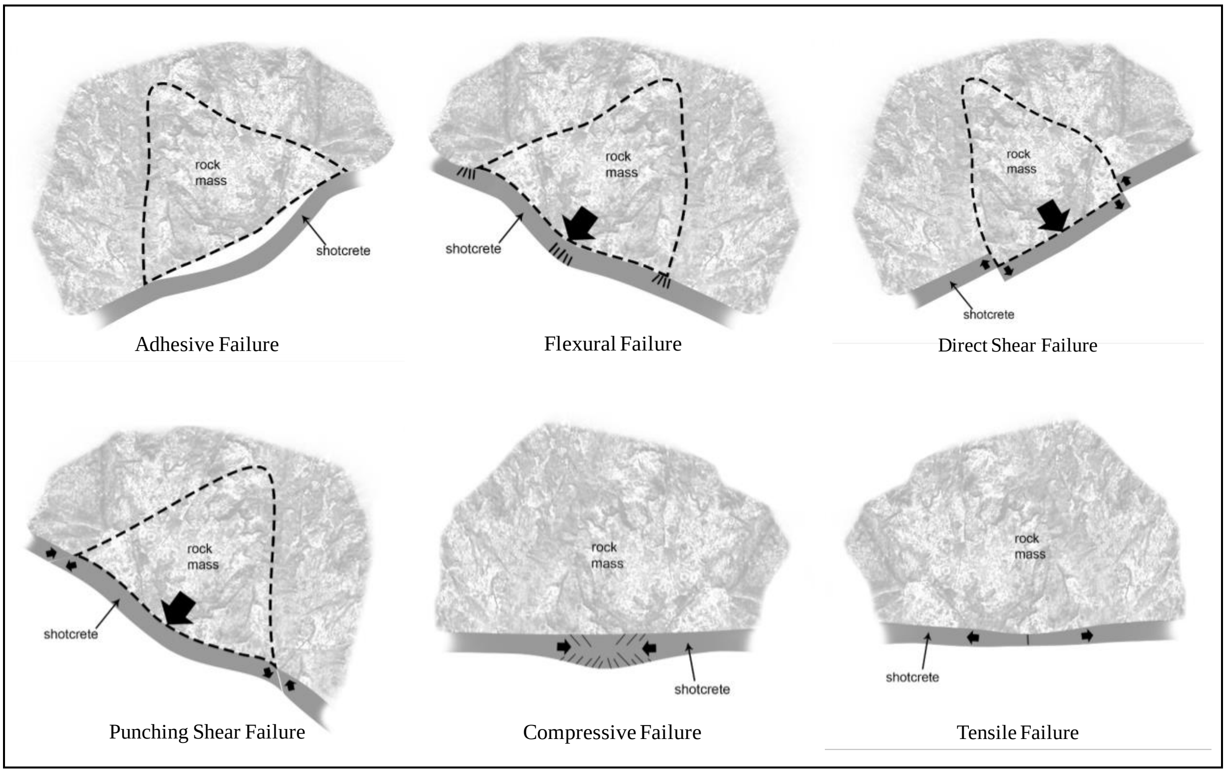



**Figure 6.23 Potential Failure Modes of Shotcrete**

Structural design of fibre-reinforced shotcrete (for temporary and permanent linings) should follow well-established design codes including the German Concrete Association design guidelines (DBV, 1992), International Federation for Structural Concrete Model Code 2010 (IFSC, 2012), Technical Report No. 63 by the Concrete Society (CS, 2007a), Technical Report No. 65 by the Concrete Society (CS, 2007b) and RILEM (2003), or other appropriate design codes/standards. An example of typical design is given by Lo et al (2009).

### 6.9.8 Lining and Lining Support

(1) *Permanent Lining*. The decision for the provision of a permanent lining and the common options for final selection of the type of lining structure to a cavern should be determined with consideration given to the following factors:

(a) serviceability requirements,

(b) geological and hydrogeological conditions,

(c) durability requirements,

(d) occupancy,

(e) future maintenance cost, and

(f) economy.

A watertight lining may be required for the sake of providing a dry environment for the particular usage of a cavern. On the other hand, a watertight lining may be needed for permeable fractured rock zones to avoid undue ground settlement induced by groundwater drawdown caused by uncontrolled groundwater inflow, but it may not be required for other sections of a cavern. For caverns with high occupancy, unlined caverns may not be suitable, depending on the rock mass conditions. If necessary, a nominal layer of shotcrete with a minimum thickness of 50 mm over an exposed crown and shoulders, as a prescriptive measure to prevent loosening and fall of small rock fragments endangering occupants, should be applied for safety and operational reasons. Such measure is only appropriate where there is no groundwater ingress into the cavern.

The need to provide a permanent pre-cast or cast-in-place concrete lining (which can be expensive) over an initial shotcrete (if applied) should be assessed carefully. With suitable rock mass conditions and groundwater control and initial rock support/reinforcement works, it is possible and more cost effective to provide, using high pressure jetting, a one-pass or multi-pass shotcrete lining structure for a cavern. In such designs, the groundwater control and initial support/reinforcement works as well as the lining structure can be designed and constructed to permanent works standards, incorporating suitable detailing to prevent residual groundwater inflow from entering the cavern space which could affect cavern usage and present problems for maintenance. Guidance on design of shotcrete lining are given in Section 6.9.7.

(2) *Lining Structural Analysis.* If a permanent lining is considered necessary, the rock mass pressure acting on the permanent lining should be determined from the Q-system as proposed by Grimstad & Barton (1993) (Equations 6.15 and 6.16) or using a rational design method:

$$P_{\text{roof}} = \frac{200\,Q^{-1/3}}{J_r} \quad \text{.................................................} \quad (6.15)$$

where $P_{\text{roof}}$ = estimated rock pressure at the crown (in kPa) and the rest of the parameters are defined in Section 6.8.6(2).

This equation is applicable for $J_n > 9$ (three joint sets or more). In cases where the number of joint sets falls below three ($J_n \leq 9$), the degree of freedom for block movement is greatly reduced and the equation below should be adopted:

$$P_{\text{roof}} = \frac{200\,J_n^{1/2}\,Q^{-1/3}}{3J_r} \quad \text{.............................................} \quad (6.16)$$

The parameters in this equation are defined above and in Section 6.8.6(2). It should be noted that the equations are interchangeable for $J_n = 9$.

The rock pressure acting on the tunnel walls, $P_{\text{wall}}$, can also be calculated using Equations 6.15 and 6.16. The Q-value in the two equations should be substituted by a hypothetical "wall quality" value, as follows:

(a) $5Q$ for $Q > 10$,

(b) $2.5Q$ for $0.1 < Q \leq 10$, and

(c) $1Q$ for $Q \leq 0.1$.

The Q-system method is not applicable when external imposed loads or potentially unstable rock wedges are present. In such cases, a structural frame analysis should be carried out for the relevant load scenarios. Common sources of loading have been discussed in Section 6.8.2.

(3) *Design Groundwater Pressure and Pressure Relief System.* Linings may be designed as drained or undrained structures. An undrained lining is designed to exclude groundwater ingress and to withstand the full in-situ groundwater pressure, whereas a drained lining allows groundwater to ingress into the underground space in a controlled manner with a permanent pressure relief system circumferencing the excavation. The majority of caverns in Hong Kong and around the world are designed as drained structures with an emphasis on groundwater inflow control such as pre-excavation grouting during construction to ensure minimal impact of residual groundwater inflow on the surrounding groundwater regime and sensitive receivers (see also Section 6.10). Undrained linings can be expensive, and are usually adopted where the long-term impact of a cavern development on the surrounding groundwater regime and sensitive receivers is considered significant.

For a permanent drained lining, a prescriptive minimum design groundwater head of 10 m above the crown of a cavern has been adopted in Hong Kong for design robustness in light of the concern that drainage systems may not function as effectively as assumed in the design throughout the design life of the structure. The prescriptive design groundwater pressure acting on the lining increases linearly from the crown down to the springline level to account for the elevation difference in hydrostatic pressure and then reduces linearly from the springline level to zero at the invert level. This has been adopted where a pressure relief system is provided. In the design, locations for access holes for maintenances of the drainage system to cater for residual groundwater inflow should be provided. Where the design detailing can ensure adequate drainage (with redundancy) and effective maintenance of the drainage system, a rational approach instead of a prescriptive approach may be adopted for the assessment of water pressures for the lining design.

(4) *Fire Resistance*. All permanent support structures should have a fire resistance which meets the requirements of the relevant code of practice (e.g. BD, 2011a).

## 6.10 Hydrogeological Impact Assessment

### 6.10.1 General

A detailed assessment of the hydrogeological setting and the impacts of the cavern development should be carried out as part of the design of any cavern development. The choice to select drained or undrained underground excavation design will mostly depend on the hydrogeological assessment and the impacts that any significant drop in groundwater level would have to the surrounding area and sensitive receivers (see also Section 6.9.8 on drained and undrained lining design).

The potential drawdown of the groundwater level and alteration of hydrogeological regime should be evaluated for both the short-term scenario (such as during construction) as well as the long-term scenario (for the operation stage).

Attention should be drawn to situations where an excavation is carried out in a zone with high hydraulic connectivity. Initial inflows are usually much higher due to the reservoir effect of storage of groundwater within discontinuity networks. The ability of the network to replenish itself depends on its connectivity to surface infiltration and nearby sources of water. Seasonal differences in recharge may also impact inflows into caverns depending on how close they are to the ground surface and whether there is direct connection to water-bearing fractured or weathered rocks. Control of groundwater inflow by probing ahead and by pre-grouting is essential to ensure limited or negligible impact to the surrounding groundwater regime.

For the operation stage, the majority of rock excavations in Hong Kong have had only limited inflows, given the groundwater control measures that have been implemented and the generally low hydraulic conductivity of the rock mass. Data available from four Mass Transit Railway (MTR) tunnels, recorded during the operation stage of the tunnels (i.e. after the grouting and the completion of the permanent lining), indicate that typical groundwater inflow rates are generally low.

Good ground investigation and monitoring is required to develop a better understanding of a hydrogeological regime to demonstrate minimal impact on the surrounding environment.

### 6.10.2 Effects on Sensitive Receivers

Water inflows into caverns may lead to a reduction in the piezometric head in the ground strata above. This will cause an increase in effective stress and induce ground movements. Ground settlement should be estimated based on effective stress and soil mechanics principles, with due consideration of the known lowest groundwater level (see Section 6.6.3). Effects of differential settlement and long-term consolidation should be assessed as necessary. Sensitive receivers include structures, foundations and utility services within the affected zone. Attention should be drawn to the possibility of negative skin friction and additional load transfer to deep foundations. Depression of water level may also lead to loss of yield and other potentially adverse effects on the groundwater resources and the environment (e.g. sustainability of vegetation).

Major groundwater inrush may impose a significant hazard to the personnel in a cavern during construction and impact stability of the excavation. The amount of groundwater inflow during the operation phase should also be controlled such that the serviceability requirements of the facility constructed can be met.

### 6.10.3 Groundwater Modelling

Groundwater modelling should be carried out to assess the effect of groundwater inflow when the effects on groundwater regime can be deleterious to sensitive receivers or where groundwater table has to be maintained at a certain level for confinement of materials stored within a cavern, such as with unlined oil and gas storage caverns. It is useful in (i) gauging the adequacy of the allowable groundwater inflow limits to be specified for a project, and (ii) identifying the possible influence zones, as well as assessing the potential ground settlement and movements and associated impacts. Guidance on the determination of groundwater inflow is given in CIRIA Report No. 515 (Preene, 2000) which also discusses the different methods for design of dewatering systems. Another useful reference is Publication No. 12 of the Norwegian Tunnelling Society (NFF, 2004) which covers groundwater control in tunnelling. Todd & Mays (2005) provided further guidance on groundwater analysis. The key procedures for groundwater modelling are summarised below:

(a) Determine the purpose of a model and later develop a conceptual model with appropriate governing equations and computer codes.

(b) Calibrate the model so that it can reproduce field data. Subsequently, determine the effect of uncertainty on the calibrated model.

(c) Verify the model by using the calibrated parameters to reproduce a second set of field data.

(d) Predict and quantify the response of the system to future events and also assess the effect of parameter uncertainty.

(e) Perform post-analyses with new field data and remodel if necessary.

The model should at least cover both the transient scenario during construction and the long-term scenario throughout the design life of a cavern.

For groundwater modelling using numerical solutions, the computation domain should be extended beyond the anticipated influence zone of groundwater drawdown. Since the input parameters that are able to completely represent the ground and groundwater conditions may not able to be practically obtained, the results of numerical modelling should be checked by sensitivity analysis with support of engineering judgement.

It is a good practice to benchmark the results of numerical analysis against those from analytical solutions to provide insight into the reasonableness of the numerical results. Commonly adopted analytical solutions include the modified Goodman's method by Heuer (1995 & 2005) and Raymer (2001).

The rate of groundwater inflow into an excavation is a time-dependant function that may vary as the hydrogeological regime adjusts to take account of the excavation. The

Goodman method in particular assumes no reduction in head and therefore represents an upper-bound assessment for anything other than immediate and early flow rates.

Measuring inflow from probe holes ahead of a cavern/tunnel excavation allows an assessment to be made for inflow into the tunnels and caverns. Lo & Cheuk (2006) discussed the use of water inflows in horizontal directional coring (HDC) as potential indicators of future inflows in tunnel excavations (see also Section 5.7.3).

### 6.10.4 Monitoring and Control of Groundwater Inflow

(1) *Monitoring*. A monitoring plan and performance review to verify an assumed groundwater inflow and alteration of a hydrogeological regime should form an integral part of a design. Parameters to be monitored should comprise inflow rates into the underground openings, and the groundwater level and piezometric heads in potentially affected ground strata. Ground surface settlement, movements of sensitive receivers and subsurface movements should also be included in the monitoring survey. The adequacy of the monitoring plan, groundwater control measures, and assumptions made in the groundwater model and the ground movement calculations should be reviewed regularly based on the monitoring data. Chapter 9 recommends good practice in developing a monitoring plan.

Should post-construction performance monitoring be required, monitoring points and instrumentation (e.g. sump pits, piezometers, V-notch and real-time monitoring system) should be included in the cavern design.

(2) *Control of Groundwater Inflow*. Based on the experience of tunnelling within the igneous rock masses in Hong Kong, it is clear that inflows vary according to rock mass conditions, in-situ stresses, weathering state and proximity to recharge sources. Site-specific conditions are very influential on the response of the hydrogeological regime to an underground excavation.

NFF (2004) serves as a reference for design of groundwater control measures. On the basis of this experience, inflows are typically low or extremely low, for a large majority of underground excavations in hard rock and systematic grouting is not typically required for the majority of these excavations. Higher potential of substantial groundwater inflows is typically associated with discrete transmissive features such as faults or the influence of the weathering profile.

Hong Kong experience suggests that inflows to excavations in rock are controllable by combining systematic probing with well-executed pre-excavation grouting. Post-excavation grouting, on the other hand, has proven difficult and not effective.

The aim of pre-excavation grouting is to seal off transmissive features such as joints and fissures in a rock mass by placing grout screens along and surrounding the tunnel or cavern, to stop or reduce water ingress before excavation. Sufficient probing and an appropriate groundwater inflow limit to trigger the pre-excavation grouting should be specified. During excavation, the adequacy of probing (including the number of probe holes and the length of holes) and the appropriateness of the triggering limit should be reviewed regularly. Guidance on design and execution of pre-excavation grouting is given in AGS(HK) (2013). Modern pre-excavation grouting (PEG) for rock tunnels can offer substantial time saving, as well as much improved results in terms of groundwater exclusion during tunnel excavation, when compared with traditional grouting technology. It is an empirical and observational design approach in which the details and the grouting criteria are designed based on observation, measurement and experience. Full benefit requires the use of stable micro cement grout with a low viscosity, dual-stop pumping criteria and relatively high pumping pressure for grout injection. Details of the execution need to be adjusted according to the site conditions and the actual results achieved, compared to the targeted residual groundwater ingress.

Where long-term groundwater pressure reduction or depression of groundwater level is predicted to exceed the acceptance criteria, the adoption of an undrained lining should be considered. Acceptance criteria should be assessed on a site-specific basis with consideration taken of the potential impacts on sensitive receivers.

Waterproofing is normally incorporated in the design of underground structures in Hong Kong. The main benefits are the prevention of groundwater ingress into an underground space and the consequential reduction of humidity. Although grouting is the primary means of groundwater control, it may not exclude groundwater ingress sufficiently for all cavern uses. Other methods of groundwater control may have to be introduced. The principal methods include:

(a) drainage at a rock face behind the shotcrete,

(b) watertight concrete lining,

(c) drip screens, and

(d) impermeable membranes.

The choice of method depends on the amount of groundwater ingress, the requirements dictated by the cavern use and the cost of the various options. Water and fuel stores, in general, do not have any requirements with respect to waterproofing beyond the control afforded by grouting, but this may not apply to treated water reservoirs. For most cavern uses, dripping into a cavern should be controlled to avoid damage to water-sensitive goods and equipment, and to minimise nuisance. Drainage installed behind the shotcrete should be considered. Drip screens have considerable advantage in that they are cheap, can be quickly erected and are less dependent on good workmanship. Drip screens can form a pleasing architectural finish and also prevent any minor loose rock fragments from falling into a cavern. They can be fitted after equipment has been installed.

# 7 Cavern Construction

## 7.1 Introduction

### 7.1.1 Purpose and Scope

This chapter describes the methods of cavern construction and advises on the good practice for the various processes involved. It also incorporates the experience which has been gained from the caverns constructed in Hong Kong.

The division between cavern design and construction is not sharp. Cavern construction technology governs many important aspects of design, and the design verification and revision process continues to the end of the construction phase. Thus, the division between this chapter and the preceding one is somewhat arbitrary.

In view of the especially close link between design and construction for tunnels and caverns, it is even more important than in other engineering projects that construction is supervised and controlled by qualified and experienced personnel with sound knowledge of the design process (see also Section 6.9.1).

### 7.1.2 Hong Kong Experience

A number of caverns have been excavated in Hong Kong for purposes including MTR stations, sewage treatment, refuse transfer, salt water reservoirs and explosives storage. Examples are provided in Appendix A.

Majority of the caverns have been constructed using drill and blast techniques and have employed rock mass mapping during the construction to confirm the level of rock support needed. Those caverns were constructed within jointed rock mass, sometimes intercepted by intruded dykes and other geological features, such as faults and shear zones, which led to various technical and construction-related issues including groundwater control.

There have been various advances in technology over the years since the first few caverns were constructed in the 1980s, with significant improvements in drilling, blasting, excavation and rock support technology.

In construction, decisions need to be made on when to split excavations into various headings and benches. For larger span caverns, multiple headings and benches may also be involved. Detailed planning and consideration given to the sequencing of an excavation to match the observed and estimated rock mass support, as well as the logistics of temporary access ramps to gain access to upper headings and mucking out, are needed.

### 7.1.3 Methods of Excavation

Drill and blast methods dominate the construction of underground space in rock. In the last few decades, the basic methods have not changed, but improvements in equipment and blasting technology have resulted in increased production rates, lower costs and enhanced ability to control blast-induced vibration and other adverse effects. The drill and blast methods are expected to remain viable for tunnelling for years to come, and will remain the only cost-effective method of forming large caverns in hard rock.

The rock blasting of caverns can involve:

(a) face blasting with horizontal drillholes for tunnelling or top heading excavation,

(b) benching with horizontal drillholes for side benches to provide good control of the excavation profile along the walls, and

(c) benching with vertical drillholes for central benches.

All three methods are commonly employed in cavern construction.

Non-blasting methods, including hydraulic drill and splitting, have been used for cavern excavation in Hong Kong (Figure 7.1). The use of non-blasting methods may allow excavation to be carried out 24 hours a day due to the minimal effects at the surface. Nevertheless, the drill and split method is slow and will only be economic when blasting is not practicable. It can be a viable alternative for smaller caverns where the ground conditions are favourable, e.g. in weak rock.

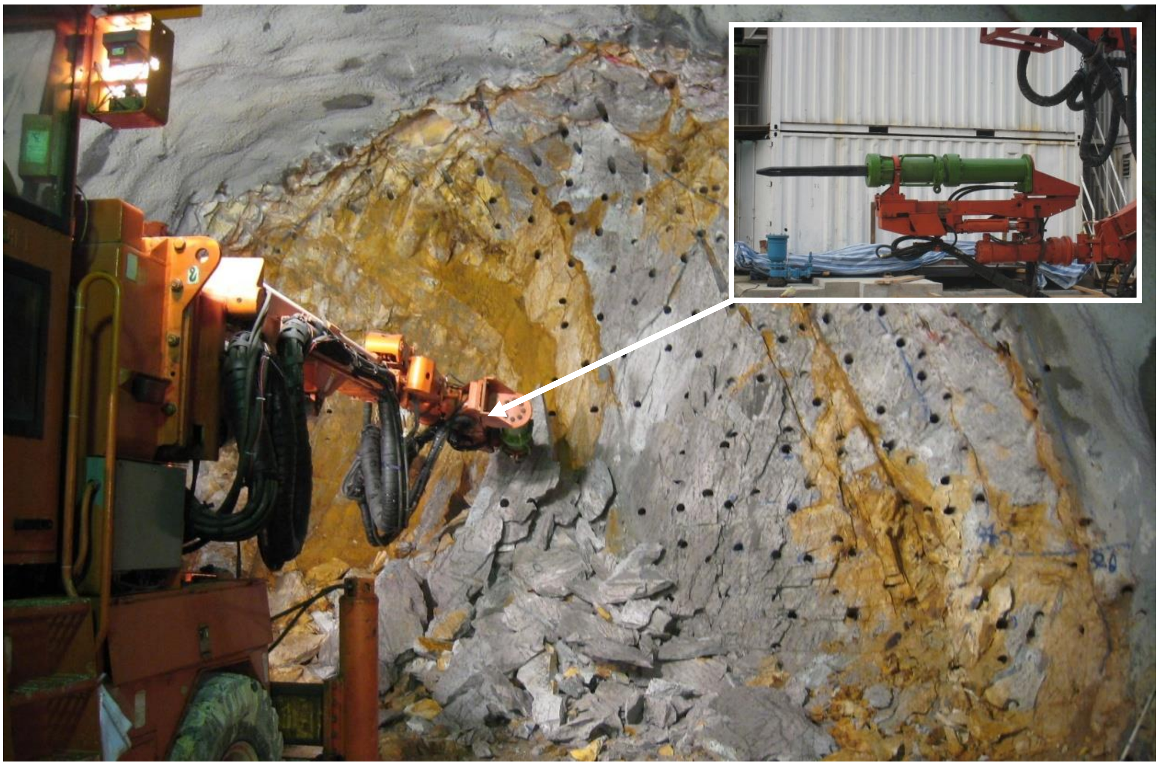


**Figure 7.1 Drill and Split Method Using a Hydraulic Splitter**

### 7.1.4 Working Cycle

Excavation by drill and blast is carried out in a series of cycles, each of which consists of:

(a) probe drilling,

(b) pre-grouting of rock mass (if required),

(c) drilling of blast holes,

(d) charging with explosives,

(e) detonation,

(f) ventilation,

(g) scaling,

(h) mucking-out, and

(i) temporary support works after rock mapping and design verification.

The cycle is essentially the same for tunnels, cavern top headings and benching. Probing ahead of the rock face in top headings and benches should be carried out to determine geological and hydrogeological conditions (see Section 7.7.1). However, if long probe holes are drilled, then it is not necessary for probing to be carried out at every excavation cycle. In some ground conditions, scaling, mucking-out and temporary support may be completed in an order other than that shown above.

## 7.2 Risk Management

### 7.2.1 General Considerations

Risk management, including risk assessments, plays an important role in the planning, design and construction of cavern developments and should be fully integrated into the project management processes. The Project Administration Handbook for Civil Engineering Works (HKSARG, 2016) by the HKSAR Government covers the general guidance for public works projects. One of the aims of risk management is to ensure that works do not adversely affect safety, environment or existing infrastructure. Therefore, risk assessments should be conducted at the start of every project to assess the risks, particularly geotechnical, with respect to health and safety, environment, cost, time and reputation. The risk management process should continue throughout a project, with all risk assessments, health and safety plans, method statements, etc. being maintained and updated regularly.

GEO (2005) provided guidelines pertinent to geotechnical risk management specific to tunnelling projects. The guidelines should be followed throughout the project life (planning, design and construction) of all cavern developments in Hong Kong. A risk register should be prepared along with mitigation and site supervision plans. It also provides examples of geotechnical hazards and construction method-related risks in tunnel construction. The use of observational method, including its benefits in identifying and designing for the most probable ground conditions (i.e. the design conditions most likely to occur in practice), is discussed. The method requires robust supervision and monitoring procedures. In addition, contingencies for unfavourable and unexpected ground conditions should be planned, and material and plant for implementation of planned contingency actions should be made available on site for prompt mobilisation. If there is insufficient time to implement a contingency design or emergency plan, the method is not recommended.

Clayton (2001) and Palmström & Stille (2010) provided useful general guidance on geotechnical uncertainties and risks, and sources of risk to a cavern development. Further guidance is provided in A Code of Practice for Risk Management of Tunnel Works (ITIG, 2012). It sets out the best practice for identification of risks, their allocation between the parties to a contract and contract insurers. In addition, it covers the management of risks through the use of risk assessments and risk registers throughout a project. For example, prior to construction, the contractor should develop method statements and working approaches that manage and target the known risks that have been identified through the earlier risk identification process. Appropriate equipment and construction methods to minimise the risks and to ensure future performance of the cavern and compliance with the design should be selected.

The GEO maintains a catalogue of notable tunnel failure case histories covering both local and overseas cases. The catalogue serves as a useful reference in the identification of geotechnical risks and effective risk management measures at the early stage of a project involving tunnel works.

## 7.3 Planning the Excavation

### 7.3.1 Equipment

Typical equipment used for cavern construction is described below.

Computerised drilling jumbos are capable of drilling up to 2 m per minute in rock with uniaxial compressive strength up to 160 MPa. Theoretically, up to 200 $m^2$ faces can be drilled from a single setup with a maximum drilling height of 13 m, although this is rarely achievable in practice in order to ensure safe and optimal working conditions. The computerised jumbo can automatically align the drilling for the next round, survey the excavated profile and log the drilling parameters for each drill hole.

Mucking-out is performed by wheel-loaders, backhoes, and trucks. Diesel or electric (as opposed to petrol) loading equipment is used to improve the environment at a tunnel face. A mobile crusher can be installed underground close to a face and connected to a conveying system, reducing the number of trucks significantly and thereby improving the air quality. The capacity of a crusher, if located underground, affects the construction logistics (i.e. the crusher capacity would control the mucking out rate) and choice of other plant and equipment. Additional cavern space may need to be created to house the crusher system.

Shotcrete is placed using computerised robots which can spray up to a height of 14 m, whilst drilling for rock bolts is usually carried out by a drilling jumbo. However, for a large cavern development, a dedicated rock-bolting machine may be preferred, especially where there are a large number of rock bolts. Scaling is normally performed by hydraulic hammers mounted on excavators, but scaling by a competent person using a handheld bar for small underground openings is still used sometimes.

Rock mass grouting is carried out using computerised units that can mix, agitate and deliver grout to several grout holes simultaneously using pre-determined termination criteria for intake volume, injection pressure or duration. Advances in grouting technology now allow for higher penetration into a rock mass, creating a drier tunnel environment (NFF, 2011).

### 7.3.2 Access Tunnels

Access to the top heading level and to the cavern bottom level will be required for a cavern excavation to proceed. Other construction access may be required for efficient construction. The design of an access tunnel system must include an optimisation of the tunnels needed for permanent use and temporary tunnels needed for construction.

The location of the access to a cavern excavation depends on the requirements for an efficient excavation, permanent access requirements, geological constraints and access for pre-excavation support. For short caverns, construction access to one end of a cavern that allows excavation from a single face may be satisfactory. With long caverns, this arrangement may lead to unacceptably long construction periods. A point of access entering near the mid-point of the top heading excavation may be required to allow two faces to be worked simultaneously in opposite directions.

Account should be taken of any cost and time advantages that may accrue from working multiple faces.

Large cavern schemes with long and high caverns require special access arrangements. One option is to excavate an inclined transportation tunnel between main caverns down to the bottom level of the caverns. Access to different bench levels may then be achieved through short side tunnels from the transportation tunnel. A tunnel system arrangement varies with the cavern use, but convenient access to different excavation levels in high caverns must be achieved to allow low cost excavation and high production rate. Figure 7.2 shows an example of such arrangement.

Efficient mucking-out from large multi-cavern schemes may require the provision of double-lane access tunnels with a maximum inclination of 1:7 as one of the options. Some contractors may prefer a gentler gradient. It may be prudent to allow for this in a design if the penalties of so doing are modest. Paving of access tunnels can be cost-effective because it reduces tyre wear and increases driving speed.

Section 8.2 provides further guidance on the design and construction of adits and tunnels.

**Figure 7.2 Cavern Access by Dedicated Transportation Tunnel**

### 7.3.3 Top Heading

The top heading of a cavern excavation should normally be excavated first using tunnelling techniques. This gives easy access to the cavern roof for installing support works. The secured roof gives safe working conditions for the excavation of the lower levels of the cavern. The lower levels may be excavated using quarrying techniques, i.e. benching, which are cheaper than tunnelling.

The size of the top heading of a cavern excavation is governed by several factors which commonly require the top heading to be divided into two or more sections. These factors are:

(a) dimensions of the cavern,

(b) accessibility to the cavern,

(c) sequence of construction,

(d) tunnel equipment size and availability,

(e) the area of unsupported roof that can be exposed at any one time, which is primarily a function of rock quality,

(f) the presence of weak rock which may limit the area of the unsupported excavation face because of instability,

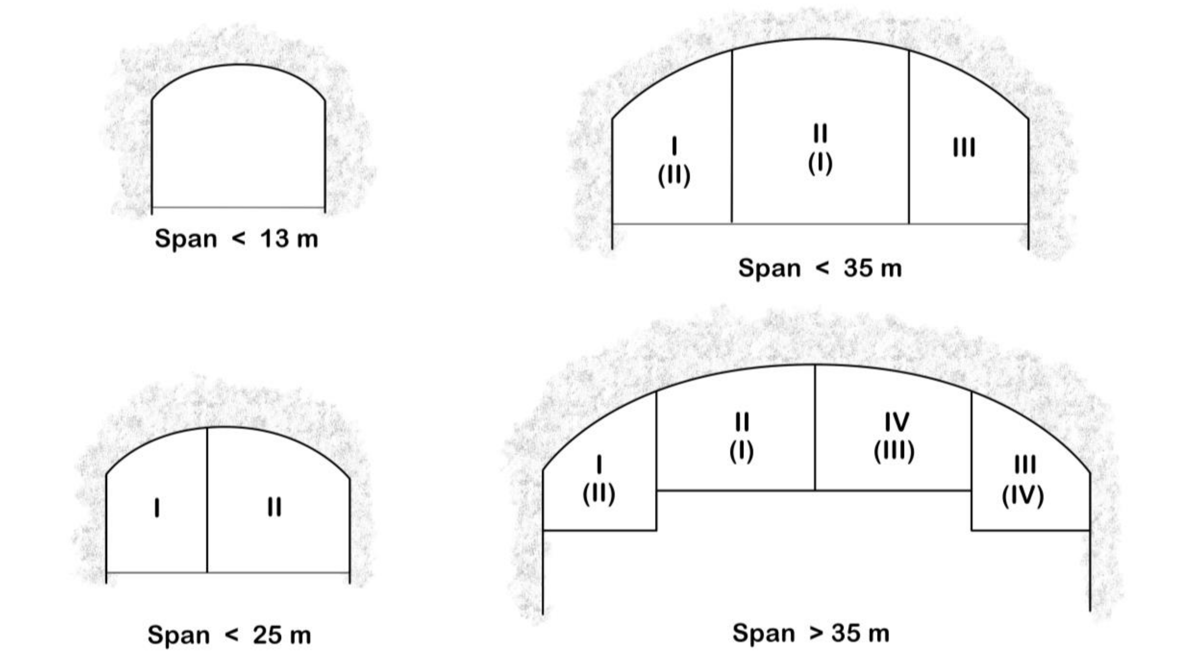
(g) limitations on the maximum instantaneous charge (MIC) given by blast vibration acceptance criteria, and

(h) practical depth of blast holes, which is up to 6 m depending on the cross-sectional area of the top heading and the length of boom on the jumbo.

It is normally economical to excavate a face as large as possible (up to 200 $m^2$), but the above factors may limit the size of the top heading to some 100-120 $m^2$.

In poor rock conditions, the initial top heading of a cavern excavation may need to be substantially smaller than 100 $m^2$ and should then be used to examine the ground conditions. Detailed excavation planning, design of rock support and other rock treatment works should then be done before the full span of the cavern is exposed.

The number of sections of the top heading depends on the span of the cavern and the maximum practical size of heading. Figure 7.3 shows typical excavation stages for a top heading excavation in hard rock conditions for different ranges of cavern span.




Notes: (1) The dimensions and sequence shown are for reference only.
(2) The actual dimensions and sequence should be determined on a case-by-case basis.

**Figure 7.3 Typical Excavation Stages for Top Headings in Caverns**

The order of driving the sections depends on the rock discontinuity orientations and the need for support. The sections should be driven in the order that avoids or minimises support of rock that will be removed by excavation of subsequent sections. The side of the cavern roof that is expected to have the least favourable stability should be driven first. Where no such stability and support problems are expected, the centre heading may be driven first.

There are various options for the timing of the excavation of the second and subsequent sections of a top heading. The principal options available are either to drive one section first to completion, followed by the excavation of the subsequent stages, or to allow each section to lead subsequent sections by a small distance. The selection of an option is dependent on the effective operation of the drill and blast cycle, including mucking-out and support. Where extensive rock support is required, it may be advantageous to separate the sections horizontally by two blast rounds.

### 7.3.4 Benching

Bench excavation is cheap because the large free surfaces allow the use of quarrying principles rather than tunnelling technology, reducing drilling and explosive costs. Support costs for benching are low because the roof has already been supported, leaving only some wall support to be done.

Production rates of bench excavation can be high and are predominantly driven by the excavation method and hauling system employed in a cavern project. Bench excavation may be carried out with vertical drillholes as in a quarrying operation, or with horizontal drillholes as in tunnelling using drilling jumbos, as mentioned in Section 7.1.3. It is preferable that vertical holes are sunk with crawler-mounted quarrying rigs as they are purpose-built for vertical drilling, although modern drilling jumbos with high production rates have increased the competitiveness of horizontal drilling compared to vertical drilling. Caverns of limited height preclude vertical drilling because of lack of headroom near the walls.

A cavern excavation should be divided into benches of a suitable height. Figure 7.4 shows typical bench excavation stages for a high cavern. The height of the benches may be determined from the consideration of the following:

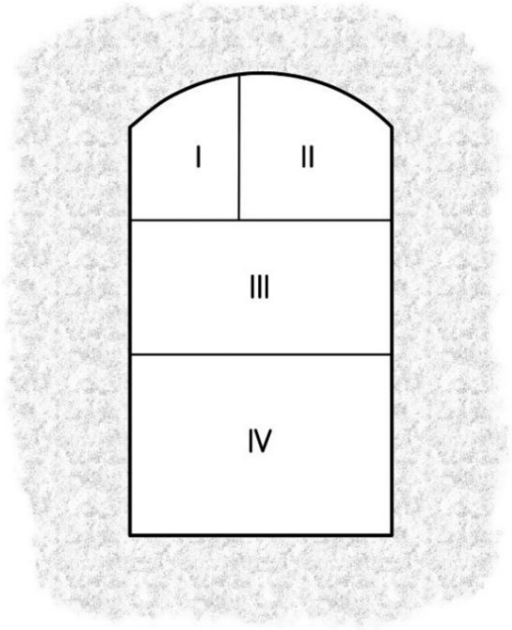
(a) *Access*. Mucking-out must be through tunnels located at suitable levels.

(b) *Blast-hole deviation*. The longer the holes, the greater the deviation which has to be matched to desired tolerances; deviations of vertical drillholes are typically 2° and peripheral horizontal holes are commonly splayed at 3-5°.

(c) *Reach of jumbo*. Bench heights with horizontal holes are limited to the reach of a drilling jumbo, which can be up to 13 m. However, as bench heights can be up to 10 m, the reach of the drilling jumbo should not generally be a limitation, and this is not a constraint for vertical holes.

(d) *Headroom for drill mast*. The size of a top heading should be designed to take into account the length of drill rig masts. This is an issue for both jumbo and quarry rigs, as rigs with short masts can increase drilling times significantly due to the increased number of rod changes.

(e) *Stability of walls*. In some conditions, walls may be unstable if too high, such that successive bench excavations and support are required.




Note: Roman numerals indicate the excavation sequence.

**Figure 7.4 Typical Excavation Stages for a Large Cavern**

(f) *Cavern use*. Some uses, such as unlined fluid stores, do not require optimisation with respect to shape; only volume is important, and caverns are optimised primarily with respect to the geological setting and efficient construction.

(g) *Cost-effective construction*. This calls for an optimisation of the excavation cycle. Relevant factors, e.g. availability of construction plant, should be duty considered.

## 7.4 Top Heading and Bench Excavation

### 7.4.1 Drilling Blast Holes

Blast holes are drilled using jumbos with hydraulic percussion drills. Figure 7.5 shows a modern drilling jumbo. Rates of drilling are of the order of 2 m per minute for standard drills in granitic and volcanic rocks and can be higher in weaker rocks. Drillholes are normally between 45 and 51 mm in diameter, with the latter being the most common. Peripheral drillholes are splayed out by approximately 3-5° relative to the excavation axis to allow space for the next round. Length of drillholes is commonly 4 to 6 m and is dictated by the overall optimisation of the operation. Blast patterns will likely require modification as an excavation progresses to accommodate changing excavation geometries and ground conditions.

Guidance on drilling pattern design is given in Section 7.6.6.

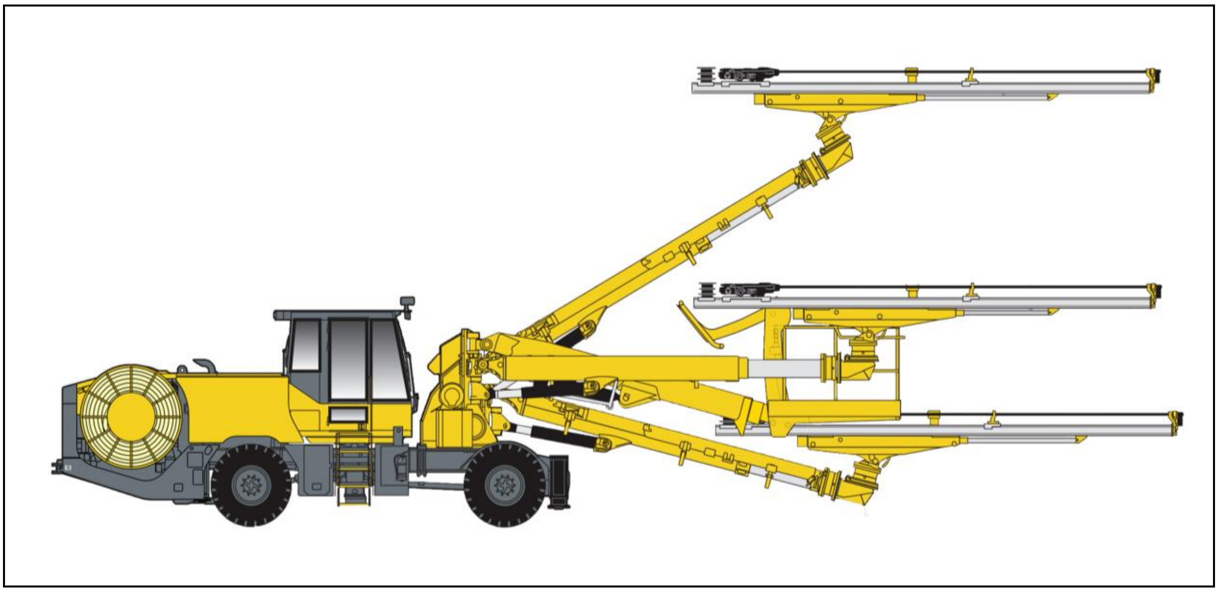


**Figure 7.5 Typical Modern Drilling Jumbo**

Bench excavation drillholes can be either horizontal or vertical. Normally vertical drillholes are preferred as the drilling of the holes is then independent of other sections of the excavation cycle except during blasting and ventilation. However, horizontal drilling may be required if a clean floor is necessary.

In deciding on the bench height to be adopted, many factors have to be taken into account, in particular, the need to have easy access to the side walls for support works. Drillhole lengths up to 18 m or more can be achieved, but drill deviations need to be taken into consideration. Vertical holes are normally drilled beyond the desired floor level to ensure that good fragmentation is achieved, but this can produce excessive overbreak in the floor. Drillhole diameters are commonly in the range of 51 to 76 mm.

### 7.4.2 Control of Overbreak

To control overbreak, and where smooth cavern crown and walls are desired, the smooth-blasting method should be used when blasting top or bottom benches. Further guidance on this is given in Section 7.6.6. It is recognised that there is an inherent construction overbreak that should be considered during the construction of perimeter blast holes due to the need for look-out angles to allow jumbo and drilling equipment to stay within the construction line. The use of computerised jumbos, well-maintained equipment and experienced drillers should be able to reduce this impact. Long pull lengths for a blast will also result in more construction overbreak.

Uncontrolled overbreak can be the result of a combination of poor blast control and geological conditions (see Section 6.4.4). It is often difficult to determine the main cause. Drilling controls, including the use of surveyed computerised drill jumbos and competent drillers, generally help in the formation of consistent drill patterns. Geological conditions, however, can still have an effect even on the best drilled profile. Guidance on drilling control is given in Sections 7.4.1 and 7.6.6.

In zones of weakness, such as in a highly or adversely jointed rock mass and faults, pre-excavation support such as spiles and forepoling can be beneficial, as they help to support the ground and reduce unravelling. Pre-excavation grouting to control water inflows can also be effective. Further guidance is given in Section 7.7.4.

### 7.4.3 Explosives and Charging

(1) *Permits*. All handling and use of explosives must be in accordance with the Dangerous Goods Ordinance (HKSARG, 1997). The contractor is required to apply for a Licence to Possess Category 1 Dangerous Goods and a Permit to Use Category 1 Dangerous Goods under the Ordinance. Protective and precautionary measures, in terms of prevention of flyrock, vibration limits or maximum instantaneous charge (MIC), control of excessive air overpressure and dust, etc., must be duly considered in a blasting assessment (Section 7.6). Details of the requirements can be found on the CEDD Website (http://www.cedd.gov.hk).

(2) *Detonators*. Electric detonators are initiated by an electric current. In Hong Kong, they are typically used for initiating a blast and have seldom been used for initiating individual blast holes. This is because there are safety issues related to the possibility of stray currents pre-maturely triggering electric detonators, as well as the fact that there are more effective blasting systems available for underground rock excavation.

Non-electric type detonating systems, which are initiated by a shock tube, are commonly used to initiate blast holes in Hong Kong. Manufacturers of such systems produce a wide range of surface and down the hole detonators, with accurate fixed time delays, which are sufficient for most blast designs. The systems are also safer than electric detonators. Although the non-electric detonating system can be initiated by non-electric means, an electric detonator is still the preferred method of initiation due to lower costs. Non-electric initiation should be used when blasting near to high voltage cables.

Electronic detonators have a highly accurate programmable timing system that enables a wide variety of possible delay timing in a blast round, and allows better prediction and control of the blast induced vibrations. The cost of these detonators is considerably higher than non-electric detonators, but where vibrations/charge weights must be minimised to avoid excessive vibration, the use of electronic detonators can allow controlled blasting in sensitive areas. However, they can suffer from damage to wires under normal blasting conditions which can result in current leakage problems. Where the blast area is wet, extra care must be taken to ensure that current leakage is kept to a minimum to avoid the situation where the blast cannot be properly fired, with uninitiated explosives left in the blast holes. To reduce the chance of this occurring, the integrity of the blast circuit has to be checked continually when the blast is being loaded and before the blast is ready to fire. It is expected that the use of electronic detonators will become more economically viable as costs reduce and blast designs in urban areas become more complex.

It should be noted that the safety fuse initiation system is no longer used in Hong Kong.

Multi-deck blasting involves the separate initiation of independent charges within a single blast hole, without simultaneously releasing the energy from the entire column of explosives within the charged hole. Electronic detonators can facilitate the use of multi-deck blasting to achieve higher production rates with less vibration. Non-electric detonators can also be used for multi-deck blasting, but there is less choice in timing delays (which may be needed for blasts involving a large number of detonators in double or triple decks) and there is less control on the precise separation of the blast propagating waves. However, when multi-deck blasting is employed, attention must be paid to issues including the identification and management of misfires and, with respect to electronic detonators, how to deal with detonators that cannot be read or fired by the blast initiation equipment.

(3) *Explosives*. Low-cost bulk explosives based on ammonium nitrate prills mixed on site with fuel oil (i.e. ammonium nitrate fuel oil, ANFO) or site-sensitised emulsion (i.e. bulk emulsion) are used mostly for a main production blasting.

ANFO has also been successfully used in underground excavations where there is little or no groundwater present. ANFO is primed with a cast booster or cartridge explosives and is loaded into a hole using a compressed air kettle, which ensures adequate compaction to allow detonation. However, the use of ANFO where water is present can result in desensitisation and, where water is present or where oxygen is deficient, can lead to the production of toxic gas.

Site-sensitised explosives (SSE), which are pumpable emulsions, are not explosives until mixed with a gassing agent and pumped into a hole. Also, ammonium nitrate does not become an explosive until it is mixed with fuel oil. However, it is essential that the explosive components are safely managed (Johansen & Mathiesen, 2000). For safe transport, storage and security of these explosive precursors (blasting agents), reference should be made to the requirements relating to the manufacture, storage, conveyance and packing of Category 7 Dangerous Goods under Part VIII of the Dangerous Goods (General) Regulations (HKSARG, 1997), and advice from FSD should be sought on their requirements.

Modern emulsion explosives are oxygen-balanced, producing a minimum of noxious fumes and far less smoke (Olofsson, 2002). Therefore, they provide better working conditions, require less ventilation and improve re-entry times. Emulsion explosives are also water resistant and can be used when water is a problem and ANFO cannot be used. The emulsion can be pumped into blast holes by a computer controlled bulk emulsion pumping unit. The pump accuracy is around ± 5%, with a minimum deliverable quantity varying between 0.1 and 1 kg, depending on the size and type of pump used. This accurate control is particularly useful for the charging of perimeter holes and the row adjacent to the contour holes, where less explosives are used to allow smooth wall blasting (Zare & Bruland, 2007). The measuring system may also help to control the uncharged length of blast holes for optimum blasting results.

High-energy explosives in the form of cartridge emulsion or cast boosters, as primers, are generally used to initiate bulk emulsion for production blasting. Detonating cords are typically used in contour holes to reduce the total energy released and to minimise overbreak and secondary cracking.

(4) *Explosive Delivery and Site Storage*. To ensure safety and security of handling explosives, on-site storage of explosives is generally not permitted. Explosives are generally delivered to a works site from a Government depot once a day, at a time dictated by the delivery operation of the Mines Division of the GEO. This can pose a significant constraint on blasting frequency. Hence, there are circumstances where provision of an explosives site magazine (which has to be a store licensed by the authorities) can be greatly beneficial, as this would allow blasting to be carried out more than once a day and at times that best suit the site operations. Where it is cost effective to do so and the hazard to life requirements under the EIA can be met, the project proponent or its consultants should identify an acceptable site for a magazine at an early stage of the project. If an acceptable site cannot be found, the works contract should state the explosives delivery arrangement and frequency, as well as any blasting constraints present, and these should be taken into account in the contract programme. As a good practice, explosives delivery to a site should be confirmed only when all the blast holes are ready for charging.

(5) *Charging*. Charging holes at height should be done by hand from a hydraulic platform, either on a jumbo or an independent piece of equipment. Charging of explosives should only commence after the drilling of all holes has been completed maintaining a safe environment for charging of the blast holes. After checking the holes for blockages, the holes are bottom primed. Non-electric detonators are placed following the blast pattern, whilst electronic detonators are placed in the holes either in a pre-programmed state or to be programmed later. Non-ferrous tamping rods should be used to place the detonator and booster into the back of the hole. Then the main explosive is charged (cartridges, bulk emulsion or ANFO) followed by stemming, normally in the form of 10 mm aggregate. The detonators are then connected (or programmed) to form a controlled blasting system. Detonation must be done remotely after clearance of workers and equipment. Contour holes are normally lightly charged to prevent excessive overbreak (smooth wall blasting). The large relief holes in the centre of a blast pattern are left uncharged to act as a free face. Further guidance on blast design is given in Section 7.6.6.

When charging holes in a bench the same procedures as above are employed, with extra care taken to clean the holes, as it is easier for foreign objects to block them.

### 7.4.4 Scaling

After blasting and ventilation, the roof and sidewalls should be scaled. Scaling should be done as soon as practically possible because failure of loose blocks can be potentially fatal. This is normally done by excavators with hydraulic breakers. Handheld scaling bars are normally only used in small access tunnels. Hydraulic breakers are very powerful and care must be taken not to remove too much rock.

The contractor is responsible for the safety of the works and must have the experience to carry out the scaling adequately. It is therefore essential that an engineering geologist reviews the ground conditions with the tunnel engineer or experienced tunneller who will then supervise the scaling operations. The engineering geologist should carry out subsequent checks of any areas of instability or loosening of the rock mass that are identified during scaling.

In some rock conditions, particularly where the rock is easily loosened in a scaling operation, scaling may be limited or dispensed with entirely and shotcrete applied to lock any loose material in place. Caution should be exercised in these locations as the detachment of shotcrete support poses a significant risk.

It should be noted that scaling can take a substantial part of the time for a full work cycle. Even in good rock, this can amount to three hours or more for a large top heading. Scaling may have to be carried out at regular intervals after excavation and until permanent rock support is installed. No scaling should be carried out and workers should be evacuated during underground blasting nearby to avoid the risk of detachment of loose rock.

### 7.4.5 Mucking-out

Diesel-powered rubber-tyred loading and transportation equipment is normally used for large tunnel and cavern top heading excavations. Front-end loaders or load-haul-dump (LHD) loaders in combination with 30- to 50-tonne off-road trucks provide large mucking-out capacities.

Alternatively, electric powered face shovels can be used for excavating the spoil. The use of electric powered equipment underground is beneficial to the tunnel environment as it reduces the pollutants generated by diesel engines.

The use of crushers and conveyors underground is an alternative way to remove the excavated rock in large volumes. In addition, this reduces the diesel emissions from large diesel powered trucks. A crusher is required to ensure the rock is broken into sizes suitable for the conveyor.

The type and the number of plant items need to be optimised with respect to cost and the construction programme of an excavation. In urban areas, smaller road vehicles may be required which may have consequences for the planning of an excavation and choice of plant due to the increased time needed to muck-out.

Appropriate mucking out procedures should be employed to safeguard against an accidental initiation of unexploded explosives that may remain in a muck pile due to a misfire. This will require vigilance initially from the shot firer when checking the blast and later the loader operator when digging out the rock. Appropriate contingency measures and an action plan should be agreed in advance for misfire management that identifies the likely hazards, suitable precautionary and protective measures and lines of communication that would be needed in the event of a misfire.

### 7.4.6 Rock Support

Engineering geological mapping of discontinuities and rock mass conditions should be carried out after an initial scaling to determine the temporary rock support required. Sometimes preliminary rock support may have already been installed to provide a safe working space for the mapping. Experienced engineering geologists should be employed in the mapping to identify any persistent sub-horizontal discontinuities or zones of weakness that may affect ground stability and thus requiring additional rock support.

Rock support is required to secure the working space and should be installed as soon as possible. Therefore, the installation of rock support may govern the rate of excavation. Temporary rock support is installed after an initial scaling and normally consists of rock bolts and shotcrete. The amount and the type of temporary support are governed by the condition of a rock mass exposed by a blast and the final support requirements, with due consideration of the construction sequence.

Permanent rock support should normally be installed when an excavation has progressed sufficiently so the installation works do not adversely affect overall progress of the excavation. Section 7.5 describes common ground support methods in detail. Guidelines on the design of support elements are given in Section 6.9.

To reduce the time spent by workers under unsupported ground, rock support should be installed where possible by machinery. It is now common practice to use robotic shotcrete machines, with the operators standing under supported ground. Currently rock bolts are generally installed by hand under unsupported ground. However, the use of rock bolting machines should be considered wherever possible. These machines are dedicated to the automatic drilling and installation of rock bolts and therefore remove the requirement for workers to be exposed to unsupported ground while installing rock bolts.

## 7.5 Common Ground Support

### 7.5.1 Rock Bolts

Rock bolting is the most common method for rock support. It is convenient and flexible to use. Rock bolts may be used both for temporary support at a working face and for permanent support. Lengths are commonly in the range of 2 to 6 m.

There have been advances in the automatic installation of rock reinforcement. Automatic rock bolting machines that can drill holes, insert reinforcements and insert resin cartridges/grout without the need for workers to work under unsupported ground can help improve safety.

Rock bolts may be used for both roof and wall support. The bolts are normally used in two ways: spot bolting to secure isolated loose blocks, and systematic pattern bolting to achieve a general increase in stability (Figures 6.20 and 6.21).

Bolts may also be employed to fix a variety of items to a rock during construction and for permanent use. Such items include ventilation ducting, lighting, cables and cable trays, shuttering for concrete pours, drip screens, concrete beams and floors. Bolts for permanent and temporary fixings should be designed to accommodate the loads to be carried and should have an adequate design life.

Bolts may be anchored at the far end and tensioned or may be fully grouted bolts without tensioning. Some proprietary bolt systems attain friction against a rock throughout their lengths by radial expansion of the bolt. The objective of pre-tensioning rock bolts is primarily to activate their anchorages and not to pre-stress the rock.

There are numerous types of bolts available, both for temporary and long-term support, and some bolt types are suited for both uses. The main types of bolts available are represented in Table 7.1 (Hung et al, 2009).

Rock bolts must be carefully specified and installed by trained personnel under close supervision. In particular, bolts for permanent support must be carefully specified. It is recommended that a design explicitly specifies the rock bolt type(s) that should be used (including details of face plates, bolt and hole diameters, corrosion protection, grout type and method, bond lengths, pre-stressing requirements (if any), etc.) (see Section 6.9.2). For reinforcement bolts, corrosion protection is provided by a combination of sacrificial metal (typically 4 mm on the diameter is considered prudent in Hong Kong) and by cement grout, with reference to Geospec 1 (GCO, 1989). In aggressive ground (e.g. submarine construction) special attention should be made to the potential of increased corrosion and in some circumstances, stainless steel bolts, glass-reinforced plastic (GRP) or double corrosion protection bolts/cables may be required. Bolts should be galvanised for corrosion protection, but the compatibility of the galvanisation and cement mortar should be checked as adverse reactions may occur (cements with low chromate content may destroy the zinc coating and severe corrosion may ensue).

Typically rock bolts are centralised in a hole with stabilisers (commonly plastic) and then fully encapsulated in grout, which is normally a cementitious product with a maximum grain size of 3 mm. Drillholes should be properly cleaned and the drillhole diameter should be sufficient to allow a grout pipe to be inserted and for the grout to fill the annulus around a bolt, providing a good bond between the rock and bolt (it is generally considered that a drillhole diameter 13 mm larger than the diameter of the rock bolt is sufficient). Grout samples should be taken and cubes formed to check the compressive strength of the grout. Various proprietary products are available that apply different approaches with different levels of corrosion protection and these may have different criteria for installation. It is important to consider the weight and methods of installation for permanent rock bolts, as some systems are difficult to install into the crown of a cavern as they are very heavy.

(1) *Pullout Test*. For the purpose of pullout test, bolts constructed for cavern support can be categorised as follows:

(a) systematic bolts designed using empirical methods,

(b) bolts designed on the basis of an allowable rock-grout bond strength $\leq 0.35$ MPa, and

(c) all other bolts.

In order to give an indication of the contractor's workmanship, the appropriateness of a construction method under specific ground and groundwater condition, and potential construction difficulties, pullout test should be carried out. For category (a) bolts, at least three initial pullout tests should be carried out at the commencement of a rock excavation. If the initial pullout tests for category (a) bolts are carried out satisfactorily, no additional initial pullout tests for category (b) bolts are required. For category (c) bolts, a separate set of at least three initial pullout tests should be carried out before installation.

**Table 7.1 Types of Rock Bolts (modified after Hung et al, 2009; Brady & Brown, 2004; Hoek, 2007; Gates, 2013) (Sheet 1 of 5)**

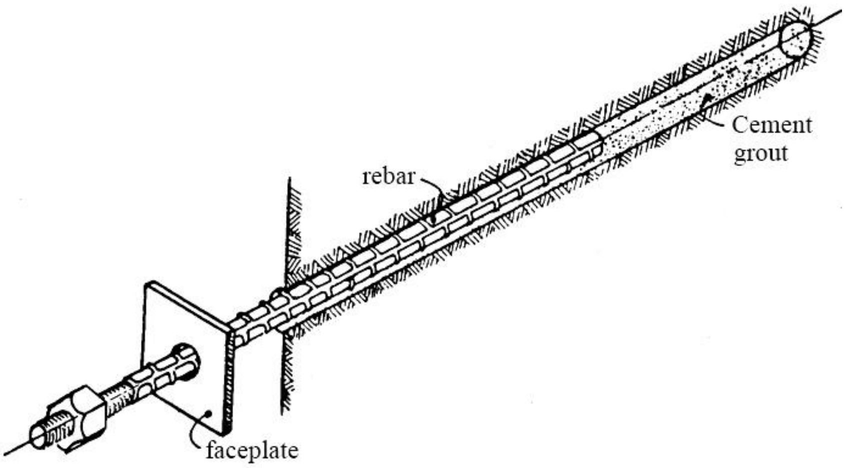
| Type | Description |
|---|---|
| Grouted rock bolt | *Pros*<br>• Materials are easy to obtain and inexpensive.<br>• Grout provides corrosion protection.<br>• It is simple to install.<br>• It performs well in weak, broken, poor quality rock and poorly drilled holes because the grout will flow into the voids.<br>*Cons*<br>• The grout setting time could be slow, i.e. lack of immediate support.<br>• Pumping grout, especially overhead, may require a particular procedure.<br>• A longer bond zone than other bolt types depending on the rock type may be required.<br>• Installation can be time consuming, requires two grouting periods for pre-stressed bolts (bond zone and unbonded zone).<br>• Grouting operation may tend to be more expensive than some other bolt types due to time and manpower.<br>• Installation cannot be automated.<br><br> |
| Resin-grouted rock bolt | *Key Features*<br>• Used in critical applications in which cost is less important than speed and reliability.<br>*Pros*<br>• It is very convenient and simple to use.<br>• It can form very high-strength anchors in rock of poor quality.<br>• A “one-shot” installation provides a fully grouted tensioned rock bolt system by choosing appropriate setting times.<br>• It is good for soft and hard rocks. |

**Table 7.1 Types of Rock Bolts (modified after Hung et al, 2009; Brady & Brown, 2004; Hoek, 2007; Gates, 2013) (Sheet 2 of 5)**

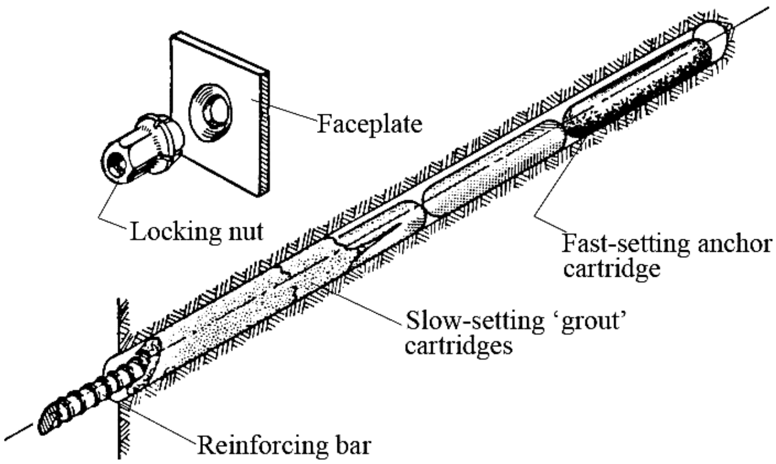
| Type | Description |
|---|---|
| Resin-grouted rock bolt (cont'd) | *Pros*<br>• It can withstand blasting vibrations.<br>• A shorter bond zone than cement-grouted bolt is required.<br>• The short curing time allows quick installation.<br>• Only one 'grout' cycle is required.<br>• The operation is relatively clean.<br>• It can be installed at any angle.<br>• It can be installed where mechanically anchored bolts are inappropriate.<br>*Cons*<br>• Resins are expensive.<br>• Shelf-life is limited, particularly in hot climates.<br>• Resin cartridges require specific dimensions of the drillhole, bolt diameter and resin cartridge diameter to achieve proper mixing and bonding.<br>• Resin cartridges can perform poorly in weak, broken, poor quality rock and poorly drilled holes because the tight tolerance does not allow for extra voids, which creates gaps and reduces the bond.<br>• Length of bolt is usually limited to around 12 m because of drag of the bar through the resin during spinning.<br>• Coupled bars typically do not work because the annular space required for the coupler results in exceeding the annular space required for the resin cartridge.<br>• Its use is constrained by temperature (sets slower in colder ambient temperatures).<br>• It is limited on tension capacity.<br>• In Hong Kong, use of resin requires long lead-in delivery times.<br><br> |

**Table 7.1 Types of Rock Bolts (modified after Hung et al, 2009; Brady & Brown, 2004; Hoek, 2007; Gates, 2013) (Sheet 3 of 5)**

| Type | Description |
|---|---|
| Expansion shell rock bolt | *Key Features*<br>• Immediate support is provided by pressurising the expansion shell, with optional grouting through the hole at the bolt plate.<br>• It is very widely used for permanent support applications in civil engineering.<br>• Mechanically anchored bolts without grout are widely used in mining.<br>*Pros*<br>• Installation is easy and speedy.<br>• Bolts can be tensioned immediately after installation and grouted at a later stage, providing corrosion protection, when short-term movements have ceased.<br>• Very reliable anchorage in good rock and high bolt loads can be achieved.<br>*Cons*<br>• It can be costly if only used as temporary support.<br>• Correct installation requires skilled workmen and close supervision.<br>• As these bolts are relatively stiff they may not be appropriate for weak or heavily-jointed rock and can fail where deformation in the rock mass is high.<br>• Grout tubes are frequently damaged during installation and a check by pumping clean water before grouting is essential. |
| Split set stabiliser | *Key Features*<br>• It is used for relatively light support duties in the mining industry, particularly where short-term support is required and rock burst conditions are mild.<br>• Slotted bolt is inserted into a slightly smaller diameter hole.<br>• Induced radial stress anchors the system in place by friction.<br>• It slips instead of suddenly failing.<br>*Pros*<br>• Simple and quick to install and claimed to be cheaper than a grouted dowel of similar capacity.<br>*Cons*<br>• It cannot be pre-tensioned and hence is activated by movement in the rock in the same way as a grouted dowel.<br>• Its support action is similar to that of an untensioned dowel and hence it must be installed very close to a face.<br>• The drillhole diameter is critical in order to establish an adequate bond between the bolt and the drillhole.<br>• In some applications, rusting has occurred very rapidly and has proved to be a problem where long-term support is required/limited life in highly corrosive environment.<br>• The device cannot be grouted. |

**Table 7.1 Types of Rock Bolts (modified after Hung et al, 2009; Brady & Brown, 2004; Hoek, 2007; Gates, 2013) (Sheet 4 of 5)**

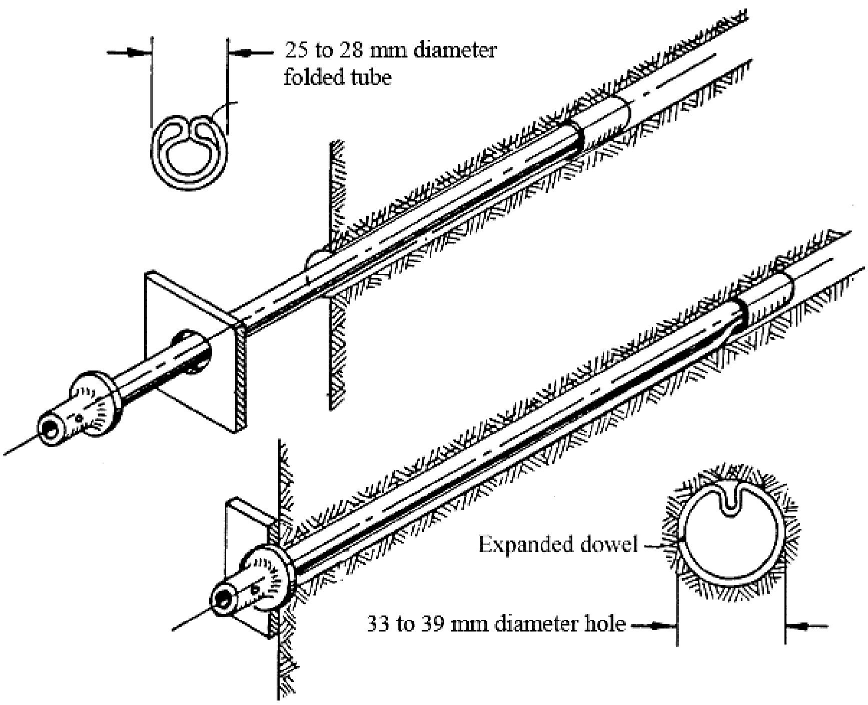
| Type | Description |
|---|---|
| Expandable rock bolt | *Key Features*<br>• Length up to 12 m.<br>• Inflation pressure ≈ 30 MPa.<br><br>*Pros*<br>• It is easy and very quick to install.<br>• Full load bearing capacity can be developed instantly.<br>• It is not sensitive to blasting.<br>• Elongation ranges: from 20 to 30%.<br>*Cons*<br>• Service life is limited in highly corrosive environment.<br>• It is generally not acceptable for permanent support for civil engineering applications.<br>• As per split set bolts, the drillhole diameter is important. |

**Table 7.1 Types of Rock Bolts (modified after Hung et al, 2009; Brady & Brown, 2004; Hoek, 2007; Gates, 2013) (Sheet 5 of 5)**

| Type | Description |
|---|---|
| Self-drilling anchor | *Key Features*<br>• It is generally used when support needs to be quickly installed in poor ground conditions.<br>*Pros*<br>• Drilling, installation, and injection can be completed in one single operational step.<br>• No pre-drilling of a drillhole by using a casing tube and extension rods with subsequent anchor installation is necessary.<br>• Less space is required for anchor installation.<br>*Cons*<br>• Optimised machinery and manpower is required.<br>• It is more costly when compared with other temporary support.<br>• It is not suitable for permanent support for civil engineering applications. |
| Cablebolt | *Key Features*<br>• It is primarily used to support large underground structures, i.e. mining applications, underground power caverns, etc.<br>• Cable capacity is confining stress dependent.<br>• At very high loads the governing parameter is often the bond between the tendon and the grout.<br>*Pros*<br>• It can handle high loads.<br>• Tendons are grouted with concrete mix or resin.<br>*Cons*<br>• It is expensive.<br>• Installation time is long.<br>• It requires skilled operatives.<br>• If resin is used the shelf life of the resin should be considered. |

Further pullout tests for the above three categories of bolts should be carried out where:

(a) the rock mass encountered is poorer than PW90/100, or the ground condition is substantially different from the prevailing ground condition at the locations of the initial tests, or

(b) ground conditions that could lead to poor quality rock bolts are encountered (e.g. presence of significant groundwater inflow or significant grout loss), or

(c) there is a genuine concern of workmanship, or

(d) a different type of rock bolt and/or rock bolt bonding is to be used (e.g. use of a different grout mix, different grouting method, or different expansion anchorage system).

(2) *Pullout Test Requirements*. Test rock bolts should be grouted or installed in conditions compatible with the design intent, and should not form part of the actual rock support system. They can be located at the sidewalls or invert of a cavern unless there is a genuine need for testing the bolts at other locations. It is recommended to carry out pull-out testing of rock bolts in accordance with Ulusay & Hudson (2007), “Suggested Methods for Rockbolt Testing”, modified as follows, or with the manufacturers’ recommendations, as appropriate:

(a) *Bond Length*. This should be 2 m or the design minimum bond length, whichever the lesser.

(b) *Test Load*. A test load shall correspond to twice the working load, but not exceeding 90% of the yield load of a rock bolt reinforcement or 70% of the ultimate load of a rock bolt if it is predicted that the rock bolt will exhibit brittle behaviour. The test load shall be maintained for 10 minutes for deformation measurement which may need to be extended to 60 minutes if continued rock bolt movements are detected. The measurement shall be taken at time intervals of 1, 3, 6 and 10 minutes, and at 20, 30, 40, 50 and 60 minutes for the extended test as described above. (For the purpose of initial pullout tests on systematic rock bolts designed using empirical methods, the working load should be established based on an assumed rock-grout bond strength of 0.35 MPa.)

(c) *Acceptance Criteria*. The test is considered acceptable if:

(i) the difference of rock bolt movements at 6 minutes and 10 minutes, or 10 minutes and 60 minutes if applicable, does not exceed 2 mm or 0.1% of the grouted length of the test rock bolt, and

(ii) the test rock bolt is able to sustain the test load, and the loss in stress in the rock bolt does not exceed 5% of the test load in 6 minutes.

For other types of rock bolts such as resin-grouted rock bolts, expandable rock bolts, etc., test load, test procedure and requirements should refer to the manufacturer’s specification or appropriate standard.

(3) *Proof Load Test*. Proof load tests on installed bolts have been specified in recent local railway projects. The location, frequency and test load of a proof load testing should

be undertaken in accordance with the contract requirements. Test load at an appropriate level should be adopted to avoid any damage to the installed bolts.

### 7.5.2 Rock Anchors

The use of rock anchors (also known as cable bolts) may be required in special cases, such as for permanent stabilisation of caverns with large spans or where rock mass conditions are poor. It is not usual for anchors to be part of the original design of caverns, as that would suggest poor rock conditions that would normally have been avoided because of the costs involved in achieving adequate stability. Anchors are most commonly used to treat unexpected stability problems.

Guidance and requirements in relation to construction issues such as testing, installation, maintenance and monitoring recommended by Geospec 1 (GCO, 1989) should be followed. Permanent anchors should have adequate corrosion protection and long-term monitoring comprising maintenance inspection, residual load measurement and regular review of grease quality. Details should refer to Geospec 1 (GCO, 1989).

Typical lengths of anchors vary between 10 and 30 m. The anchors may have grouted fixed lengths or may be fixed at their far ends within anchorage tunnels driven for that purpose. An advantage of rock anchors is that they can be tensioned to a high force which is not the case for rock bolts.

### 7.5.3 Long Bolts Ahead of the Face: Spiling

The use of spiling (or forepoling) is presented in Section 6.9.4.

The spiling holes should be drilled from the perimeter of an excavation face and slanted outwards, typically between 5 and 15°, into the rock surrounding the length to be excavated. Deformed reinforcing bars of 25 mm diameter or more should be placed in the holes and fully grouted. Spile lengths are governed by the length of the round (4 to 5 m), with a minimum spile length being 1.5 times the round length. The spacing of the spiles is typically 0.3 to 0.5 m. However, in all cases, the design of spiles should be site-specific and based on a detailed evaluation of the ground conditions.

### 7.5.4 Canopy Tubes

The use of canopy tubes is presented in Section 6.9.5. An example of their applications is shown in Figure 7.6. They are normally drilled as a single row of tubes, but sometimes a double row in a staggered pattern is adopted. Modern computerised drilling jumbos or purpose-built drilling rigs may be used. Canopy tubes can be up to 24 m long and are installed at shallow angles to reduce overbreak. In practice, lengths of 12 to 15 m are typical to ensure adequate alignment control. Canopy tube deviations are typically minimal, but a nominal allowance should be made for setting out. Once a set of canopy tubes has been installed, they are grouted using highly permeable grout to provide contact with the ground and to assist with water cut-off and ground improvement. The canopy tubes are normally used in conjunction with lattice arch or steel ribs.

The selection of pipe diameter and connection type should consider the implications in terms of installation rate and safety for the workforce, as the use of heavier tubes may increase risks associated with handling of heavier structural sections.

**Figure 7.6 Canopy Tubes**

### 7.5.5 Steel Ribs and Lattice Arches

Section 6.9.6 gives the design principles for such steel supports. Steel ribs (or ribs and lagging) are not commonly used in Hong Kong; lattice arches in conjunction with shotcrete are preferred where poor ground requires more significant support as they are lighter and easier to install. In addition, due to their voided profile, they allow better and more uniform infilling of shotcrete behind arches. However, close site supervision should be provided to ensure good contact between an excavated face and the supports.

Tunnelling in soft or mixed ground may involve sequential excavation and support with a subdivided tunnel face. Each stage of tunnel excavation and lining construction must be considered for stability, as well as their effect on ground movements. Where the adequacy of bearing capacity is a concern, provision should be made for a localised enlargement (termed an elephant's foot) at the base of a tunnel invert. In addition, early closure of the tunnel support ring is critical to the control of ground movements in shallow tunnels. This may require rapid extension of steel supports during benching operations. The use of longitudinal support beams or wall plates may also be considered if necessary to enhance support provided by the shotcrete shell.

### 7.5.6 Shotcrete

Shotcrete is concrete placed by spraying and can be used as a temporary or permanent lining to a cavern and associated adits, tunnels and shafts. Shotcrete has properties similar to those of poured concrete. The material acts as rock support through penetration of cracks and crevices, which prevents movement at an early stage and locks the rock into place, and as structural concrete in some circumstances. Shotcrete has a stabilising effect considerably beyond that predicted by engineering analysis. The use of shotcrete in underground works requires an understanding of both rock mechanics and concrete technology.

Compared to cast in-situ concrete lining, shotcrete has the advantages of:

(a) short mobilisation time for equipment,

(b) no requirement for formwork,

(c) independence of the shape of an excavation,

(d) high rates of application,

(e) ease of combination with other support methods, and

(f) no need to fill over-break volume (unless a secondary lining is required).

The above advantages have led to shotcrete replacing cast-in-situ lining in many cases. The principal disadvantage is the rebound and occasional collapse of shotcrete applied to swelling rocks and other rock types and conditions giving poor bond, e.g. clayey rock or extensive water seepages.

Shotcrete is placed by pumping to a nozzle from which it is ejected at high velocity. The shotcreting process creates a potentially poor working environment. Therefore, workers should have appropriate protective equipment in addition to good ventilation and lighting.

The composition and properties of shotcrete are a compromise between the requirements for pumpability, ease of application, loss reduction and properties of the final product. Shotcrete should be applied onto a newly washed surface to help with bonding to the rock surface, with a nozzle approximately 1-1.5 m away and spraying perpendicular to the rock surface. If mesh, steel ribs and lattice arches are used, the angle of the stream should be varied to avoid voids forming in the shadow of the reinforcement. This requires careful attention from the operator.

The wet-mix shotcrete process is normally used in tunnelling because of several advantages over the dry-mix process as summarised below.

(a) *Consistency*. In the dry-mix process, water is added at a nozzle under operator control. It is easy to see if the concrete is too wet, but it is difficult to tell if it is too dry. This leads to an increased rebound loss and inconsistent quality.

(b) *Layer thickness*. Dry-mix can be placed in layers of 30 to 50 mm, whereas wet-mix with the addition of accelerator can be placed in layers up to 200 mm thick without the risk of slumping or separation from the rock. A normal layer thickness is usually in the range 75 to 150 mm.

(c) *Rate of application*. Dry-mix is commonly placed at up to 3 $m^3$/hour for expert contractors, against a rate of up to 10 $m^3$/hour for wet-mix.

(d) *Rebound losses*. The wet-mix process gives a rebound loss of 10 to 15% whilst the rebound for the dry-mix process is typically 20 to 25%.

(e) *Adhesion to rock*. Dry-mix can give better adhesion to rock than wet-mix when spraying on wet rock because the water/cement ratio and accelerator content can be varied rapidly to meet changing conditions.

(f) *Use with fibres*. Fibre loss from a dry-mix can be considerable, often in the range 20 to 30% and sometimes up to 50%, while a loss of 5 to 10% is typical for a wet-mix.

(g) *Working environment*. Contrary to the wet-mix process which produces a fog, the dry-mix process produces much more dust. However, when space is limited, hand spraying with dry-mix remains the only practical method.

Wet-mix shotcrete is usually used for tunnelling works in Hong Kong. However, pre-mixed and bagged dry-mix shotcrete is a useful risk mitigation measure for emergency temporary face support, when wet-mix shotcrete may be unavailable or delayed.

The shotcrete strength achieved is similar to that of poured concrete. Both wet- and dry-mix shotcrete can give compressive strengths of typically 30 to 50 MPa with adhesion typically being 0.5 to 1.5 MPa, but poor workmanship can reduce this to zero. Detachment of recently sprayed shotcrete from an excavation crown and shoulders is a safety risk and should be controlled.

The bond between layers is normally good, provided the subsequent layer is applied after the initial set of the lower layer, but within 24 hours of its application. However, consideration should also be given to the use of high early-bond strength shotcrete, if this suits the excavation pace and the construction methodologies employed.

Shotcrete has low permeability and will exclude water, provided that the water pressure does not cause cracking or debonding.

Shotcrete shrinks by 0.06 to 0.10%, depending on the water/cement ratio. In underground environments with good ventilation shotcrete can dry out fairly quickly. Precautions against drying out during curing should be taken to prevent development of shrinkage cracks.

The grading requirements for sand and aggregate for shotcrete are the same as for any other quality concrete. Additives are used in a mix design to improve the shotcrete properties. Plasticisers and super-plasticisers may be added to the mix to improve pumpability or achieve lower water/cement ratios and higher strengths. Air-entrainment agents improve resistance to frost damage where freeze-thaw cycles occur and may have advantages in Hong Kong in cold storage caverns. Accelerators, which are added at a nozzle, can be used in most mixes to achieve improved adhesion to wet surfaces, increase layer thicknesses and allow a shorter time interval between the applications of successive layers. Silica dust may be added by up to 8% by weight of cement to reduce losses and fall of concrete, to increase strength and to improve resistance to sulphate attack. Retarders are used to "sleep" shotcrete and can stop the reaction for many hours depending on the dosage, which is very useful when the shotcrete needs to stand-by for hours. The shotcrete will be reactivated once it comes into contact with the accelerator.

Care should be exercised in using shotcrete as a temporary support at an excavation face or as part of a permanent support. Shotcrete may be damaged by blast vibrations and may conceal stability problems which will only become apparent later. If shotcrete is not removed, it will become incorporated in the permanent support. Therefore, to prevent an inadequate permanent lining, a total support strategy must be determined before a section is sprayed. Normally, de-bonded shotcrete is removed and a further layer of shotcrete is applied.

Shotcrete does not adhere to clayey material. Where clayey seams are wider than the thickness of a shotcrete layer, the clayey seam should be bridged with reinforcement anchored into the rock on each side. Where swelling clays are proven or suspected, a cushion of mineral wool should be placed over the seam (Figure 7.7). Shotcrete can span clayey zones of up to 500 mm width, provided the clay does not swell. Shotcrete may not be suitable for stabilising wide zones of swelling clay. Support of such zones must be subject to special designs and in-situ concrete should also be considered.

Water pressure may build up behind shotcrete and may cause cracking of the material and destroy the bond to rock. The best practice is to divert water from a rock surface before spraying. Figure 7.8 shows a typical detail of surface drainage. The pipe shown there may be omitted if the seepage is minor. This type of surface drainage can be effective if correctly installed, but it requires good workmanship.

Fixing reinforcing mesh to a rock surface is time-consuming and costly, and requires personnel to work under partially supported ground. The simple and effective solution is offered by fibre-reinforced shotcrete (FRS) and has substantially reduced the use of mesh reinforced concrete. The fibre dosage should be determined considering the specific shotcrete performance requirements. However, mesh reinforced shotcrete may still be required in areas of poor bond, such as weakness zones.

Steel fibre-reinforced concrete (FRC) has been used for over 30 years and at the end of the 1990's, synthetic fibres were being introduced, including micro- and macro-synthetic fibres which are manufactured from polypropylene and polyolefin. Polypropylene fibres can be used to improve the performance of concrete in fires and the polyolefin for structural performance, similar to the use of steel fibres. It is also common to combine the use of shotcrete and rock bolts.

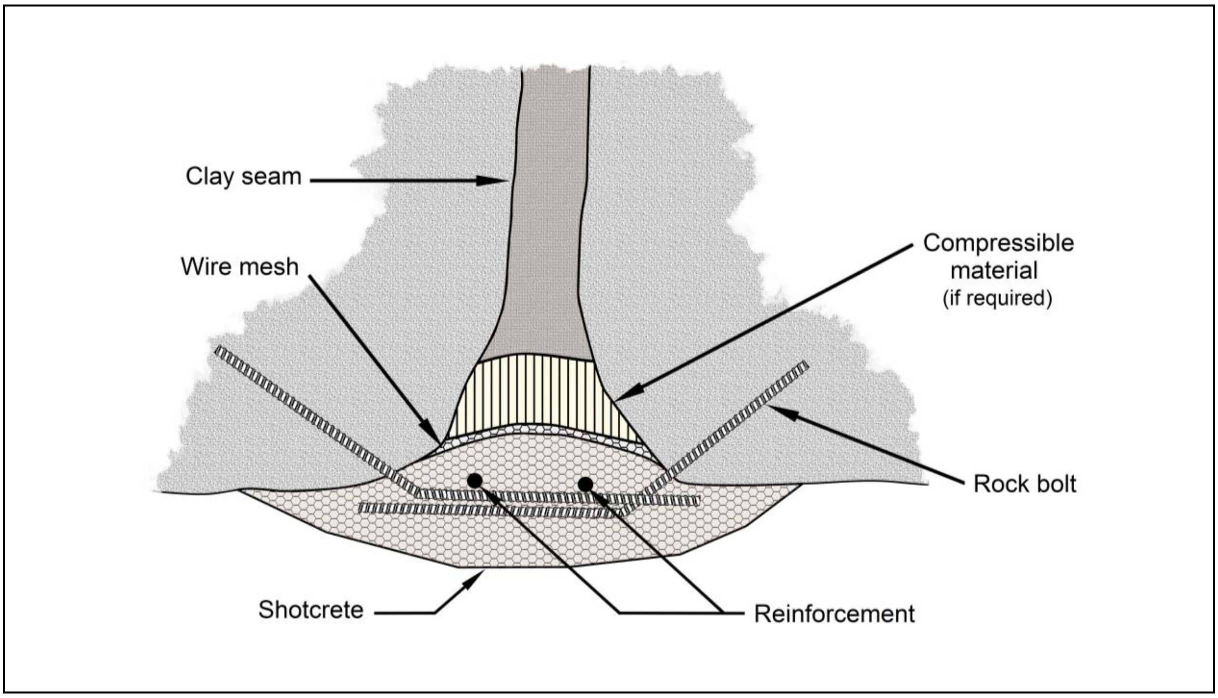



**Figure 7.7 Pre-treatment of Clayey Seams for Shotcreting**

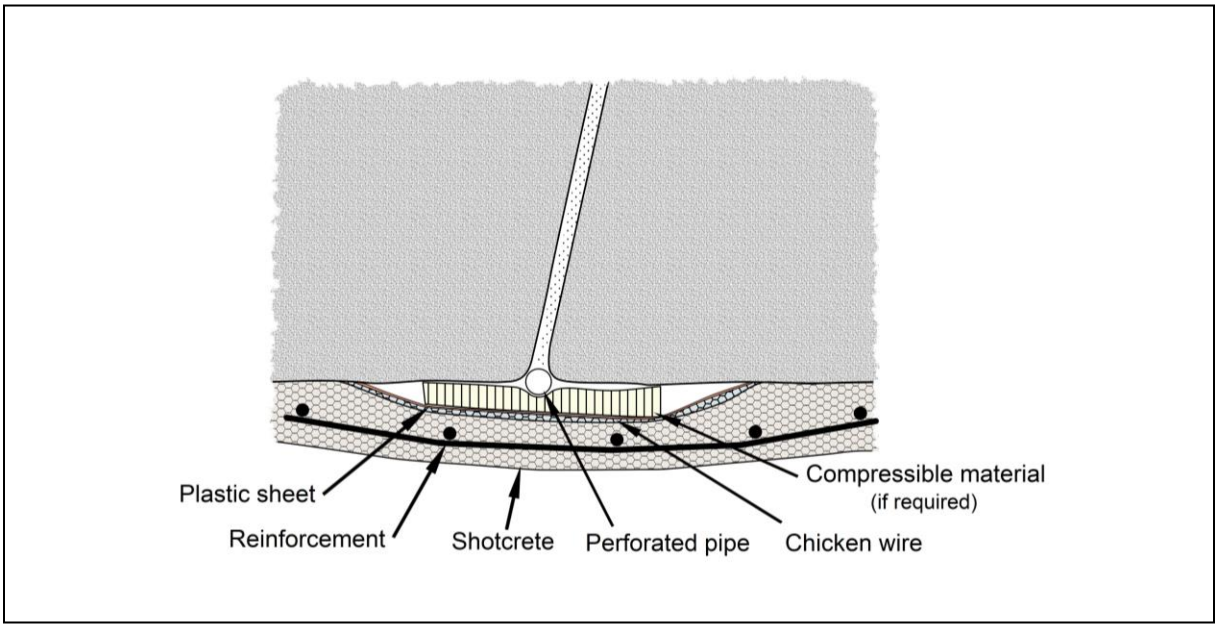



**Figure 7.8 Drainage Installed prior to Shotcreting**

There are now also polypropylene fibres that can be used instead of steel fibres for increasing the tensile strength with dosage rates around 1 to 6 $kg/m^3$. Polypropylene fibres are also used to prevent spalling of shotcrete due to fire, and, for this purpose, may be added at a rate of 1 $kg/m^3$. Expertise and care are required to prevent balling of fibres in a mixing process.

A shotcrete mix design is normally submitted by the contractor. Trials are normally conducted to ensure the proposed mix design meets the design specifications of the shotcrete. Trials are conducted by spraying test panels on a frame to replicate a tunnel. Concrete cores are then taken and crushed. Clements (2004) describes methods for early age strength testing of shotcrete. For the testing of FRS, beams are sprayed and used to test flexural strength. Compliance testing is then continued throughout the duration of a project.

Permanent FRS has been used in Hong Kong as the final lining in localised areas such as junctions and niches, where the geometry and complexity of the profile require difficult formwork provision and the quality of the final cast in-situ lining may be problematic.

The combined use of FRS with steel or plastic fibres and rock bolts is the most common method of temporary support. Where spot bolting is to be used in conjunction with shotcrete, the spot bolting should be done first, as the shotcrete obliterates sight of all features necessary for locating the bolts. It is considered that little advantage can be generally obtained by applying more than 200 mm of shotcrete when this is combined with rock bolting. This is particularly the case when the shotcrete contains steel fibre as reinforcement. However, thick layers of shotcrete may be required for localised areas with poor rock quality, and when using reinforced ribs of shotcrete.

### 7.5.7 Reinforced Ribs of Shotcrete

The ribs are constructed with a combination of steel bars, either in single or double layer (usually with a diameter of 16 or 20 mm), shotcrete and rock bolts (NGI, 2015). When using steel bars of 20 mm diameter, the bars have to be pre-shaped in order to gain a smooth profile. Attention should be paid to the provision of sufficient shotcrete cover to the reinforcement bars.

### 7.5.8 Concrete Lining

Cast-in-situ concrete lining can be used as the final support for caverns where large areas of poor rock are encountered. It may also be required to achieve high robustness. However, it is usually the most costly and time-consuming support method.

Due to its long construction time, concrete lining is not normally used as initial support. The initial stability of rock is often achieved primarily by other measures, such as rock bolts and sprayed concrete that precede the installation of concrete lining.

Plain concrete is commonly used in tunnels, but reinforcement may be used for large span caverns and other special circumstances as demonstrated necessary by structural analysis.

Formwork can be designed to suit the length to be concreted. For single pours, it is likely that a shutter in the form of scaffolding and timber formwork is used. When numerous concrete pours for permanent lining are proposed, a steel shutter with hydraulic ramps and wheels is used for positioning and moving the formwork quickly and accurately. Provisions should be made within a formwork before casting for back grouting, crack inducers, preparation of construction joints and waterproofing requirements (e.g. water stop hydrophilic strips).

Back grouting is required to ensure there is good contact between a concrete lining and the rock and is carried out by adding grout through grout holes formed in the permanent lining during concreting. The purpose of back grouting is to fill voids due to overbreak, and excessively high pressure should not be applied as this could create cracks within a permanent lining. In addition, care should be taken to ensure that blast vibrations do not affect the properties of concrete and reference should be made to Kwan & Lee (2000).

## 7.6 Blast Vibrations, Air overpressure, Flyrock and Blast Design

### 7.6.1 General

The process of blasting to break rock produces a large amount of energy from the rapid expansion of gases inside a blast hole. In so doing, the rock is broken. But other effects occur, such as ground vibration, air overpressure in the vicinity of the blast and locally flyrock can be ejected from the face of the blast. Various measures can be taken to mitigate and control these effects.

### 7.6.2 Blast Ground Vibrations

Ground vibrations from rock blasting are transmitted through the surrounding rock to the surface into the surrounding soils and structures. If sufficiently strong, these vibrations may cause damage to structures and equipment, as well as discomfort, disquiet or injuries to individuals. The ground vibrations that can be accepted may limit the size of the blast or necessitate vibration mitigating blast designs. Furthermore, the rock which forms the final surface of an excavation can be damaged if blasting design or execution is unsuitable.

Blast design is aimed at breaking up and loosening rock as quickly and economically as possible within the constraints set by limits on blast vibrations in terms of peak particle velocity (PPV). The design of economical blasting close to structures requires knowledge of the nature of the structures, installations and the building occupancy in areas that might be affected by vibrations such that realistic vibration criteria can be set. As it is difficult to obtain the actual energy propagation properties of the rock and soil between a blasting site and the receivers, attenuation of vibrations with distance is normally estimated using assumed, or site-specific, PPV versus scale distance relationships.

As a general guide, for typical blast charge weights in tunnelling, the blast vibrations from sub-surface works are normally not potentially damaging at distances of more than 50 m and only exceptionally at distances of more than 100 m. The potential for damage is dependent on the maximum instantaneous charge (MIC) and transmission of the blast wave through the ground, as well as the sensitivity of the facility or structure that may be affected by the vibrations. There are many factors involved in the transmission of the blast wave from the source. Site specific analysis is usually required to assess the impact of the blasting vibrations.

Vibrations are measured using seismographs which record particle velocities or particle acceleration and dominant frequencies. Sensors are mounted in three orthogonal directions and register separately. Sensors are rigidly mounted on or nearby the sensitive receivers to measure blast vibrations.

### 7.6.3 Blast Ground Vibration Acceptance Criteria

Blast ground vibration acceptance criteria depend on the type of structure, technical installations and occupancy, as well as the dominant frequency of the vibration. Blast ground vibrations normally have a frequency of 20 to 200 Hz which exceeds the natural frequency of most buildings. The dominant frequency depends on the medium transmitting the vibrations and can be some 40 Hz for soil, 40 to 70 Hz for soft or broken rock and 100 to 200 Hz for hard rock (Tamrock, 1989). The natural frequency of tall buildings may be estimated from the expression $f = 46/H$, where $H$ is the building height in metres and $f$ is the frequency in hertz (Ellis, 1980). Therefore, in general, amplification of vibrations due to resonance will not occur.

In Hong Kong, the term peak particle velocity (PPV) is commonly used in blasting assessments and in the vibration limits stipulated in various publications and by facility owners. For public works projects, the measurements of vibrations due to blasting should follow the requirements set out in the General Specification for Civil Engineering Works (HKSARG, 2006), or its latest revision. Table 7.2 sets out the current MTR vibration limits in Hong Kong. It should be noted that, at the time of writing, there is no vibration limit for human response, but air overpressure is limited to 120 dBL according to GEO Report No. 232 (Richards, 2008). The vibration limits that utility providers in Hong Kong generally apply to their assets are summarised in Table 7.3 and vibration limits for green concrete are provided in Table 7.4. In setting the vibration limits for blasting in blasting permits, the values stipulated by facility owners are sometimes adjusted to take account of the built quality and maintenance conditions of the facilities affected. Depending on the nature of a cavern project, requirements and good practice recommended by Government departments (e.g. Buildings Department, Water Supplies Department, etc.) should be observed.

The vibration limits outlined in Tables 7.2 and 7.3 are for reference only and a project proponent should seek agreement with the facility owners on the allowable vibration levels that can be tolerated. Depending on their detailing, built quality and maintenance conditions, the behaviour of facilities subjected to blast vibrations can be quite different. The condition of the affected facilities, in particular sensitive buildings, should be reviewed based on monitoring data and surveys as blasting works proceed and the vibration limits adjusted if there are any signs of blast-induced damage. Occasionally, small exceedance of the vibration limits occurs. This is normally accepted where no structural damage is found.

It should be noted that vibration-sensitive equipment in facilities such as hospitals, medical clinics or research laboratories may require a reduction in the PPV limit.

During the design stage, the owners of assets identified as sensitive receivers should be contacted to inform them of the future blasting works and to seek their agreement on the proposed vibration limits assigned to their property. The owner should, in turn, respond, confirming their acceptance or a proposed alternative limit.

Blast vibrations should be measured at ground level adjacent to a sensitive receiver or at the base of the slope affected. The vibration limits for slopes, retaining walls and natural terrain must be established separately for each case. Where it is not possible to measure blast vibrations at ground level or at the base of the slope, the vibration limit may need to be adjusted to take into account dynamic amplification effects.

There have been developments in the blasting limits set on soil slopes as outlined in GEO (2010), which promulgates a control framework for soil slopes subject to blasting vibrations. The PPV limits should be assessed in accordance with a rational method, or follow the prescriptive PPV limits given in Annex A of GEO (2010). Rational methods include the pseudo-static approach detailed by Wong & Pang (1992). For rock slopes affected by blasting, the energy approach described by Wong & Pang (1992) may be followed.

**Table 7.2 MTR Vibration Limits (Materials and Workmanship (M&W) Specification Tables 25.1 and 25.2) (MTR, 2009) (for reference only)**

| Installation | Allowable PPV (mm/s) | Max Amplitude (mm) |
|---|---|---|
| MTR railway structures and permanent way | 25<br>100(1) | 0.2<br>0.2 |
| Computers | (2) | - |
| Circuit breakers | (2) | - |
| Q relays | 40 | 0.2 |
| Insulators and overhead lines | 50 | 0.2 |

Notes:
(1) 100 mm/s PPV permitted during times approved by the Engineer and Operations Managers.
(2) Computers and critical elements that affect overall MTR operation should be reviewed on a case-by-case basis, as appropriate. When blasting near MTR facilities, computer limits should be reviewed pending latest hardware configuration/performance and isolation/damping of the equipment.

**Table 7.3 Vibration Limits for Utility Installations and Other Structures (for reference only)**

<table>
<tr><th>Utility/Facility</th><th>Installation</th><th>PPV (mm/s)</th><th>Max Amplitude (mm)</th></tr>
<tr><td>Buildings</td><td>Estates / structures (including historic) / schools / private properties[1]</td><td>5 to 25</td><td>≤ 0.2</td></tr>
<tr><td rowspan="3">WSD</td><td>Water retaining structures / water tunnels</td><td>13</td><td>0.1</td></tr>
<tr><td>Water mains / other structures and pipes</td><td>25</td><td>0.2</td></tr>
<tr><td>Pipes and tunnels within WSD protection zones</td><td colspan="2">Limits subject to WSD approval</td></tr>
<tr><td>Towngas</td><td>All installations<br>Gas governor</td><td>25<br>13</td><td>0.2<br>0.1</td></tr>
<tr><td>Hong Kong Gas</td><td>Pipes<br>Gas tunnels<br>Gas governors</td><td>25<br>13<br>13</td><td>0.2<br>0.1<br>0.1</td></tr>
<tr><td>CLP Power</td><td>Power stations<br>Sub-stations (major)<br>Sub-stations (minor)<br>Underground cable joints<br>Underground cable and pylon foundations<br>Cable Tunnel</td><td>11<br>13<br>25<br>13<br>25<br>13</td><td>0.1<br>0.1<br>0.2<br>0.1<br>0.2<br>0.1</td></tr>
<tr><td>Hong Kong Electric Company</td><td>Transmission (275/132 kV) cable and joint</td><td>12</td><td>0.1</td></tr>
<tr><td>Drainage Supplies Department (DSD)</td><td>All</td><td>25</td><td rowspan="7">not specified[2]</td></tr>
<tr><td>PCCW</td><td>All</td><td>25</td></tr>
<tr><td>Hutchinson Global Communications</td><td>Cables</td><td>25</td></tr>
<tr><td>Hong Kong Cable Television Ltd.</td><td>Cables</td><td>25</td></tr>
<tr><td>Hong Kong Broadband</td><td>Not specified</td><td>not specified</td></tr>
<tr><td>New World Telecommunication Ltd.</td><td>Not specified</td><td>not specified</td></tr>
<tr><td>Wharf T&T Ltd.</td><td>Cables</td><td>25</td></tr>
<tr><td>Highways Department</td><td>Structures and road drains</td><td>25</td><td>0.2</td></tr>
</table>

Notes:
(1) Each of the structures should be assessed separately and an allowable PPV determined based upon the structural form and condition of the building.
(2) "Not specified" means that no limit has been provided or set by the utility company.
(3) These limits are those generally applied by utility providers to their assets and additional site specific limits may be applied for particular sensitive assets.
(4) General Reference: GEO (2015).

**Table 7.4 Vibration Limits on Green Concrete**

| Age of Concrete | PPV (mm/s) |
|---|---|
| ≤ 4 hours | 10 |
| 6 to 8 hours | 20 |
| 10 to 12 hours | 30 |
| 24 hours | 70 |
| 3 days | 100 |
| 7 days | 125 |
| More than 28 days | 150 |
| Note: Limits based on GEO Report No. 102 (Kwan & Lee, 2000). | |

For retaining walls designed and constructed up to the current geotechnical standard with no signs of distress/instability or other stability concerns, the vibration limit can be taken as 25 mm/s. Alternatively, the vibration limit should be established based on a rational method such as the Mononobe-Okabe method (Dowrick, 1987).

Natural terrain often surrounds cavern developments and the potential for any adverse impacts (e.g. boulder fall) that may occur as a result of blast vibrations should be investigated.

Foundations and underground concrete structures have generally not posed a problem during blasting works, as they are confined structures and react in the same way as the ground. It is noted that for many MTR projects, the temporary works adopted by contractors, such as deep basement propping and retaining wall designs, have had PPV limits applied that ranged from 25 to 150 mm/s. This is based upon the analysis of the loads and the additional dynamic loading conditions on the temporary structures during the blasting.

Cast concrete and shotcrete support in underground construction will be exposed to blast vibrations but practice shows that these vibrations do not normally cause significant damage. Dowding (2000) describes experimental results by Esteves (1978) and observes that the threshold particle velocity inducing cracks in green concrete, between 7 and 20 hours after casting, is high, with a value of at least 150 mm/s. This is comparable to the results from experiments presented in GEO Report No. 102 (Kwan & Lee, 2000) for the lower bound of vibration resistance, with recommendations for allowable PPV limits for concrete as shown in Table 7.4. Furthermore, it has been observed that shotcrete without reinforcement can withstand vibration levels as high as 500 to 1,000 mm/s (Ahmed & Ansell, 2012). Therefore, for temporary support of underground excavations, the limit can potentially be raised (providing vibration criteria at the surface are not exceeded), but the integrity of the concrete would need to be verified after each blast.

Blast vibration limits for cavern schemes in Hong Kong should be set following a condition survey of existing structures, equipment, services and building use. The extra costs of adapting blast design to strict limits can be high. The most effective method to reduce vibration impacts on nearby structures is to reduce individual blast charge weight. Electronic detonators can also be used to improve the control on the accuracy of the timing of the blast detonation thus reducing the overall vibration with careful blast design (see Section 7.4.3). Blasting assessments should be carried out based on the condition of the adjacent buildings/structures/utilities/slopes in order to determine the blasting vibration limits.

Blast vibrations below the prescribed limits of sensitive receivers can still cause nuisance to the public. Experience shows that this problem and the ensuing complaints can be substantially reduced if affected persons are kept informed of the project through a suitable publicity campaign and the establishment of channels of communication, commencing at the design stage. This has been a successful approach in Hong Kong, especially when surface blasting is also required. Projects that have successfully included blasting in urban areas are those that have a detailed plan of informing the public of blasting times and conditions. This plan is usually targeted at various levels of engagement from district councils to residents and retailers. Holowenczak et al (2013) present a case study in the respect of public engagement for a tunnelling project, where blasting was carried out for the shaft and running tunnels in a densely populated area.

### 7.6.4 Air Overpressure

A blast underground produces an air overpressure wave within underground excavations that needs to be controlled and mitigated in order not to be harmful to workers and personnel nearby. Air overpressure design and considerations are discussed in GEO Report No. 232 (Richards, 2008). The underground area in the vicinity of a blast is vacated prior to blasting. In addition, a set of blast doors to control air overpressure and flyrock at the initial stages of an excavation at the portal are normally required in Hong Kong.

The use of refuges and safe places of shelter underground can be used to allow blasting to be carried out without evacuation of all workers to the surface. This allows work to be quickly restarted after each blast. Underground refuges were used during the construction of the Hong Kong West Drainage Tunnel (DSD, 2011) where concurrent blasting and tunnel boring machine (TBM) operations were carried out.

### 7.6.5 Energy Propagation

The vibrations that result from a blast may be calculated using an equation of the form:

$$\mathrm{PPV} = K\left(\frac{R}{\sqrt{Q}}\right)^{-b} \quad \text{.................................................} \quad (7.1)$$

where
- PPV = peak particle velocity (in mm/s)
- $K$ = a "rock constant"
- $R$ = distance in metres between the blast and the measuring point
- $Q$ = maximum charge weight per delay interval (in kg)
- $R/\sqrt{Q}$ = scaled distance
- $b$ = attenuation exponent.

The local vibration attenuation constants $K$ = 644 and $b$ = 1.22 (Li & Ng, 1992) are used for an initial assessment when predicting vibrations and for producing vibration contours in blasting assessment reports. These constants are based on the upper 84% confidence level. It should be noted that a higher confidence level is typically required for site-specific constants.

Geographic information system (GIS) based models are typically developed to assess the impacts of blasting in Hong Kong. The locations of all underground excavations and all of the sensitive receivers are entered into a GIS model (e.g. utilities, buildings, slopes, etc.). The output of the model can then generate the maximum instantaneous charge (MIC) for any section of a tunnel. Once the blasting works commence and site specific monitoring data becomes available, the GIS model can be updated with new site specific constants. In addition, models can be developed to predict air overpressure for open blasting works (Richards, 2008). Chuang et al (2009) provide further information on the GIS approach. The adoption of new site-specific constants needs to be approved by the regulatory authorities.

In urban areas, the calculated MIC can be low because of the presence of critical sensitive receivers. This may restrict production and make rock blasting expensive and time consuming, as normally the round length will be reduced to take into account the low MIC due to the critical sensitive receivers. Alternatively, length of drillholes can remain the same but explosives can be decked in the hole. This method has been used in Hong Kong recently with success.

When critical vibration sensitive receivers are present, blasting trials should be carried out on the initial production blasts with careful vibration monitoring to establish the local relationships between charge weight, distance and vibrations, thus optimising the charge weights.

Once site-specific blast attenuation constants have been established, an observational approach may be considered where the recorded PPVs are much lower than the vibration limits for blasting at the monitoring measurement locations in the blasting permit. In using this approach, the site constants should be refined continually, taking into account new monitoring data (with those old monitoring data no longer representative of the site conditions discarded), in order to optimise the blast design. The new site constants, based on appropriate statistical confidence levels, should be agreed with the regulatory authorities. As geological conditions and blast-induced vibrations can be highly variable, a 90-95% statistical confidence limit is generally used in assessing the maximum instantaneous charge for a blast design. This is to help ensure that the vibration limits for blasting in the blasting permit are not exceeded by more than 5-10% of the blasts. However, the acceptability of any vibrations above prescribed limits needs to be agreed with the facility owner.

### 7.6.6 Blast Design

The objective of a blast design is to break the rock to meet the excavation objective, leaving the smoothest possible rock walls, with minimal over-break and damage to the surrounding rock, as quickly and cheaply as possible, whilst conforming to the blast vibration and air overpressure criteria. A pattern of drillholes and a charging pattern have to be determined. With modern drilling rigs, holes can be placed very accurately with correct angles. As far as practicable, a blast design should also consider the optimum degree of fragmentation for subsequent reuse of excavated materials. The Blasters' Handbook (ISEE, 2011) is a good guide for the design of blasts and other explosive-related issues.

Drillholes for tunnel blasts can be classified into five groups according to their purposes:

(a) *cut holes,* which are blasted first, are heavily charged and create the free surface for the rest of the blast,

(b) *relief holes,* which are the large uncharged holes providing the free face for the rock to break into,

(c) *production holes,* which break the bulk of the rock,

(d) *cushion holes,* which are the row of holes next to the contour holes, lightly charged, reducing the chance of damage propagation in the remaining rock mass, and

(e) *contour holes,* on the edge of the blast, which define the excavation perimeter and which limit the overbreak and damage to the surrounding rock.

Figure 7.9 shows a typical blast design for a top heading. The cut design is the parallel-hole cut. The cut holes are located close to the middle of the total blast and consist of charged holes and large uncharged holes.

Length of holes is normally in the range of 4 to 6 m and the pull (the length of tunnel blasted) is typically 0.5 m less than this.

Smooth blasting is very important to minimise damage and over-break in rock walls and crown in underground excavations. The method is characterised by correct drillhole spacing and charge distribution in the perimeter and preferably simultaneous or minimum delay difference between detonations of adjacent drillholes. Efforts to achieve correct setting-out and reduce drillhole deviation are important. The use of modern computerised jumbos is preferred to reduce drillhole deviation.

Where non-electric detonators are used, a minimum separation of 8 ms between each firing is recommended by the United States Bureau of Mines (USBM) and is the common practice in Hong Kong. However, if electronic detonators are employed, the time separation can be further reduced, if necessary, to around 2 ms, depending on the accuracy of the electronic detonators.

Locally, the most common method for carrying out smooth wall blasting is to use detonating cord in combination with a cartridge at the base of a blast hole. The advantages of this method are that it provides a low charge weight per linear metre and is easy to carry out. The low charge weight in association with the high speed of detonation of a detonating cord creates a tension crack in the rock that propagates from hole to hole around the perimeter. In addition, the low quantity of explosives means there is less damage to the rock mass. Contour hole charges are detonated after the main blast. The charge in the contour holes, as well as the blast holes nearest to them (cushion holes), should be carefully controlled so as to prevent damage to the rock beyond the contour holes. Due to constraints placed on the use of detonation cord in perimeter holes and for economic reasons, emulsion-filled tubes, as well as string-loaded pumped emulsion, have been used on a number of projects with mixed results.

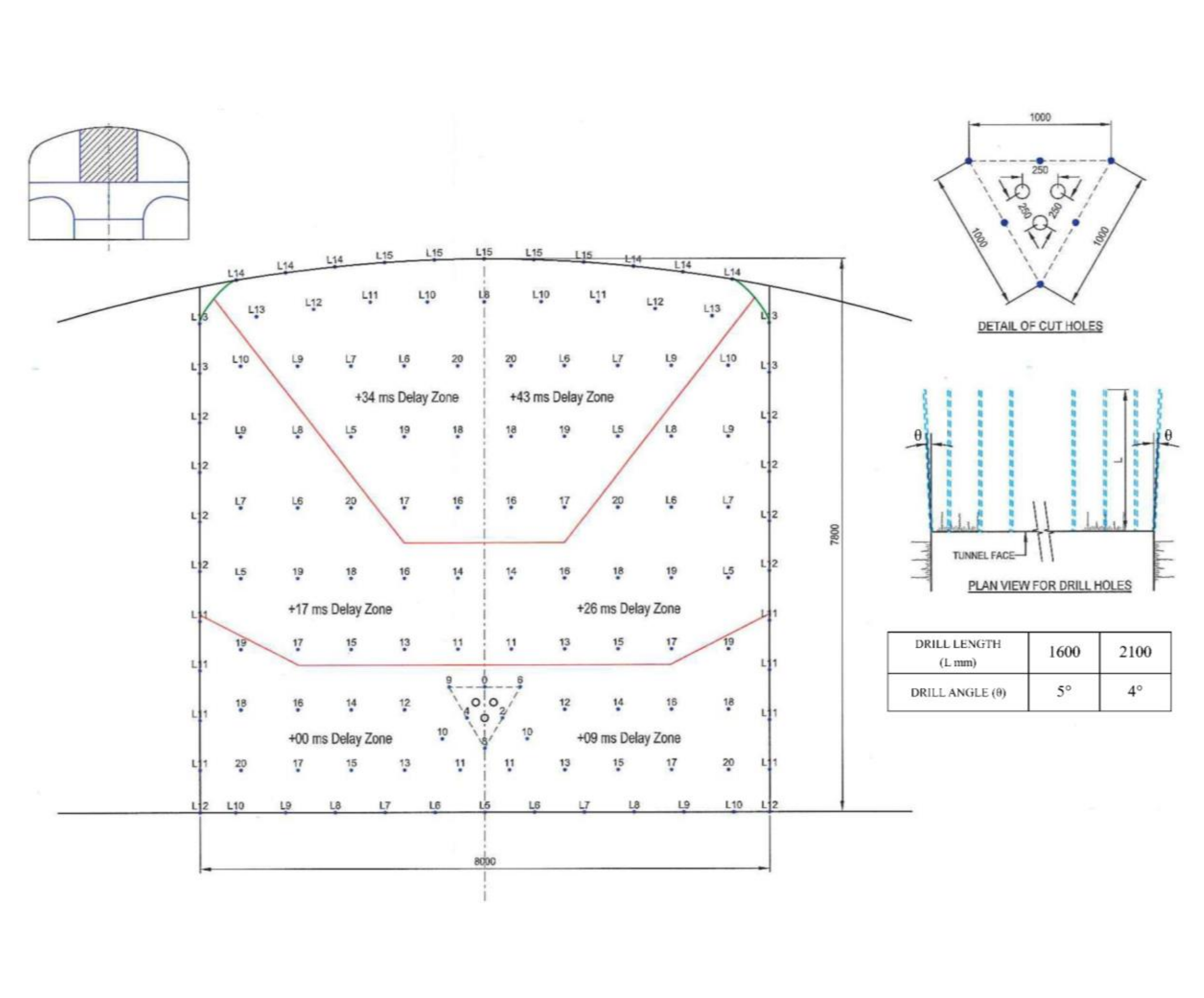



**Figure 7.9 Typical Blast Design for a Top Heading**

Drillhole diameters for cavern top headings in Hong Kong are commonly either 45 or 51 mm. For the bench blasting, with vertical holes, the diameters can increase up to 76 mm. The powder factor, defined as a ratio of the quantity of explosives (in kg) to the volume of rock (in $m^3$) to be excavated, can provide a guide for determining the quantity of explosives for rock excavation. The factor is largely dependent on the rock conditions, size of the excavation and the optimum size of the blasted rock for removal. Cavern top headings are excavated in a similar way to large tunnels. For cavern benching, the drilling will be either vertical or horizontal and will depend on the contractor's method and equipment.

Figures 7.10 and 7.11 show the relationship between the number of blast holes and face area, and the powder factor and face area, respectively, for full face blasting for various local tunnel projects. The figures are based on data collected from the Mines Division of the GEO for projects completed between 2010 and 2016. The grey zone on each figure indicates the typical range used in Hong Kong. It should be noted that the number of blast holes, as well as the powder factor, can vary significantly among projects as a result of differences in the blast design, type of explosives and competence of the designer and shot firer, blast hole size and accuracy of drilling, geology, including rock strength and presence of discontinuities, fragmentation requirements, and the constraints of sensitive receivers, as well as the production cycle programme.

Although single-deck blasting is less risky and complicated, multi-deck blasting has been carried out in some recent local cases of tunnel and underground applications. The primary objective of this type of blasting has been to increase pull length and production for each blast where the maximum instantaneous charge (MIC) is restricted. It is essential that all the blast holes in the bottom deck are live before the upper deck initiates in a multi-deck blast. Misfires and/or initiation problems can occur when multi-decking is used and detailed risk assessment, adequate preventive/mitigation measures and contingency plans must be in place to ensure blasting safety. The adoption of this method needs to be balanced against a longer setup time in preparing the blast.

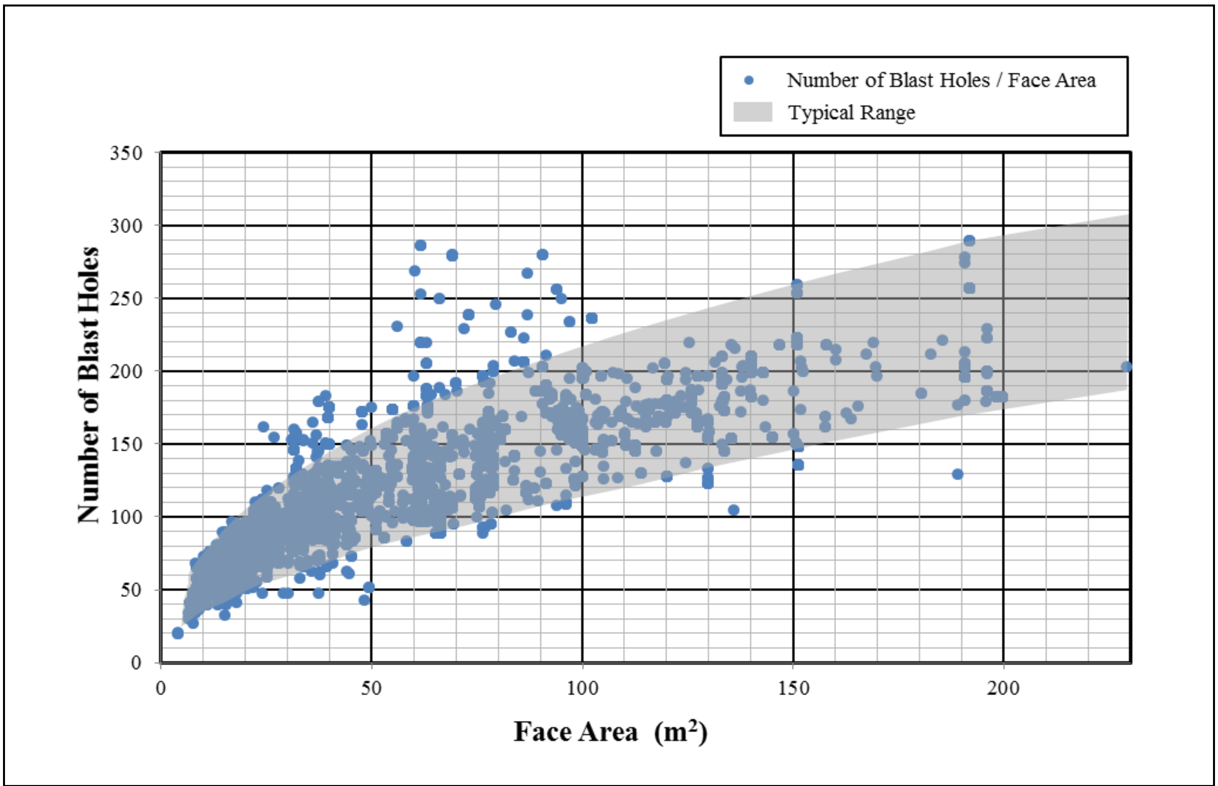



**Figure 7.10 Number of Blast Holes vs. Face Area for Full Face Tunnel Blasting in Hong Kong (based on data from the Mines Division)**

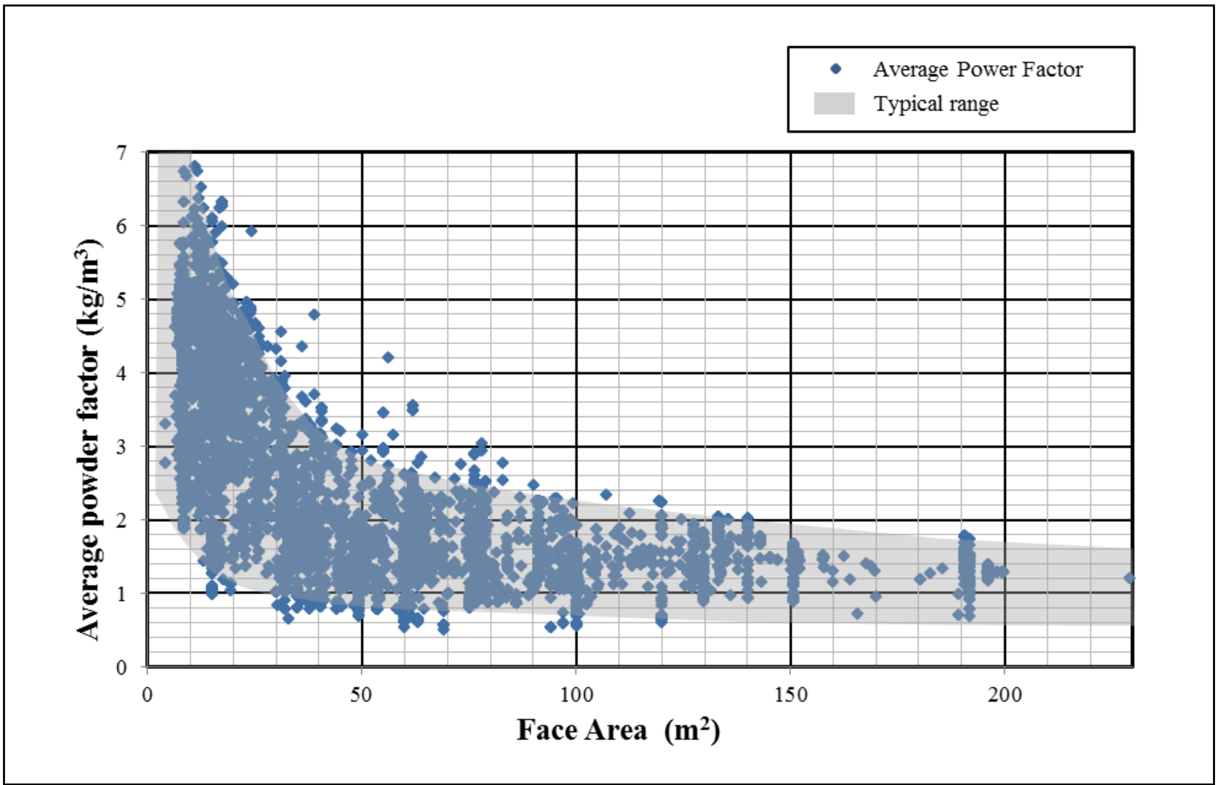



**Figure 7.11 Powder Factor vs. Face Area for Full Face Tunnel Blasting in Hong Kong (based on data from the Mines Division)**

The explosives commonly used in Hong Kong are cartridge emulsion, bulk emulsion and ANFO. Cartridge emulsion is commonly used for smaller tunnel faces. Bulk emulsion is used for larger faces and is supplied by truck mounted pumps and directly pumped into the face. ANFO can also be used underground and is blown into holes, but it is important that ANFO is only used when a tunnel face is relatively dry. When bulk emulsion or ANFO is used, it is common for holes to be bottom primed using cast boosters or cartridge emulsion to initiate the main charge.

### 7.6.7 Blasting Trials

Blasting trials carried out in drillholes for the purpose of verifying the attenuation coefficients in the vibration attenuation equations have been attempted from time to time in other countries, but have generally not improved the predictability of the equations. This is because of the differences in confinement between the charges in a trial drillhole blast and production blast-holes, and because a single detonation in a drillhole cannot simulate the delays in detonation between different blast holes. At the design stage, it is considered that vibrations are best predicted using available empirical equations (see Section 7.6.5).

In practice, the initial blasts for a proposed works are usually used as the blasting trials and then attenuation constants are derived from these to get a new site-specific equation (see Section 7.6.5).

## 7.7 Probe Drilling and Grouting

### 7.7.1 Need for Probe Drilling and Grouting

Systematic probe drilling should generally be carried out to check the ground conditions ahead prior to excavation. Such drilling should check for any zones of weakness and water bearing zones to determine if grouting may be required for stabilisation or seepage reduction. Depending on the size and shape of the cavern, probe holes may need to be conducted at several locations to get an understanding of the ground conditions, and consideration should be given to extending probe holes slightly above the top heading.

### 7.7.2 Drilling Probe Holes and Grout Holes

It is recommended that probing ahead is carried out as a component of rock tunnel and cavern excavation, as it provides the opportunity to plan ahead, recognise changing ground and groundwater conditions and implement appropriate changes.

Probe and grout holes are normally drilled using a drilling jumbo. Hole lengths of up to 60 m can be achieved, although there can be problems with increasing length, including hole stability, rods jamming, significant deviation and increasing drilling time. In practice, probe holes of 30 to 40 m are typically the longest length regularly carried out.

Exploratory holes should extend beyond the line of the next stage of excavation and should be sufficient in number to explore the rock adequately, with the number of holes, their lengths and directions being determined on site based on the ground conditions encountered. It is preferable to maintain a probed zone 20 m in front of an excavation. Verification probing should be carried out during an excavation, even if horizontal directional drilling has been carried out prior to the construction phase.

Probe holes can be drilled along the full length of a cavern as soon as the top heading face is established. Drilling these long holes takes time but may benefit the overall excavation programme, as exploratory drilling can then be excluded from the subsequent drilling, blasting and excavation cycle.

Where the information from probe holes is considered to be insufficient, perhaps due to possible zones of weakness, changing ground conditions, high groundwater inflow or contradictory information, then additional probe holes should be considered. In certain circumstances, coring may be beneficial, although this can be time consuming and may not be possible during excavation due to operational reasons.

The direction and number of grout holes should be such that the hole spacing will ensure good treatment of the surrounding rock. There should be an adequate safety margin of explored and/or treated rock. The safety margin should be determined on the basis of the geological information and the risks involved.

### 7.7.3 Interpretation of Probe Drilling

The decision to grout can be taken according to pre-set criteria on the basis of the water inflow from probe holes. This approach gives better results than grouting specified on the basis of data from investigation holes drilled from the ground surface.

Experienced engineering geologists or tunnel engineers should be employed to log probe holes, taking into account changes in the penetration rate, rods jamming (indicating possible weak or highly jointed material), flush colour, water inflow rate and chippings. This information, if logged properly, can accurately determine the location of an area which will require pre-treatment.

Modern drilling jumbos can be equipped with data loggers that record data from the drilling that can be used to help with the interpretation of the ground conditions. An example of a 3D output model based on logged drilling data is shown in Figure 7.12, where potential weaker zones are highlighted in red.

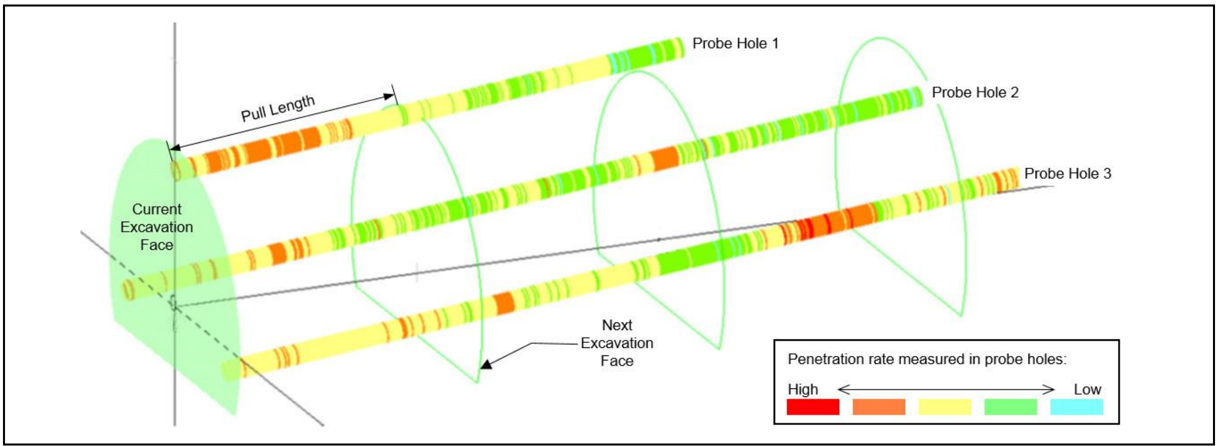



**Figure 7.12 Example of 3D Model Based on Logged Drilling Data from Probe Holes ahead of Tunnel Face (based on records provided by Nishimatsu Construction Co. Ltd.)**

### 7.7.4 Pre-grouting and Post-grouting

Grouting and sealing works, where required, in rock should preferably be performed ahead of an excavation ("pre-grouting") and further information is provided by Garshol (2007), Barton (2011), and the Norwegian Tunnelling Society (NFF, 2011). High pressure applies only to pre-grouting well ahead of a tunnel/cavern face, while grouting and sealing carried out after an excavation ("post-grouting") is generally more costly, time-consuming, less effective, and has to be done at lower pressure (Tattersall & Grov, 2009; Barton, 2011). Pre-grouting can save both time and money compared to post-grouting. Figures 7.13 and 7.14 illustrate pre-grouting concepts.

The grouting equipment should preferably be designed to inject multiple holes simultaneously to maintain production rates in large caverns. The grouting equipment should also be able to take pressure readings for individual holes and should allow the grout take of each hole to be recorded using a flow meter. Dual stop criteria of grouting should be set by combined grouting pressure and grout take quantity. Further guidance is given in Section 6.10.4(2).

The quality of grouting works should be checked by drilling new exploratory holes prior to an excavation to ensure water ingress is sufficiently suppressed.

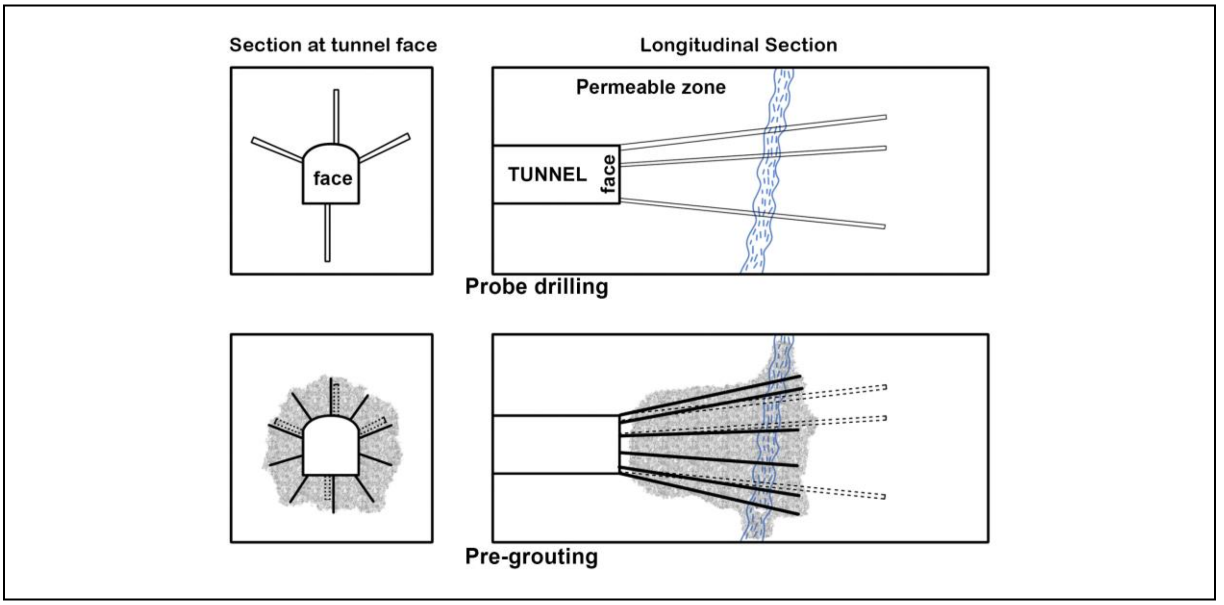



**Figure 7.13 Probe Drilling and Pre-grouting ahead of Tunnel Advancing Face or Cavern Heading**

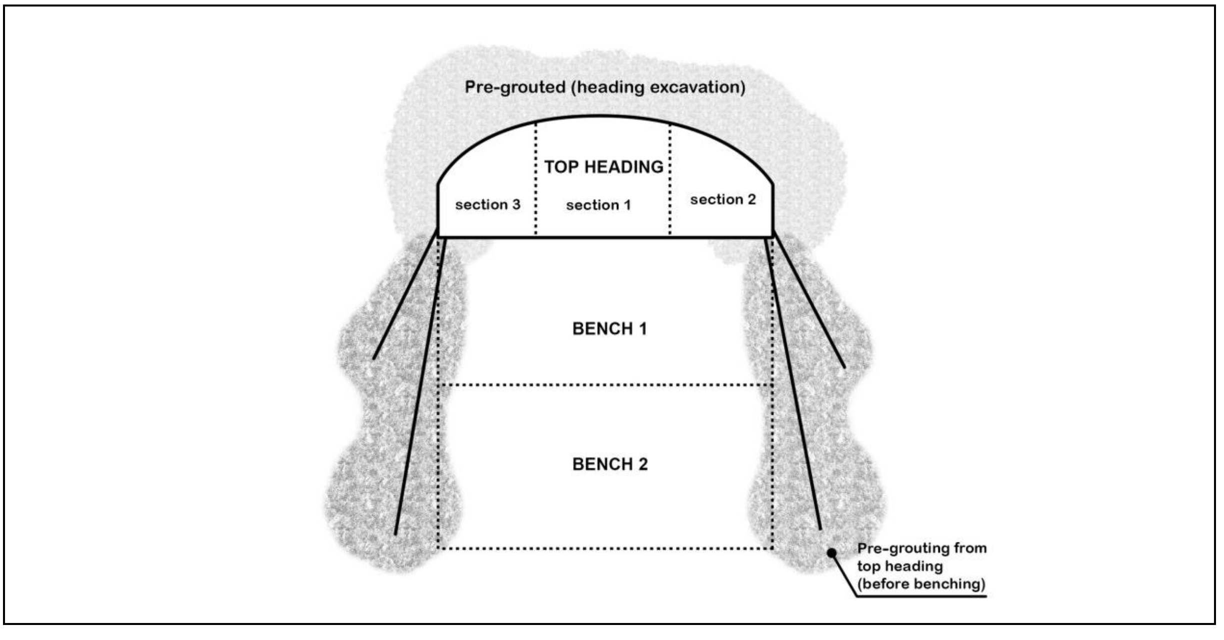



**Figure 7.14 Pre-grouting Downwards from the Heading in a Cavern**

In areas where pre-grouting has been carried out, the grout curtain must be thicker than the length of any rock bolts, particularly where the bolts will not be grouted. Otherwise, the rock bolts will simply penetrate the grout curtain and form new pathways for water to enter the excavation.

### 7.7.5 Grouts and Grouting

The most commonly used grouts are cement-based. Chemical grouts may be required where there are strict criteria for water tightness or inflow. Coarse-grained and expanding cement grouts are used to stem leakages arising from wide, open discontinuities in a rock mass. Neat cement grouts are used to treat rock with Lugeon values down to 0.5, but fine cements (microcements) are usually employed for the lower range to ensure sufficient penetration into a rock mass. Bentonite and preferably other agents (superplasticisers and microsilica) are used in some cement grouts to improve grout properties. It should be noted that bentonite is not a suitable additive when grout is used under high pressure.

For high pressure grouting in rock, microfine cement is the primary grouting material, supplemented by colloidal silica where cement cannot penetrate, and further sealing off is still required. Grout must be stable, non-bleeding and have low viscosity. To achieve this, the properties of cement must be right and the use of dispersant admixture (e.g. superplasticisers) is needed.

Polyurethane-based foam may be considered as an alternative material to stem large inflows of water, particularly where low grout pressures have to be used. This grout type expands on contact with water. Grouting with silicates or epoxy grouts is appropriate where good water tightness is needed, and is commonly applied after first stage sealing with cement grouts.

Current grouting equipment can achieve high pressures of up to 100 bars. High pressure grouting should be adopted wherever possible, as this allows the grout to penetrate further into the discontinuities providing a larger area of grouted ground. However, where ground cover is low, then the grout from high pressure grouting may penetrate to the surface or to near surface assets, potentially causing problems. Micro-fine cement and colloidal silicates may need to be used (possibly in multiple phases) due to particular inflow and penetration requirements and the size of discontinuities.

It is normal for water inflow to come from above a cavern and therefore penetrate through the crown and sidewalls. However, in some areas of Hong Kong, high water pressure has caused inflow through the invert of excavations and this should be considered and checked when assessing a groundwater system and while probing and grouting.

## 7.8 Reuse and Disposal of Spoil

### 7.8.1 Rates of Spoil Production

A cavern excavation typically requires high excavation rates. Rates of 60,000 $m^3$ of rock per week can be achieved in large cavern construction projects overseas. It should be noted that the production and spoil disposal rates are dependent on a number of factors, including ground conditions, number of faces available for excavation and the allowable maximum instantaneous charge (MIC).

### 7.8.2 Options for Spoil Reuse and Disposal

Reuse of excavated material should be a priority and, in particular, the disposal of good quality granite as low cost fill, whether in Hong Kong or elsewhere, should be avoided wherever possible. For instance, good quality granite can be sent to a quarry or a rock crushing plant for further processing into aggregates, which provides a more sustainable solution. Therefore, an assessment of the suitability of the rock for use as aggregate should be carried out, with appropriate testing during the planning and design stage of a project (see Section 5.8.12).

The options for spoil reuse and disposal vary from site to site and include:

(a) crushing on site (underground or surface) or transportation to quarry sites for processing into aggregates,

(b) reuse on the same project, or nearby, for rock fill slopes, and

(c) transporting from site to barging points via conveyor or road transportation for export, to reclamation or other disposal site.

For large cavern projects, in particular, detailed consideration at the planning stage of a project should be given to setting up an underground (as part of the cavern development or as an associated entity) rock processing operation and related facilities, such as a concrete batching plant. These facilities can process suitable rock into aggregates for reuse in the project or for off-site sale and, as such, can help offset the project costs. They would also minimise environmental and traffic impacts on the community during construction. The demand for aggregates and the scarcity of suitable facilities for processing surplus rock locally means that a rock processing facility, with sufficient capacity, can also generate additional income by crushing good quality surplus rock from adjacent construction projects.

In many areas, road transport of spoil from a large cavern with high production rates to a disposal site may not be feasible, due to limitations in road traffic capacity or adverse environmental impacts. Therefore, these issues need to be assessed during the planning stage of a project.

## 7.9 Waterproofing and Other Mitigation Measures

The principal methods for water control in caverns have been introduced in Section 6.10.4. Drainage installed behind shotcrete should be considered (see Figure 7.8 for typical details of such drainage). The method is effective but is dependent on good workmanship. It should be noted that it can easily become blocked over a number of years due to mineral build-up. Drip screens have considerable advantages in that they are cheap, can be quickly erected and are less dependent on good workmanship, but they do need to be maintained. Drip screens can form a pleasing architectural finish and also prevent any minor loose particles from falling into the cavern. They can also be fitted after equipment has been installed but this can cause disruption. In Hong Kong, caverns have been known to leak after their completion. Owners should consider whether the additional cost of a fully lined and waterproofed opening is beneficial to avoid potential problems later.

For concrete lined openings, a combination of geotextile and sheet membranes diverting water to an under drain below the base slab is common. Based on the manufacturers' specifications and experience gained from the projects in Hong Kong, a smoothing shotcrete layer should be sprayed before placing geotextile and membrane. This is to smooth out any sudden changes in profile which can cause piercing or ripping of a membrane when concrete is subsequently poured.

Drainage of any collected water is normally by a pipe laid in the invert. Alternatively, cuspated sheets of high-density polyethylene (HDPE) material can be placed in the invert across the whole width of an opening. Cuspated sheets can also be used to provide drainage in the walls and crown of an opening. Cusps can be formed in various sizes to cater for different flow rates.

A cementitious polymer-based, spray-applied membrane can also be used. This bonds to the inner and outer lining, preventing water migration behind a waterproofing membrane (Verani, 2009; ITAtech, 2013). Examples of the use of sprayed-applied waterproofing membranes in Hong Kong are given in ITAtech (2013), Yim et al (2009), Ooi (2008) and Salisbury et al (2006). The following key requirements for successful sprayed membrane installation are drawn from the above guidance and local experience:

(a) method suitable for excavation with limited water ingress, or for tunnels treated with dewatering or pre-injection methods to control water ingress to manageable levels during construction,

(b) adequate substrate surface quality and preparation,

(c) adequate selection and maintenance of spraying equipment, to promote adherence of membrane to the surface with minimum rebound and maximum adhesion and coverage,

(d) proper training and accreditation of applicators, and

(e) application trials, and close and systematic quality control of an in-situ produced membrane, to ensure correct thickness and coverage, and curing of the membrane under the tunnel environmental conditions.

## 7.10 Construction Time

### 7.10.1 Rates of Excavation

The excavation rates achieved for the tunnel projects by blasting in Hong Kong, based on the data collected by the Mines Division of the GEO, is principally a function of the overall project constraints, including the size of the tunnel face area. For the tunnels with significant site, programme or resource constraints, e.g. the contractor is only blasting once every two days, or is delayed by related works or in obtaining regulatory approvals, overall average rock excavation rates had been less than 5 metres/week from the start to finish of the blasting for a section of the tunnel. In contrast, for the projects with few constraints, where the contractor had utilise, at or close to, optimum resources for the size of the tunnel, overall average rock excavation rates of between 21 to 38 metres/week had been achieved; those tunnels with larger face areas (generally greater than 100 $m^2$) tend to achieve better overall rates of advancement when compared to those tunnels with smaller face areas.

For full face blasting, a maximum weekly excavation rate of 70 metres/week is the practicable maximum that can be achieved when undertaking two blasts per day, six days per week, with a 5.8 metre drill/pull length. Data from the local tunnel projects shows that this rate has been achieved for the tunnels with a face area greater than 100 $m^2$. For the tunnels with smaller face areas, the maximum advancement rate tends to be dictated by the pull length that could be successfully achieved when blasting; smaller face areas can also constrain the overall production cycle.

There are various reasons for the production rates in Hong Kong being lower than the internationally achievable rates. For example, in Hong Kong, much of the blasting is carried out within or very close to the urban areas where blasting control is more stringent and where sensitive receivers are present. Various other control measures, such as grouting to control groundwater inflow as well as requirements to apply shotcrete support early, lead to longer cycle times, shorter pull lengths and slower blasting progress.

Rates of bench excavation in caverns can be high, but vary considerably. The production rate is dependent on the volume of rock that can be blasted in any one round, the time required for ventilation after a blast, the type and size of plant available, transport distances and spoil disposal arrangements.

Where drill and break are adopted (i.e. where blasting is not feasible), an advance rate of less than 1 m/day is common in good quality rock, even with multiple equipment working on the same face. Rates are very low in massive granite due to the lack of discontinuities.

Rock support and groundwater control requirements can also affect production. Overall production rates from a scheme are dependent on the number of caverns or benches that can be worked simultaneously. Drilling rates in bench excavation are high due to good accessibility and do not normally affect excavation rates. Ignoring external constraints, production rates from single caverns can be of the order of 5,000 to 10,000 $m^3$ per week increasing to 60,000 $m^3$ per week or more for multi-cavern excavations (although such high rates have yet to be achieved in Hong Kong).

### 7.10.2 Rates of Support Installation

The production rates that can be achieved for rock support are:

(a) Rock bolting: 6 to 15 bolts per hour; and

(b) Wet-mix shotcrete: 25 $m^3$ per hour.

For rock bolting, the lower rate applies to tensioned bolts and the higher rate applies to friction bolts. The production rates achieved are highly dependent on the bolt length, the working conditions, the experience and quality of the crew, and the type and quality of the equipment. Some proprietary bolt types can be installed at a higher rate using mechanised bolting rigs.

## 7.11 Construction Records

### 7.11.1 General

There are a variety of construction records for a cavern development that should be prepared, carefully managed on site and ultimately archived so they are accessible in the future. These include:

(a) blast records including design, loadings and monitoring records, vibration, air overpressure,

(b) rock mapping records,

(c) groundwater inflow,

(d) grouting records,

(e) settlement (e.g. of surface structures and utilities),

(f) excavation deformation monitoring (e.g. convergence monitoring of pins and extensometers),

(g) temporary support installation,

(h) testing records (e.g. material testing and shotcrete and rock bolt pullout tests),

(i) design verification,

(j) as-built records, and

(k) certification.

### 7.11.2 Rock Mapping

Rock mapping records are of great importance, as rock mapping forms the main basis of a temporary support selection during construction and also influences the selection of permanent support. Key information which should be included in a rock mapping record is:

(a) key project data and the details of individual mapping locations,

(b) basic rock mass information, such as lithology, mass weathering, geological structures, overbreak, and water inflow condition,

(c) details of mapped discontinuities, including joints, faults, bedding, foliation, flow-banding, slickensides, etc. – the logging engineering geologist is expected to follow standard guidelines for lithological description contained in Geoguide 3 (GEO, 2017a), and

(d) details for assessment of the Q-System, or other rock mass ratings, where applicable.

A summary of the rock mapping information should also be shown on a long section plan. The long section should include information on rock structure and material, location and type of temporary support installed and location and rate of any water inflow.

An example electronic template (i.e. Rock Mass Mapping and Classification Sheet) for rock mapping has been developed by the GEO, which can be downloaded from the CEDD Website.

### 7.11.3 Groundwater Inflow

Groundwater inflow at the pre-excavation stage can be recorded from probe holes. The water inflow limit should be specified in the specification, and if any inflow exceeds the limit, then grouting should be undertaken.

Groundwater inflow should also be monitored and recorded after an excavation to ensure the inflow is within the specification, and the designed drainage capacity is sufficient. There are different methods used to measure groundwater inflow, e.g. inflow measurement through a V-notch weir and flow meters on pumps (the instrument spacing shall be determined on project specific basis).

According to Heuer (1995), hard rock blasted tunnels commonly encounter inflows that are in the range of 2 to 5 times the flow from a probe hole. However, the actual inflow varies depending on many factors, including rock type, groundwater regime and rock mass fracture system, and site-specific monitoring is always crucial.

## 7.12 Safety during Construction

### 7.12.1 General

Safety during construction is of paramount importance in underground excavations and there are additional considerations compared to a normal construction site because the excavations are generally confined spaces until later stages in the construction.

Common construction risks that should be managed include slips, trips and falls, noise, dust and fumes, fire, flood, plant movements, operating mechanical and electrical equipment, rockfall, cave in and other falling objects.

Each cavern development is unique and health and safety risks will need to be assessed continuously throughout every project. Common methods of managing safety for underground excavations include mandatory health and safety qualifications and training, adequate training in the safe use of equipment, plant and vehicles, accident and near miss reporting systems, safety walks, risk assessments, internal and external health and safety audits, a tally system for entering and exiting an excavation, comprehensive emergency procedures, designated haul roads and walkways (preferably with barrier separation), adequate lighting and ventilation, appropriate personal protective equipment (including respirators if required), no petrol powered plant or vehicles permitted underground (due to fumes), adequate drainage, fire prevention, fighting and survival measures, no or minimal working under unsupported ground (use of shotcreting robots and dedicated rock bolting machines wherever possible) and overhead protection provided on working platforms.

During design and construction, health and safety risk assessments and registers should be prepared and maintained. These will become live documents until the completion of the works. Health and safety risk register should be regularly reviewed to determine the status of the risks (i.e. are they closed, mitigated sufficiently or has the risk increased?).

The Labour Department Website contains useful information on construction site safety, some of which is related to tunnel and cavern works. Further guidance on the health and safety aspects of cavern developments can also be found in:

(a) BS 6164 – Code of practice for health and safety in tunnelling in the construction industry (BSI, 2011), and

(b) A code of practice for risk management of tunnel works (ITIG, 2012).

### 7.12.2 Fire Hazard

Fire is a major hazard in an underground excavation and can cause fatalities through heat and smoke inhalation. The use of combustible material underground should be controlled using a tagging system (which should include information on the location of the material in an excavation) and only a small amount of such material should be permitted underground at any one time. To deal with fire incidents underground, it is advised that, in addition to self-rescuers being used by all underground workers, a refuge chamber is provided close to working faces. The refuge chamber will have its own air supply and other safety provisions such as first aid equipment and water. Guidelines for provisions of refuge chambers are given in ITA (2014).

Access of workers should also be restricted and a tagging system adopted to track which workers are underground. In large cavern complexes, tag boards can be split into different areas to make it easier to locate workers. There are now electronic tags available that enable the control room to check the location of all personnel on a computer display. A wireless radio system with handheld transceivers (i.e. walkie talkies) can also be used so the control room can be in continuous contact with supervisors and other designated personnel.

### 7.12.3 Ventilation

Ventilation of caverns during construction is an essential item for the working environment. In Hong Kong, the requirements of the Hong Kong Occupational Safety and Health Ordinance as well as the Factories and Industrial Undertakings Ordinance should be followed when designing a ventilation system. Following the code of practice, an adequate quantity of fresh air should be introduced into access tunnels and caverns to remove pollutants from diesel powered plant and blasting operations. In addition, a ventilation system should be compliant with the relevant air quality standards in Hong Kong, in particular those related to working in confined spaces. BSI (2011) provides airflow and ventilation requirements in tunnel construction, which can be used as a reference. Air quality should be monitored to make sure the quality is sufficient and that there is not a build-up of potentially dangerous gases (e.g. nitrogen oxide, methane and radon).

The most common way to ventilate a tunnel and cavern system to ensure fresh air at working faces is to use external fans to blow fresh air through rigid or flexible ducts. This force exhaust air to the surface through access tunnels. The advantage of this method is that fresh air can be cooled and delivered directly to a face, but the disadvantage is that the polluted air is then driven through the access tunnels and discharged from the portal. Another option is to provide an extraction system using a tunnel or a shaft equipped with extraction fans providing fresh air to a working face.

Hong Kong has a sub-tropical climate and in summer the air temperature is generally over 30°C and humidity can be over 90%. This should be considered when planning a ventilation system, as otherwise air forced underground may not be sufficient to maintain an acceptable ambient temperature in a tunnel. Chiller plants have been installed as part of the ventilation system for several tunnelling projects in Hong Kong to reduce the temperature and humidity of the clean air forced underground.

As detonated explosives release significant quantities of toxic and asphyxiating gases, the time required to reduce sufficiently the amount of blast gases with forced ventilation so that workers can re-enter an excavation can be considerable. This can lead to significant impacts on the construction programme and cost, particularly for large caverns. Mitigating measures may have to be introduced, which may include reversing the ventilation direction by using extraction fans mounted at the inner end of a flexible ducting, providing rigid ducting with external exhaust fans and constructing tunnels and shafts to aid ventilation. In the interest of the overall project economy, such tunnels and shafts should preferably have a permanent use. The advantage of exhaust fans is that polluted air can be contained within the duct and can be more easily treated, if required, before being emitted to the atmosphere. However, the disadvantage of using exhaust fans is that fresh air cannot be cooled and delivered directly to a face, which can result in uncomfortable working conditions.

Gas detectors should be used to test air at the start of each shift and after each blast. Gas detectors can be portable or fixed. In addition, due to the nature of the ground in Hong Kong, radon can be emitted and therefore detectors should be used to monitor the quantity of radon underground and ensure ventilation is sufficient to keep radon levels to an appropriate limit (see Section 9.3.8).

To prevent significant quantities of dust being introduced into a ventilation system, damping of the excavated spoil prior to an excavation and transportation is required.

# 8 Portals, Access Adits, Tunnels and Shafts

## 8.1 Introduction

Portals, adits, tunnels and shafts form the accesses between cavern structures and the ground surface. Although adits, tunnels and shafts are often of relatively smaller sizes compared to the main cavern structure, their design and construction can be complex because excavation in soft ground near the surface, hard rock close to the main cavern structure, as well as mixed ground in the transition zone between them would be involved. The consequence of collapse of these access structures can be serious due to their proximity to the existing surface structures. While guidance on design of cavern structures and support elements given in Chapter 6 is generally valid for these auxiliary access structures, this chapter provides supplementary guidance pertinent to the specific considerations and issues for their design and construction. When considering a portal or shaft layout there may be other associated cavern facility structures or buildings that need to be provisioned external to the cavern facility. Space proofing and adequate provision of these may need to be incorporated into the local site formation design at the portal or shaft locations. Section 8.2 pertains to portals, adits and tunnels which form the sub-horizontal accesses to caverns; and Section 8.3 covers similar topics for shafts, i.e. vertical or sub-vertical accesses to caverns.

## 8.2 Portals, Adits and Tunnels

### 8.2.1 Design Considerations and Approach

Construction of a portal often involves formation of cut slopes and retaining walls. The design of the site formation should be carried out in accordance with the following technical guidance documents and other relevant de facto geotechnical standards given in GEO Technical Guidance Note No. 1 (GEO, 2012).

(a) Geotechnical Manual for Slopes (GEO, 1984),

(b) Geoguide 1: Guide to Retaining Wall Design (GEO, 2017d),

(c) Geoguide 6: Guide to Reinforced Fill Structure and Slope Design (GEO, 2017e),

(d) Geoguide 7: Guide to Soil Nail Design and Construction (GEO, 2017f), and

(e) GCO Publication No. 1/90 (GCO, 1990).

In cases where the portal is potentially affected by natural hillside landslides, a natural terrain hazard study should be carried out in accordance with Ho & Roberts (2016). Landslide hazard mitigation measures should be implemented as appropriate to protect the portal.

It is desirable to optimise the extent of the site formation as much as practically possible to reduce disturbance to the environment and visual impact. The use of vegetation to shield the portal may further minimise any visual impact.

Slopes in Hong Kong very often have a thick layer of regolith. Consequently, the excavation of adits may involve a substantial length of soft ground tunnelling and a transition from soft ground to hard rock. The design should ensure stability of the ground under these ground conditions with appropriate support measures, such as spiling or canopy tubes and steel arches or lattice girders and shotcrete, as mentioned in Chapter 6.

The hydraulic conductivity of soft and mixed ground is generally much higher than that of a competent rock mass in Hong Kong. Perched or main groundwater tables are often intercepted by adits. Measures such as pre-excavation grouting should be designed to prevent excessive water ingress and associated instability which may cause undue ground movements. Particular attention should also be paid to assessing the invert stability of adits/tunnels in soft ground if a large piezometric head is present.

Portals located at low points or near watercourses are prone to directing surface runoff into the caverns and cause flooding. Appropriate measures, such as drainage diversion, drainage interception, and raised entrance, should be designed with adequate robustness to alleviate the risk of flooding.

Earthquake hazards, with reference to the considerations provided in Section 6.8.2 (6), should be considered in portal design. The seismic load to be considered may be determined based on a site-specific seismic hazard assessment.

### 8.2.2 Construction Considerations

Formation of portals and adits in soft ground is usually executed by mechanical excavation. Rock may be encountered during excavation. More efficient methods for rock excavation such as blasting should be considered, subject to the results of a blasting assessment. Other methods may be cost-effective if the quantity of rock is not large. These include hydraulic breaking, drill and split or chemical swelling. Chemical swelling holes should be checked to make sure that they have split the rock and have been exposed so that no further movement would continue that may cause ground instability in future.

The construction of adits and tunnels most likely involves both soft ground and hard rock tunnelling/shaft sinking methods. In addition to the methods of hard rock excavation described in Chapter 7, the use of tunnel boring machines (TBMs) should be considered if there is a significant length of adits or tunnels of uniform size to be excavated. This is particularly the case if the ground conditions are weak.

For portals affected by potentially unstable slopes or boulders, stabilisation and mitigation measures such as rockfall fences, debris-resisting barriers and canopy structures may be required. Some of these measures will need to be installed before the start of the portal and shaft works to provide a safe working environment.

In Hong Kong, the zone of interface between soil and rock can be water-bearing and extra care should be taken to ensure enough advance probing is carried out. If necessary, pre-excavation grouting should be conducted to control excessive water ingress and related stability problems. For the majority of cases in Hong Kong, adits would be inclined upwards in order to allow gravity drainage of the groundwater and waste water used in a cavern or facility. Quite often niches are formed at the sides of adits to locate electric substations, sumps and other associated features. For those adits that cannot be inclined upwards, adequate provision of sump pits and temporary water storage at the lowest point of the inclined adit would need to be provided.

## 8.3 Shafts

### 8.3.1 Design Considerations and Approach

Shafts are required for a variety of purposes including ventilation, access and process requirements. Similar to adits, excavation of shafts often involves a transition from soft ground to hard rock excavation. Adequate lateral support should be designed to retain soft ground without causing undue settlements. Guidance on the design of temporary and permanent retaining structures is given in GEO Publication No. 1/90 (GCO, 1990) and Geoguide 1: Guide to Retaining Wall Design (GEO, 2017d) respectively.

When a shaft is constructed in soft ground, the excavation should be designed against ground movement that may cause undue movements and straining of existing structures in the proximity. Settlements due to depression of groundwater pressure as a result of inflow into the shaft should be duly considered.

The design methodology for caverns is generally applicable to the design of the rock portions of shafts. In cases where empirical methods are used, suitable adjustments to the rock mass classification systems should be adopted to cater for sub-vertical orientations and geometries of shafts. For the Q-system as an example, adjustment to the Q-value for wall support and the excavation support ratio (ESR) for shafts are given in Barton et al (1974) and NGI (2015). An ESR of 2.5 for circular shafts and 2.0 for rectangular or square shafts are recommended. However, an appropriate value should be used depending on the purpose, importance and service life of a shaft.

The empirical method presented in McCracken & Stacey (1989) may be used for estimating raisebore quality of the rock mass, and to confirm the feasibility of planned raise-bored shafts (see Section 8.3.3(4) for the description of this method of construction). This empirical method was developed to assess geotechnical risk for large-diameter raise-bored shafts. It uses a raisebore rock quality ($Q_R$) index obtained on the basis of the Q-value, and a raisebore stability ratio (RSR) which is similar to the Q-system ESR. McCracken & Stacey (1989) suggested a value of 1.3 for raise-boring shafts with a medium to long term service life. RSR values of 1.6 and 2.5 have been used in two recent Hong Kong projects featuring temporary raised-bored shafts up to 3.7 m in diameter with a permanent cast-in-situ concrete lining constructed subsequently.

Peck et al (2011) presented various case studies for a range of rock types, rock mass conditions and shaft diameters, including summary plots correlating raise diameter, lower bound $Q_R$ values and actual performance of the shafts. The rolling average of rock quality over 3 m down-hole increments was used for the empirical assessment method proposed by McCracken & Stacey (1989).

For excavation in rock using conventional top-down excavation methods, inspection of the excavation face and installation of necessary rock supports are included as part of the excavation cycle. In contrast, for bottom-up methods such as raise boring (Section 8.3.3), there is no access to the excavation face, which precludes the installation of rock support measures during the shaft excavation. Adequate pre-excavation ground improvement by grouting should be in place if any zone of weakness is anticipated. Grouting may be injected through an array of holes drilled around and within the area of concern using e.g. tube-à-manchette. Confirmation boreholes and testing should be carried out to verify that the ground improvement has been carried out as design intended.

Significant groundwater drawdown and ground settlement may be induced by shaft construction unless adequate mitigation measures are implemented (Kwong, 2005; Pakianathan et al, 2003). Considerations of allowable shaft inflows and induced surface settlement may also influence the selection of shaft permanent lining type, and the potential need for pre-excavation grouting around shafts. Particular attention should be given to the connection between shafts and caverns (or tunnels), which may require robust pre-excavation grouting, additional temporary support to prevent overbreak during back reaming, and the provision of a collar beam for permanent support, depending on the relative shaft size and actual rock mass conditions.

The guidance on natural terrain hazards, risk of flooding and earthquake hazards for portals given in Section 8.2.1 is also applicable to the design of shaft entrances.

### 8.3.2 Ground Investigation

This section supplements the guidance given in Chapter 5, focusing on the requirements for shafts constructed by raise boring method (as described in Section 8.3.3(c)).

Ground investigation drillholes may be best located close to a shaft but not within the plan area of the shaft if it is planned to be constructed using the raise-bore technique. This is because vertical divergence of ground investigation drillholes, which if located in the plan area of the shaft, may also cause the subsequent pilot hole for the raise bore to deviate and follow the incorrect alignment of the ground investigation drillhole.

Any ground investigation for shafts should employ rotary coring techniques with frequent permeability tests, discontinuity surveys and checks on drillhole verticality. Detailed and careful logging of rock cores by experienced engineering geologists is essential to facilitate the assessment of rock mass quality and the future stability of the shafts, in particular for raisebore during back reaming.

### 8.3.3 Methods of Construction

(1) *General*. Shafts may be entirely in hard rock or formed through soft ground at the surface. Construction methods for soft deposits and residual soils are not covered in this document. The choice of excavation method depends primarily on the diameter and length of a shaft and accessibility to its top and bottom. The site geology may also influence the choice, but is usually of secondary importance.

Depending on the method of excavation, the unit cost for excavating shafts can be substantially higher than that for tunnel construction. This is mainly due to the increased powder factor for blasting the same amount of rock as mucking out to the surface is slower in top-down shaft blasting than in tunnelling. Consequently, the rates of advance for large diameter shafts are lower than that for the same cross-sectional area of a tunnel.

Table 8.1 gives typical shaft diameters and lengths achieved by common methods of construction. The requirement for access to the lower end of a shaft, to remove spoil from the shaft excavation is common to all cost-effective methods of shaft excavation. Bottom access should be provided wherever possible.

**Table 8.1 Typical Shaft Dimensions**

| Method | Typical Length (m) | Typical Diameter (m) | Typical Inclination (°) | Access |
|---|---|---|---|---|
| Shaft Sinking | < 1,000 | ≥ 5 | 90 | Top only |
| Drilling | 150 - 2,000 | 1 - 12 | 70 - 90 | Top only |
| Raise Boring | < 1,000 | 1.2 - 8 | ≤ 90 | Top and bottom |
| Pilot Shaft | < 400 | ≥ 8 | ≥ 45 | Top and bottom |

For deep shafts, particular attention should be paid to ensure that the specified allowable vertical tolerance of a shaft matches the plan dimensions of the intersecting underground opening at its base.

Detailed and specific risk assessments are essential for all stages of a shaft construction.

(2) *Conventional Shaft Sinking*. This is one of the most common methods of shaft sinking and in hard rock, the drill-and-blast method is most commonly used. Shaft diameters vary from 5 m upwards and can be excavated to any length. The top-down method requires a shaft sinking infrastructure to be situated at the surface. This includes a crane or gantry which is the main form of haulage to remove rock and lower down equipment. Scaling and rock support should be carried out successively before the blasting of the next round. The drawback of this method is that it is difficult to clean the floor before drilling the next round and pumping is required to keep the shaft floor relatively dry. The powder factor for this type of blasting is higher than that for the pilot shaft method because the blast initially needs to create a void for the remainder of the shaft to break into. One possible solution is to drill large diameter holes from the surface and fill them with sand. These can then be washed out and used as a free face for the rock to break into when required.

Where top-down construction using blasting is adopted for shaft excavation, vibration and air overpressure need to be carefully considered. Water ballast has been used to address this issue as described in Holowenczak et al (2013).

Near the surface, shafts may initially be excavated through soft ground before the bedrock is reached. There are various types of shaft excavation support that may be adopted in the soft ground section, from sheet piles, pipe piles, secant piles to diaphragm walls depending on the depth and types of ground supports needed. Temporary support works need to be designed to withstand subsequent blast vibrations as necessary. Excavation through rock may also be carried out using a mechanical breaker, the drill and split method or chemical swelling agents. However, these methods are extremely slow, costly and should be avoided wherever possible.

The shaft being excavated needs to be of sufficient size to allow accommodation of the equipment for drilling and excavating as well as the ancillary equipment such as ventilation, water compressed air pipes and a ladder way to provide a means of egress.

(3) *Drilling.* Shafts may be drilled from the surface using large drill rigs. Depending on the types of machine, shaft diameters up to 12 m are possible. This method is usually slow and expensive, but there are mechanised shaft-sinking machines that are especially dedicated to sinking blind shafts using the top-down method in different ground conditions. Some shaft-sinking equipment are also able to install segmental linings to support the side walls of a shaft during the excavation process, similar to that of a TBM. However, as yet, this method has not been used in Hong Kong.

(4) *Raise Boring.* The raise-boring method, or reaming method, is commonly used for small and medium diameter shafts. Its use is best suited to relatively competent rock masses requiring minimal support. Both vertical and inclined shafts may be formed with this method, which is illustrated in Figure 8.1. A pilot hole with a diameter of about 250 to 350 mm is made and the reaming head (see Figure 8.2) is fixed to the lower end of the drill rod. The shaft is cut by the reamer being rotated and pulled upwards towards the drilling rig. Shafts with diameters between 1.2 and 8 m and lengths up to 1,000 m have been formed by this method. To maintain verticality of the pilot hole and subsequent shaft, a very precise set-up and accurate surveying of the raise bore rig, and careful drilling and reaming are required. For long shafts, the pilot hole should be surveyed frequently and wedges or a steerable bit used to control deviation. When the pilot hole is completed, if groundwater discharge exceeds the design requirement, then grouting may be required prior to back reaming. This method requires both top and bottom accesses, but the top access only needs to provide sufficient space for the drill rig, reamer, drilling mud tanks and power supply. The resulting shaft is smooth and seldom requires scaling or support, but this should be checked after the raise bore has been completed.

Raise boring is one of the safest shaft excavation methods as it does not require man access to the excavation face. It achieves a good rate of progress even in hard rock as demonstrated by previous projects in Hong Kong (Aldridge et al, 2009; Evans et al, 2012). It is recommended that these types of works be carried out by a specialist raise-boring contractor with appropriate equipment and experience of raise boring in similar ground conditions at the depths and shaft diameters under consideration.

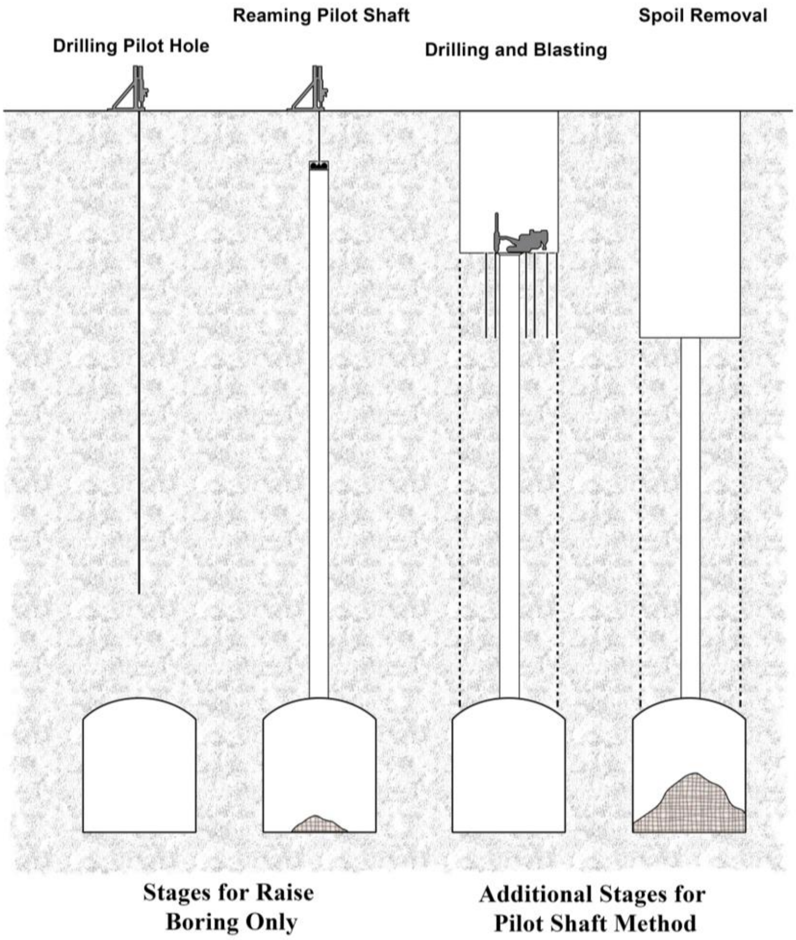



**Figure 8.1 Shaft Excavation by Raise Boring and Pilot Shaft Methods**

**Figure 8.2 Installation of Raise Bore Cutter-head Underground (courtesy of ARUP)**

A recommended option for inspection of a completed shaft is to lower a CCTV camera from the surface, or top of the shaft, under the direction of an experienced engineering geologist, to check for any instability on the walls of the shaft. If necessary, shotcrete may be sprayed onto the walls of the shaft with a shotcrete robot lowered from a mobile crane. After confirming that the shaft is safe for inspection, an engineering geologist may then inspect the shaft from a man cage and recommend further stabilisation measures as necessary.

The amount of time saving depends on the accuracy of the pilot hole. However, the alignment of the pilot hole should be carefully controlled, especially in mixed ground conditions where significant deviation from the intended alignment is not uncommon. This may cause significant delay in construction and the advantage over conventional methods would be greatly reduced.

The required works area for the raise-bore rig, requiring power pack and accessories may be smaller than that for a traditional top-down shaft excavation spoil handling area at the surface. In addition, underground spoil removal may result in significantly less impact on the surrounding environment and the local traffic than that from the conventional shaft. The raise boring method also allows easy removal of underground spoil through a tunnel or cavern network.

(5) *Pilot Shaft Method.* For large vertical shafts with restricted space at the surface, it is common to adopt the pilot shaft method. The pilot shaft is first excavated to provide a free face and act as a chute for the excavation spoil to fall down into the cavern for removal during subsequent rock excavation to the full shaft diameter using the top-down method. The safest way to excavate the pilot shaft is by raise boring. The pilot shaft should be of a minimum plan area of 5 $m^2$ to allow smooth mucking down the pilot shaft. The size of the blasted rock should be carefully controlled to avoid blocking of the pilot shaft. Scaling and installation of rock support should be carried out before commencement of the next round of blasting. Large-diameter shafts may be efficiently excavated by this method which is described in Figure 8.1.

# 9 Quality Control, Monitoring and Maintenance

## 9.1 Introduction

Quality control and monitoring is an essential part of the construction process to ensure that a cavern development is constructed in accordance with its design. Underground works have to be monitored in the same way as any other constructed facilities.

Monitoring does not, in itself, ensure good performance. Performance is affected fundamentally by the design and construction. Quality control and quality assurance are therefore needed for each stage of a project development. Monitoring is the final stage of this control.

Instrumental monitoring of caverns may be required for engineering reasons and to give confidence in the design, construction and performance of the facility. Monitoring by means of periodic visual inspection will suffice for most occupied caverns.

The geotechnical performance review carried out, together with the monitoring/survey data and the factual geological and hydrogeological data, such as field data collected from the excavation faces and monitoring data obtained for tunnel works, should be properly documented. For public works projects, the information should be submitted to the GEO for record.

Where long-term monitoring of any type is required, details of the long-term monitoring should be documented in a maintenance manual. Relevant construction details viz. support works and drainage should be included. As-built records including information related to testing and verification during construction should also be kept and recorded. The need to provide 3D as-built models including integrated civil, structural and E&M provisions may be considered to assist in future maintenance and operational planning. Physical monitoring and control systems for underground space may also be integrated into building information modelling (BIM) technologies.

A monitoring plan should be devised as part of design of cavern structures. The scope and requirements viz. frequency, period and instrumentation of the monitoring plan should be continuously reviewed and enhanced subject to the actual ground conditions.

## 9.2 Quality Control

Quality control and assurance routines should be instituted for all stages of a project development from feasibility study to construction and operation. Independent detailed checks of engineering drawings, analyses and reports for the feasibility study need to be carried out. Formalised routines may be required for the detailed design and tender document production. The site investigation and construction phases will require specification of ground investigation and laboratory tests, acceptance tests for components of the construction, e.g. rock bolts and shotcrete, and inspection and acceptance of the works as set out in conventional specifications. In addition, quality assurance routines should be implemented to ensure that the specified quality is achieved.

Of particular importance in underground projects are the works needed to ensure adequate stability of the excavation. Necessary excavation supports must therefore be installed in a manner that will ensure efficacy and durability. Suitable acceptance criteria for both should be specified.

In Hong Kong, the process of project management for government projects is set out clearly in the Project Administration Handbook for Civil Engineering Works (PAH) (HKSARG, 2016) or its latest edition in which the various standards and requirements in the management process are outlined. Generally the quality control requirements for a cavern development require that the consultants, contractors and suppliers have effective quality assurance processes and procedures in place in accordance with quality management standards e.g. ISO 9001 (BSI, 2009). When devising quality control and monitoring plans, Geotechnical Control for Tunnel Works TC(W) No. 15/2005 (ETWB, 2005), Code of Practice for Safety in Tunnelling in the Construction Industry (BSI, 2011) and the latest version of the PAH should be referred to.

Supervision of the works by experienced and competent engineers and engineering geologists is essential in order to ensure that the specifications, standards and drawings are followed and adhered to. It is usual to set up site supervision plans and procedures to ensure that the works are constructed according to the design.

In underground excavation works, the process in supervising and designing excavation supports should be developed to allow for sufficient flexibility in the design and construction to handle the actual ground conditions encountered on site. The quality assurance scheme should allow and outline how design changes during construction are managed.

## 9.3 Monitoring

### 9.3.1 Planning of Monitoring Programmes

All cavern installations require monitoring by direct observation and by the installation of instrumentation. Planning of a monitoring programme should address the following items:

(a) objectives of the monitoring programme,

(b) parameters to be monitored,

(c) monitoring method and instrumentation,

(d) monitoring period and frequency,

(e) response values and action plans, and

(f) procedures of implementation and assignment of duties.

### 9.3.2 Objectives of the Monitoring Programme

The purpose of monitoring is to confirm compliance with design and specifications, to verify design assumptions and adequacy of design, to reveal developing abnormal or adverse conditions and to identify the effects on sensitive receivers, thus allowing timely corrective action. There may also be a need to install instruments that monitor long-term performance of the ground and supports of cavern and important drainage measures.

Monitoring also serves to achieve design optimisation. Installing some instrumentation, such as extensometers, even in relatively simple cases may still be of significant benefit to gain confidence in some aspects of design, for instance the use of rock bolts and shotcrete as permanent supports, or deciding on the value of the "rock mass load" that should be assumed for in-situ concrete linings in the long term. Thus, at a relatively negligible cost of instrumentation and monitoring, the ability of a rock mass to act as a structural material is proven and utilised, any concerns are mitigated and a more economic and sustainable engineering solution may be realised.

### 9.3.3 Parameters to be Monitored

Table 9.1 gives a list of parameters for reference. The table is not intended to provide an exhaustive list of parameters for monitoring, which should be selected on a project by project basis.

**Table 9.1 Examples of Parameters to be Monitored**

| Purpose | Parameter |
|---|---|
| To verify adequacy of spot bolts | Load in the spot bolts (where some tensioning has been specified)<br>Movement of rock blocks/wedges |
| To detect abnormal deflections of cavern | Convergence<br>Crown movement |
| To monitor effects on buildings or sensitive receivers within influence zone | Vibration of building or sensitive receiver (during blasting)<br>Building settlement<br>Tilting of buildings<br>Width and length of existing cracks on buildings, if any<br>Subsurface ground movement<br>Groundwater/piezometric level<br>Groundwater inflow rate into caverns |

Parameters to be monitored should include deformation of rock masses within a cavern with clearly defined failure mechanisms, such as major unstable wedges. Appropriate surveillance on the adequacy of related stabilising measures should also be in place.

Monitoring of sensitive receivers (e.g. structures and utilities) affected by a cavern development is also of paramount importance. Relevant parameters include settlement, width and length of existing cracks, ground vibration and blast air overpressure.

Where groundwater seepages may affect the durability of permanent supports, the aggressiveness of water and the local stained or saturated lining should be monitored to ensure that weakening and alteration of the permanent supports has not occurred.

Monitoring of both cause (e.g. groundwater inflow rate) and effect (e.g. change in piezometric head in soil strata) should be specified as far as practicable, so that the monitoring data provides a means to identify the relationship between the two. This assists to formulate appropriate actions to rectify the adverse effects by removing the cause.

### 9.3.4 Monitoring Method and Instrumentation

(1) *General*. The majority of caverns in Hong Kong will probably be constructed in hard rock at shallow depth. The most likely cause of problems encountered may be a progressive loosening of isolated blocks of rock. The actual failure of rock blocks tends to be near-instantaneous. Instrumentation is not suited to detecting this type of failure, but visual examination of the bare rock or the shotcreted surface can give warning of developing problems.

Visual inspection must be conducted regularly during cavern construction. Similarly, where drip screens have been installed, the tops of the screens and guttering should be checked for any fallen materials. In the event that any such material is identified, an inspection of the roof above the screens should be called for and remedial measures instituted as necessary.

Instrumental monitoring should be adopted for large span or high caverns and locality of stability concern, including locations of shallow rock cover and weakness zones. This also applies to critical support installations which have been designed with field data collected early in the design or construction period.

The instrumentation adopted should be kept simple and robust. The method of measurement should also be simplified with an appropriate accuracy, precision and repeatability. A suitable amount of redundancy should be allowed in the monitoring programme for cross-checking purposes. This also provides backup data in the event of malfunction of instrumentation.

Many different methods of monitoring are potentially applicable to cavern developments, in particular surface settlement, ground movement such as convergence/extension, building movement, groundwater variation/impacts, construction vibrations (mechanical & blasting), air overpressure, air quality, etc. Figures 9.1 and 9.2 show schematic layouts of the instrumentation that may be used for monitoring underground excavations. Table 9.2 provides further details on the numbered items in the figures.

Monitoring method and instrumentation for displacement measurement, stress/load measurement, shotcrete lining and prestressed rock anchors are detailed as follows.

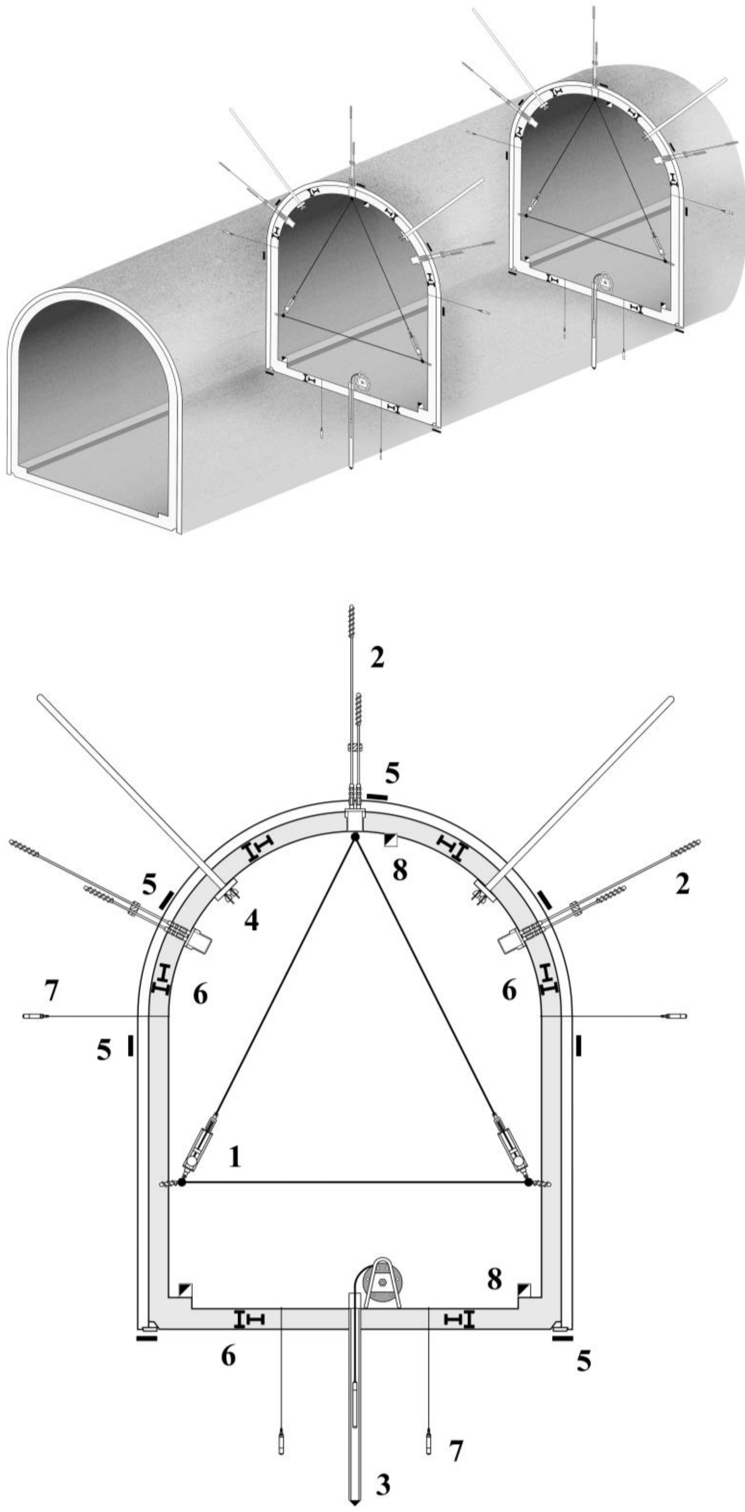



**Figure 9.1 Schematic Layout of Typical Monitoring for an Underground Excavation – Isometric View and Cross-section**

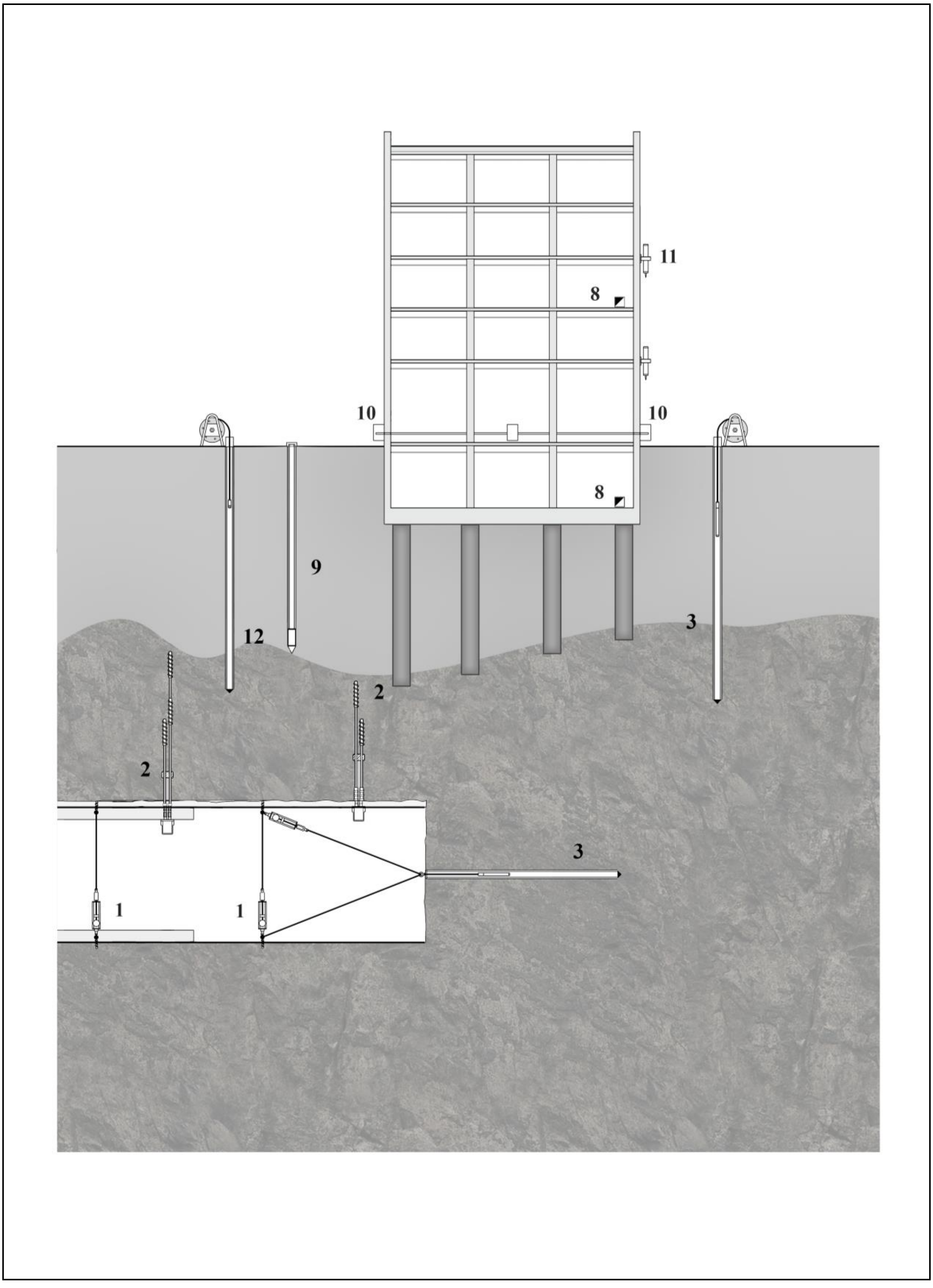



**Figure 9.2 Schematic Layout of Typical Monitoring for an Underground Excavation – Longitudinal-section**

**Table 9.2 Typical Set of Monitoring for Underground Excavations**

| Ref. No. in Figures 9.1 and 9.2 | Instrument | Application |
|---|---|---|
| 1 | Convergence tape extensometer | Movement of underground excavation lining |
| 2 | Drillhole extensometer | Rock movement around the underground excavation |
| 3 | Incremental mobile extensometer | Strain and deformation around the underground excavation |
| 4 | Anchor load cell | Monitoring of support load |
| 5 | Pressure/load cell | Pressure measurement in underground excavation linings |
| 6 | Embedment strain gauge | Concrete strain measurements (shotcrete/concrete) |
| 7 | Electrical piezometer | Ground pore pressure control |
| 8 | Vibrograph (in building or on ground) | Ground vibration impact on sensitive receivers |
| 9 | Piezometer and ground extensometer | Water level and surface settlement control |
| 10 | Ground and building settlement markers | Monitoring of ground movement and building movement control |
| 11 | Surface and building inclinometer | Monitoring the tilt of building and lateral movement of the ground |
| 12 | Ground extensometer | Ground settlement above the underground excavation |

(2) *Displacement Measurement*. Monitoring of displacements in the surrounding rock can also be carried out by convergence measurements. The survey of reflective targets is probably the most commonly used method in Hong Kong to measure convergence as it is simple and inexpensive to carry out and can be incorporated into the ongoing surveying operations. The convergence measuring device is mounted on bolts set in the rock on opposite faces of the opening and the distance between them is measured. This device is normally an invar (nickel-iron alloy) wire and the accuracy of this type of measurement is up to ± 0.1 mm plus $10^{-6}$ of the measured distance. Convergence measurements show only the

overall shortening of the reference distance. No information is given about the degree or depth of the loosening rock.

In order to detect the zones of rock loosening around a cavern, extensometers can be placed in drillholes perpendicular to the cavern walls or roof. Usually, multiple-point extensometers with grouted anchorages are used, but single-point extensometers are also installed. Extensometers with recessed heads can be installed close to the working face and allow monitoring at a very early stage. Early monitoring can be useful in determining the required bolt lengths. Other methods that may be used include laser surveying, digital photogrammetric techniques and fibre-optic sensors.

Tell-tales may be used to detect local relative movement between adjacent blocks of rock in selected areas, should stability be of a particular concern. Levels of the land surface above a cavern may require monitoring, particularly if the cavern is at shallow depth.

It is worth noting that relaxation of the ground may occur before an instrument is installed and commissioned. This should be duly considered in establishing the triggering levels, design of the monitoring programme and review of the monitoring results.

(3) *Stress/load Measurement*. Hydraulic stress cells can be placed around the circumference of a cavern. For instance, cells can be embedded at the rock/shotcrete interface to monitor the contact stresses, and cells embedded in the shotcrete or concrete lining itself can be used to measure the radial and tangential stresses in the concrete. However, in current practice, deformation monitoring is the normal approach and the use of stress-measurement cells is typically reserved for situations where knowledge of the stress regime is important (e.g. high or unusual in-situ stress and high surface loads). The cells are generally perceived to have a number of issues including installation complexity, unreliability and inaccuracies.

Strain gauges can be used for determining the loads of structural elements. For example, they can be installed on rock bolts to identify whether any creep is occurring in the bolts.

(4) *Shotcrete Lining*. Shotcrete linings need to be inspected regularly during construction to discover sections which may have become de-bonded from the rock and which may fall with time. The visual inspection aims to identify signs of cracking and other distress. Monitoring using tapping surveys or crack markers may be carried out to check de-bonding but these should only be conducted where the integrity of the shotcrete looks generally secure. Detailed recommendations of the monitoring frequency are given in Section 9.3.5.

(5) *Prestressed Rock Anchors*. All rock anchors should be monitored in accordance with the requirements and advice of Geospec 1: Model Specification for Prestressed Ground Anchors (GCO, 1989).

(6) *Others*. Residual groundwater inflow (after rock pre-grouting) and water pressures acting on the lining are important for stability and maintenance, particularly for unlined caverns and tunnels. Monitoring and review of the residual groundwater inflow may be required if there is a genuine concern. Such monitoring should cover at least one wet

season where practical. The monitoring should be further extended if the results of the review show that there is a genuine need to do so.

### 9.3.5 Monitoring Period and Frequency

The monitoring period and frequency should be determined to establish variations prior to, during and after an underground space is created. Monitoring prior to a cavern excavation refers to baseline monitoring and attention should be made to capture the seasonal variations of some parameters (e.g. groundwater level) in specifying the monitoring period.

Rates of variation, for example in convergence, are critical in determining the stability condition. An increasing rate of deformation may indicate a potential detrimental situation. Therefore, monitoring frequency should be specified and regularly reviewed with an aim to revealing the rates of variation.

For a cavern developments in good rock mass, instrumental monitoring will not record noticeable changes in deformation immediately before and shortly after the time of excavation. Similarly, loads on support elements reach long-term values quickly. Obtaining those data requires that the instrumentation is installed in advance of or immediately after the excavation and is continuously monitored in the period leading up to the excavation and afterwards until deformations approach zero. Subsequently, the instruments should be regularly monitored but at a lower frequency.

Once this iterative design process is completed, further monitoring with instruments is not normally required, except those to review the long-term performance of the cavern structures or to assess the long-term effects of the cavern development.

### 9.3.6 Response Values and Action Plans

Response values which trigger follow-up actions should be established for each of the parameters included in the monitoring programme as appropriate. A system comprising three levels of triggering values viz. alert, alarm and action levels (AAA levels) is normally adopted in local practice. Table 9.3 presents the follow-up actions corresponding to different triggering levels.

The AAA levels should be established based on the predictions made in the design with consideration given to experience and serviceability requirements of the support works and sensitive receivers. These levels need to be selected carefully to ensure that the monitoring and alert system is effective but not too restrictive. A pre-construction condition survey and due assessments should be undertaken to confirm the serviceability requirements of the sensitive receivers.

The rates of change of displacement and the absolute displacement values are equally important in determining the stability condition. Therefore, the establishment of the AAA levels shall also consider the rate of displacement as necessary.

**Table 9.3 Follow-up Actions of Different Triggering Levels**

| Level | Follow-up Actions |
|---|---|
| Alert | • Conduct more frequent monitoring measurements and/or add additional monitoring points.<br>• Investigate cause(s).<br>• Review relevant design e.g. structural supports, groundwater inflow limit, and construction method.<br>• Assess effects.<br>• Formulate action plans to mitigate the potential detrimental effects concerned.<br>• Review reasonableness of the AAA levels. |
| Action | • Implement the action plans.<br>• Review effectiveness of the actions. |
| Alarm | • Suspend construction. |

### 9.3.7 Procedures of Implementation

The key issues related to the procedures of implementation and assignment of duties are detailed below.

(a) *Data collection and verification*. While some data may be collected manually, there have been advances in real-time and remote monitoring, including the use of fibre optics, which have many potential benefits and should be considered for the monitoring of cavern developments. The monitoring information can be centralised into a computer-based database for processing and plotting, and accessed via the internet on a real-time basis.

Verification by the project owner should also be considered. In addition, the appointment of an independent instrumentation and monitoring contractor may be considered, although this would have cost implications.

(b) *Assignment of duties and lines of reporting/communication*. The duties and responsibilities of involved personnel should be clearly defined. A reporting system should be put in place to ensure effective communication of monitoring data and risk mitigation actions.

The geotechnical risk management process does not take place in isolation from other project activities. Communication of risk information and consultation with project participants responsible for other areas of project implementation, as well as major stakeholders including the affected public are two-way processes that should be proactively undertaken and should continue throughout the duration of the project.

(c) *Data review and interpretation*. The contractor shall establish a procedure that enables prompt and regular review and effective response to the results from the instrumentation and monitoring. The temporary works designer, or a person familiar with the design assumptions and assumed sequencing of the works, should be included in the monitoring review procedure.

It is possible to use computer-based systems to compare monitoring results with the values predicted in design and consider instrumentation data with respect to the progress and the proximity of excavations (Maxwell, 2009). Such systems feature an automatic alert system which sends messages to personnel in charge of the works.

(d) *Arrangement of remedial actions*. When the response values are reached or abnormal response is observed, the pre-defined contingency or remedial action needs to be taken in a timely manner. A performance review process is normally put in place for cavern developments, based on the data from the various monitoring methods. The purpose of these processes is to make sure appropriate actions are taken on the basis of any values exceeding the AAA values and also to identify subtler potentially detrimental trends with respect to aspects such as convergence, settlement and groundwater levels. There should be a mechanism for reviewing the opportunity to reduce instrumentation and the frequency of monitoring where appropriate. Monitoring may continue for some time to allow assessment of long-term effects and to provide reassurance to clients and the public.

### 9.3.8 Other Monitoring Issues

(1) *Rock Temperatures*. Monitoring of rock temperatures for environmental reasons where a cavern is being operated at temperatures substantially different from ambient, such as cold stores, may be required. This monitoring would have little influence on design or operation, but may be required to address proprietors' concern and may provide useful data for future schemes.

(2) *Radon and Other Gases*. Radon gas is formed from decaying uranium and/or thorium and is soluble in water from which it can be released into the air. Some Hong Kong rocks contain these radioactive materials. Therefore, if there is a significant amount of water inflow and the ventilation is inadequate, there may be a risk of radon gas being found in the underground workings at elevated levels (Robertshaw & Tam, 1999). 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. Existing rock caverns in Hong Kong occupied by workers are monitored and the results have indicated that there are no serious levels of the gas encountered under normal ventilation provisions. Nevertheless, radiation levels should be monitored after blasting during construction. Testing for radon should be conducted periodically to check the limits of radon gas in the underground workings (BSI, 2011).

The World Health Organization (WHO, 2009) recommends to limit the radon gas concentration to a level of 100 Bq/m$^3$. Wherever this is not possible, the WHO suggests that the chosen level should not exceed 300 Bq/m$^3$. The Norwegian Radiation Protection Agency recommends that in rooms for habitation, mitigation should be made when the radon level is higher than 100 Bq/m$^3$ and that a maximum level of 200 Bq/m$^3$ should not be exceeded. In the United Kingdom, the action levels are 200 Bq/m$^3$ for homes and 400 Bq/m$^3$ for workplaces.

It is important that regular monitoring of other gases is carried out, particularly in confined spaces and where ventilation is poor:

(a) oxygen (should be in the 19.5 to 23% range, average be 21%),

(b) flammable gases (including methane), and

(c) harmful gases (including hydrogen sulphide and carbon monoxide).

The above gases can be generated from natural sources, construction activities, specific operations within caverns which emit gases and contamination from nearby sources such as landfills, petrol stations and stores of hazard materials, etc. A practical and effective way of ensuring good air quality is to provide adequate ventilation as shown by local and overseas experience. If necessary, risk assessments can be carried out to identify sources and appropriate mitigations of hazard gases.

### 9.4 Maintenance of Cavern Structures

The designer of a cavern facility should specify upon completion of the construction any features of concern or items that should be maintained or reviewed regularly. This may include left-in-place monitoring to assess the overall performance of the facility or features encountered that need to be checked. An as-built cavern performance manual should be handed over to the operator for subsequent follow-up.

Only minor maintenance works to engineered rock caverns are anticipated. The purpose of the maintenance works is confined to ensuring the physical integrity of the cavern structures.

The maintenance of structural linings within caverns should follow existing maintenance practice pertinent to other ordinary buildings. Owners or maintenance agents of caverns can consult a Registered Professional Engineer in the structural, civil or geotechnical discipline for advice.

For unlined portions of caverns or portions covered by a non-structural lining, the scope of the maintenance works would be similar to rock slopes. Relevant maintenance requirements are detailed in Geoguide 5 (GEO, 2003). Suitable personnel, as stated in Geoguide 5, should carry out routine Maintenance Inspections and Engineer Inspections. The inspections can be undertaken from a vantage point, possible with a pair of binoculars (Section 4.5 of Geoguide 5). Permanent access provisions (e.g. stairways) for inspections are not required. It is recommended that Routine Maintenance Inspections and the Engineer Inspections should be undertaken annually and every five years respectively.

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# Appendix A

# Examples of Cavern Uses in Hong Kong and Overseas

# Contents

# List of Tables

# List of Figures

## A.1 Hong Kong

In Hong Kong two of the existing MTR stations on the Island Line were built either wholly or partly in caverns (at Sai Wan Ho and Tai Koo, Figure A1). In addition, four recently completed stations were built in caverns, two of these are on the South Island Line (at Admiralty, Figure A2, and Lei Tung) and two on the Island Line (at the University of Hong Kong and Sai Ying Pun). Two caverns were also recently built as part of the Ho Man Tin Station on the Kwun Tong Line.

A temporary explosives storage magazine for the construction of the West Island Line has also been built in a cavern (Figure A3).

Other caverns are located at the Kau Shat Wan Government Explosives Depot, Island West Transfer Station in Kennedy Town (Figure A4), the Western Salt Water Service Reservoirs at the University of Hong Kong (Figure A5), Stanley Sewage Treatment Works and the valve chamber in the Western District Aqueduct.

As can be seen, the primary use of caverns in Hong Kong to date has been rail-related, with other important uses being water-related and for explosives storage. Table A1 provides details of the caverns that have been built in Hong Kong.

An online catalogue of existing tunnel and tunnels currently under construction in Hong Kong prepared by GEO of CEDD based on literature review, is available on the CEDD Website as a useful source of information.

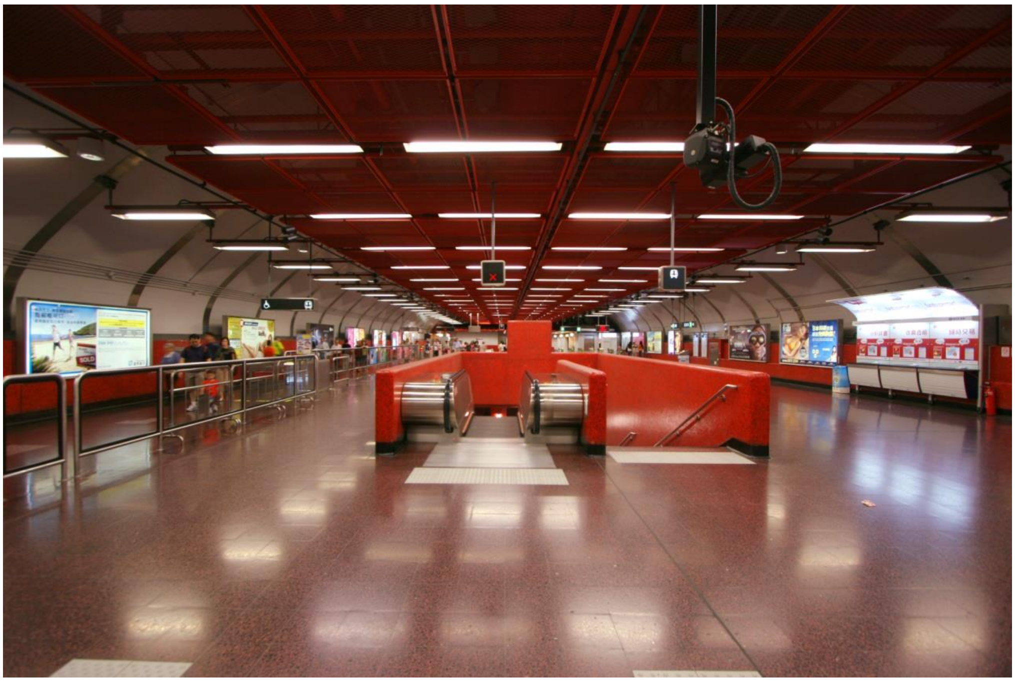
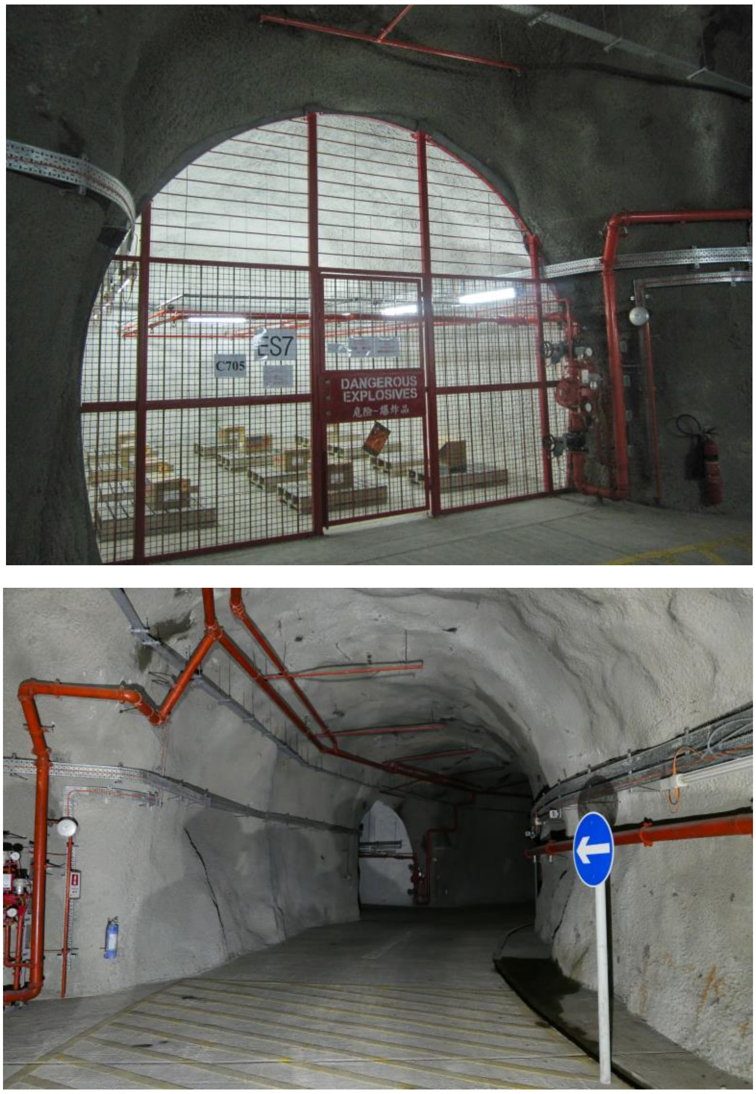


**Figure A1 Tai Koo MTR Station (courtesy of ARUP)**

**Figure A2 Artist's Impression of the New Admiralty Station Platform Cavern (courtesy of MTR Corporation Ltd.)**




(a)

(b)

**Figure A3 One of the Explosive Storage Niches (Left), and a Section of the Horseshoe-shaped Access Adit at the West Island Line Explosives Magazine (Right) (courtesy of ARUP)**

**Figure A4 The Tipping Hall at Island West Transfer Station (courtesy of ARUP)**


**Figure A5 Inside the Water Supplies Department Western Salt Water Service Reservoir at the University of Hong Kong**

**Table A1   Caverns in Hong Kong (Sheet 1 of 3)**

| Category (Use) | Location | Dimensions (in m) (W × H × L ) | Rock Type | Other Details and Reference |
|---|---|---|---|---|
| Dangerous Goods (Explosives depot) | Kau Shat Wan Government Explosives Depot, Lantau | Chambers – 13 (W) × 8 (H) × 22 (L)<br>Tunnel – 6.5 (W) × 6 (H) × 900 (L) | Granite intruded by feldsparphyric rhyolite | Tunnel and chambers excavated by drill and blast in 1995/1996, the depot came into operation in June 1997. The tunnel is U-shaped. There are 10 chambers located on the northern side of the tunnel. A concrete magazine 10 m wide, 4 m high and 18 m long was built inside each chamber for storage. Each magazine is notionally designed to store 5,000 kg net explosive quantity (NEQ) of commercial explosives. However, in practice 3,500 kg NEQ is typically stored in each magazine to facilitate stock rotation. The chambers have sufficient rock cover to confine the effects of accidental explosion in the magazines at the ground surface. |
| | MTR West Island Line Explosives Depot, West Island Line | Chambers – 5.5 (W) × 4.2 (H) × 8.6 (L)<br>Adit – 5.5 (W) × 5.15 (H) × 333 (L) | Tuff/granite interface | Completed in 2010 and decommissioned by MTR in 2013. The facility comprises a horseshoe shaped access adit connecting nine separate chambers. Eight of the chambers were used for explosives storage and one was used for detonator storage. Each of the explosive storage chambers was designed to hold approximately 300 kg of explosives during the magazine operation. The chambers had sufficient ground cover to provide for safe storage of the explosives. While the facility has been decommissioned, it has not been backfilled for future use. |
| Transportation (Railway station) | MTR Tai Koo Station, Island Line | 24.2 (W) × 16 (H) × 251 (L) | Granite | Completed in 1985. Shortly after construction of the reinforced concrete lining, the overburden was reduced from a maximum of 80 m to about 11 m by site formation works for the Kornhill development (Sharp et al, 1986). |

**Table A1 Caverns in Hong Kong (Sheet 2 of 3)**

| Category (Use) | Location | Dimensions (in m)<br>(W × H × L ) | Rock Type | Other Details and Reference |
|---|---|---|---|---|
| Transportation (Railway station) (cont'd) | MTR Sai Wan Ho Traction Substation, Island Line | 16.2 (W) × 13.8 (H) × 18.5 (L) | Granite | Completed in 1985. Adjacent to the MTR Sai Wan Ho Station which was built in an open cut. |
| | MTR Sai Ying Pun Station, Island Line | 22.8 (W) × 16 (H) × 187 (L)<br>3 m thick rock pillar between platform tunnels | Granite | Completed in 2015. The cavern is located approximately 40 to 65 m below ground level. |
| | MTR Hong Kong University Station, Island Line | 22.4 (W) × 16 (H) × 240 (L) | Granite | Completed in 2014. The cavern is located approximately 48 to 80 m below ground level. |
| | MTR Admiralty Station, South Island Line | 24.3 (W) × 14.6 (H) × 112 (L) | Granite | Completed in 2016. The cavern is an extension of the existing Admiralty Station which is located beneath the existing platforms. |
| | MTR Lei Tung Station, South Island Line | 19 m (W) × 15 m (H) × 160 m | Tuff | Completed in 2016. The cavern is located approximately 38 m below the surface. |
| | MTR Ho Man Tin Station, Kwun Tong Line | Large cavern – 22 (W) × 16.7 (H) × 33-51 (L)<br>Small cavern – 21.6 (W) × 11.8 (H) × 44-37 (L) | Granite | Completed in 2016. Two caverns constructed at Ho Man Tin Station as part of the Kwun Tong Line Extension project. |

**Table A1 Caverns in Hong Kong (Sheet 3 of 3)**

| Category (Use) | Location | Dimensions (in m) (W × H × L ) | Rock Type | Other Details and Reference |
|---|---|---|---|---|
| Water (Valve chamber) | WSD, Western District Aqueduct | 10 (W) × 8 (H) × 29 (L) | - | Completed in 1984. |
| Waste (Refuse transfer station) | EPD, Island West Transfer Station (Kennedy Town) | 27 (W) × 11 (H) × 66 (L) | Tuff | Completed in 1997. Two different sized caverns offset vertically, specified dimensions are of largest cavern (tipping hall). |
| Water (Service reservoir) | WSD Western Service Reservoir at the University of Hong Kong | 17.6 (W) × 17 (H) × 50 (L) | Tuff with sedimentary beds | Completed in 2009. Two equal sized caverns. The design total storage capacity of the reservoirs is 12,000 $m^3$. |
| Water (Sewage treatment) | DSD Stanley Sewage Treatment works | 15 (W) × 17 (H) × 120 (L) | Granite | Completed in 1995. The sewage treatment plant is housed within three caverns, including approximately 450 m of road access and ventilation tunnels and shafts. |

**A.2 Overseas**

Table A2 provides examples of some cavern developments overseas. Two notable cavern developments in Norway and Singapore are discussed below.

The Gjøvik Olympic Mountain Hall (Figure A6), built for the 1994 Winter Olympics and completed in 1993, is currently the world's largest span cavern for public use. It is 61 m wide, 91 m long and 25 m high and uses rock bolts and shotcrete as the permanent support for the cavern. The facility is capable of accommodating 5,500 people and contains an ice rink (the home of Gjøvik Hockey), a 25 m swimming pool and telecommunication installations. A wide variety of activities and concerts have been held in the facility, including the 1995 World Short Track Speed Skating Championships, making it an excellent, well used public facility that is frequently visited by tourists. The lack of external noise to the facility makes it a good concert hall venue. The cavern also has a secondary role as a civic defence shelter. The main drivers for building the cavern were that Gjøvik is situated in a mountainous area with generally good quality rock at the surface. Furthermore, situating the ice hockey rink in a cavern avoids taking up valuable downtown property space and interfering with the town's landscape. The central location also reduces travel costs and provides a stable-year round temperature, thus reduces cooling costs.

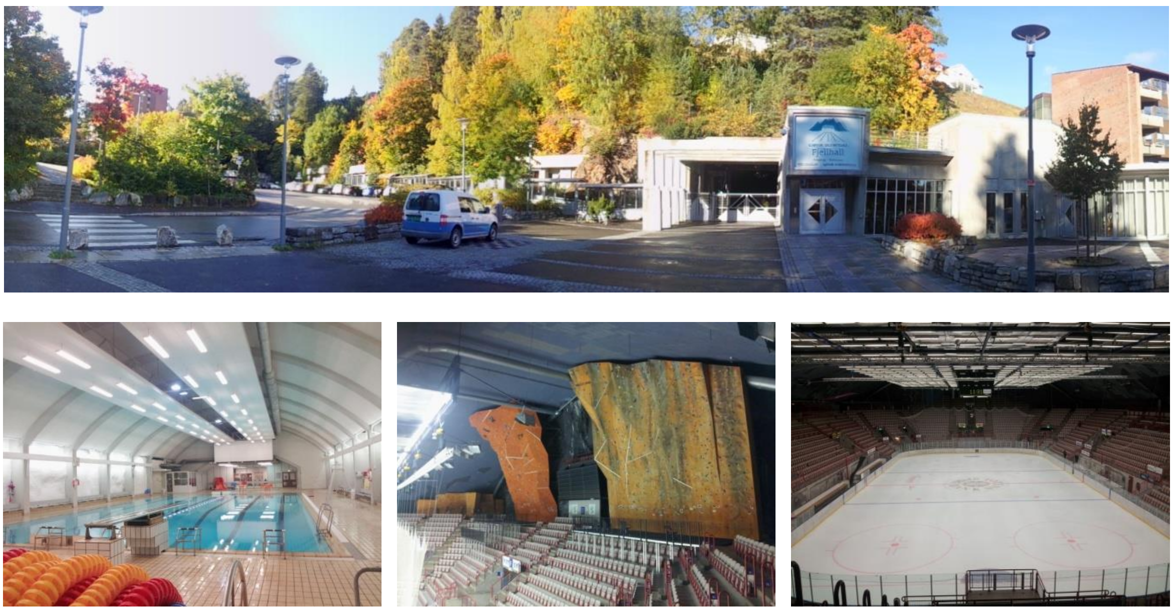


**Figure A6 The Gjøvik Mountain Hall Facility (courtesy of ARUP)**

The entrance to the Jurong Rock Caverns oil storage facility is located on Jurong Island, while the actual caverns are constructed deep below the seabed of the Banyan Basin. Liquid hydrocarbons such as crude oil, condensates, naphtha and gas oil will be stored in the JRC, which is the first such facility to be built in Southeast Asia. The project, with a total storage capacity of 2.8 million m³, will be carried out in two phases. The first phase, with a storage capacity of 1.47 million m³, includes construction of two 132 m deep vertical access shafts of diameters between 18 and 24 m. It also includes a network of horizontal tunnels of

9 km length in total which connects the five storage caverns to each other and the surface. While Phase 1 was officially opened in September 2014, Phase 2 of the caverns has been currently under planning. Refining and petrochemical plants are planned to be built on part of the 60 hectare land freed above ground. Following completion, the caverns are rented to private firms at an equivalent price to similar above-ground facilities. The primary drivers for the caverns are growing demand for oil storage and the lack of industrial land available in Singapore, combined with the Government's strategic aim of becoming Asia's oil hub.

## A.3 References

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**Table A2 Examples of Overseas Cavern Usage (Sheet 1 of 6)**

| Category | Uses | Location | Dimensions (in m)<br>(W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Archives | Archive | Elmer L. Andersen Library, University of Minnesota, USA | 20 (W) × 7 (H) × 207 (L) (both caverns) | Roof – limestone<br>Walls and base – sandstone and shale | Two caverns mined into the sedimentary strata of the river bank with curved pre-fabricated concrete ribs to support the limestone roof. Designed to withstand a 500-year flood event. Problems reported with respect to contaminated groundwater and ground gas. |
| | Archive | National Archives of Norway, Oslo, Norway | 178 (W) × 15 (H) × 100 (L) (Four numbers of cavern built) | Hornfels | First built in 1978, the archive facility consisted of a four-storey shock-proof building erected inside rock cavern. Drip screen is erected in the roof for water-proofing.<br><br>Four-stage expansion has been planned up to year 2025, with the first and second stages of extension completed in 1998 and 2009 respectively. Two more cavern halls are planned in the next two stages of expansion. |
| Dangerous goods | Ammunition storage | Underground Ammunition Storage Facility, Singapore | - | Granite | Resulted in a 90% reduction in the land area that would have been sterilised by safety buffers for a conventional above ground facility. |
| | Industrial by-product storage | Norway | - | - | Two caverns currently operational with two additional caverns commissioned. |

**Table A2 Examples of Overseas Cavern Usage (Sheet 2 of 6)**

| Category | Uses | Location | Dimensions (in m) (W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Dangerous goods (cont'd) | Nuclear waste storage | Forsmark, Sweden | - | Granite | One facility at a depth of 50 m and with a capacity of 63,000 $m^3$ for the storage of low and intermediate level waste, in operation since 1988. An application was submitted in 2011 for extension of the facility at a depth of 120 m with an estimated capacity of 110,000 $m^3$ for the storage of spent fuel. |
| Data centre | Data centre | Pionen Data Centre, Sweden | 1.1 ha (GFA) | - | The facility also acts as a co-location hosting centre (i.e. equipment space and bandwidth are available for rental). |
| Food and drink storage | Alcohol storage | Aristodal, Sweden | 20 (W) | - | Two caverns for wine and liquor storage, completed in 1957. |
| | Cold storage plant | Bergen, Norway | 20 (W) × 51 (L) × 10.8 (H) | Granitic gneiss | Other than a concrete floor, the rock cavern is completely unlined, with 305 rock bolts providing support in the roof (1 bolt per 5 $m^2$), and a further 40 in the walls. |
| | Food processing | Willow Brook Foods Precision Slicing Facility, USA | 2.5 ha (GFA) | - | Willow Brooks Foods lease space within an existing underground facility. |
| | Food storage | Gonjiam Underground Food Storage Facility, Korea | 12 (W) × 8-10 (H) × 50-60 (L) | - | The facility consisted of four cold caverns, two refrigerating rooms and two chilling rooms with a total capacity of around 3,420 $m^3$. |

**Table A2 Examples of Overseas Cavern Usage (Sheet 3 of 6)**

| Category | Uses | Location | Dimensions (in m) (W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Fuel storage | Ammonia | Glomfjord, Norway | 16 (W) × 20 (H) | Granitic gneiss | Completed in 1986, 60,000 $m^3$, no leakage. |
| | Crude oil | Mongstad, Norway | 18 (W) × 33 (H)<br>50 m wide pillar | Gneiss | Completed in 1987, no problems reported, 1,800,000 $m^3$. |
| | Crude oil, condensate, naphtha and gas oil | Jurong, Singapore | 20 (W) × 27 (H) × 270-380 (L) | Sandstone, siltstone | Comprises five rock caverns connected by 9 km of tunnels and pipelines.<br>Ultimately to be nine caverns, storage also includes condensate, naphtha and gas oil. |
| | Natural gas | Ukraine | Various | - | Comprises 13 underground storage facilities to store 19 billion cubic metres of natural gas underground. |
| | Petrol & naphtha | Herøya, Norway | 10 (W) × 15 (H) | - | - |
| | Propane | Aukra, Norway | 21 (W) × 33 (H) × 95 (L); &<br>21 (W) × 33 (H) × 270 (L) | Gneiss | Completed in 2007, 63,000 $m^3$ and 180,000 $m^3$. |
| | Propane & butane | Yeosu, S. Korea | 18 m (W) × 22 m (H) | - | 170,000 $m^3$ propane;<br>120,000 $m^3$ butane. |
| Industry | Chipboard factory | Waferfab Chip Factory, Switzerland | - | - | The final decision to construct the facility underground was based on the economic advantage of decreased installation costs associated with the vibration-sensitive equipment required. |

## Table A2 Examples of Overseas Cavern Usage (Sheet 4 of 6)

| Category | Uses | Location | Dimensions (in m) (W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Museums | Art Museum | Takyama Festival Float Art Museum, Japan | 40.5 (Dia) × 20 (H) | - | Dome shaped cavern. |
| | Monument | The Chillida Cavern, Fuerteventura<br>(proposed) | 49 (W) × 45 (H) × 65 (L) | - | To include two rectangular light shafts (20 × 20 m and 21 × 30 m). |
| | Museum | Miho Museum, Japan | 13.1 Ha (GFA) | - | Designed to house private family collection. |
| Parking | Car park and civil defence shelter | Finlandia Hall, Finland | 150 ha | - | - |
| Power Stations | Nuclear power plant | Underground Nuclear Power Plant, France | 21 (W) × 44.5 (H) × 42 (L) | - | Only the nuclear parts are placed into two caverns at 200 m depth whist the steam turbines and electric generators are above ground. |
| | Pumped storage hydroelectric power station | Dinorwig Power Station, UK | 8.1-23.5 (W) × 17-51.3 (H) × 5.19-179.3 (L) | - | Europe's largest pumped storage hydroelectric power station, as well as containing Europe's largest cavern. |
| | Stored energy park | Iowa Stored Energy Park, USA | - | - | Planned project. |
| Research Laboratories | Research laboratory | CERN Large Hadron Collider, Switzerland | 23 (W) × 18 (H) × 65 (L); & 35 (W) × 42 (H) × 56 (L) | - | The equipment installed for different experiments are installed in large caverns located around a 27 km tunnel ring; dimensions are computer hall and detector hall respectively. |

## Table A2 Examples of Overseas Cavern Usage (Sheet 5 of 6)

| Category | Uses | Location | Dimensions (in m) (W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Sports | Indoor shooting centre | Brünig-Indoor Underground Shooting Centre, Switzerland | Multiple caverns of various sizes, including shooting cavern galleries of 100, 150 and 300 m. | Limestone | Although no dimensions are specified, the facility includes 100-metre, 150-metre and 300-metre shooting ranges. |
| | Sports centre, community centre and civil defence shelter | Gjøvik Olympic Mountain Hall, Norway | 61 (W) × 25 (H) × 91 (L) | Gneiss | The largest span cavern for civilian use. Lack of external noise makes it a good concert hall venue. |
| | Swimming complex and civil defence shelter | Itäkeskus Swimming Complex, Finland | - | - | Has a capacity of 3,800 when in its civil defence role. Although no dimensions are specified, the facility includes a 50 m swimming pool and various other facilities. |
| Storage and warehousing | Storage | Ekeberg Warehouse, Norway | 15 (W) × 150 (L)<br>13.5 ha (GFA) | - | Comprises six rock caverns and two access tunnels. |
| | Storage & Commercial | Springfield Underground, Missouri, USA | 240,000 $m^2$ leasable floor area | Limestone | Operation began in the 1960s when it first rented its underground space created from limestone mining for cold storage. The stable temperature attracted various tenants including cold storage, data centre, document storage and food storage, while mining continued. Mining operation ceased in July 2015. |
| Tourism | Tourism | North Cape, Norway | - | - | Theatre, restaurants, viewing galleries. |

## Table A2 Examples of Overseas Cavern Usage (Sheet 6 of 6)

| Category | Uses | Location | Dimensions (in m) (W × H × L) | Rock Type | Other Details and References |
|---|---|---|---|---|---|
| Underground Quarrying | Underground quarrying | Ytre Arna, Bergen, Norway | 25 (W) × 50 (H) × 250 (L) voids | Gabbro/meta-gabro | Development of underground quarry began in 1999. Produces 350,000 to 400,000 tons of good quality aggregate a year. Caverns to be backfilled with lightly contaminated material. |
| | Underground quarrying | Schollberg Underground Quarry, Trübbach, Switzerland | Pillar-and-stall cavern, 12 (W) × 18 (H) | Limestone | Underground quarrying commenced in 1985 in response to a change in cantonal and federal land-use planning policy, and environmental and technical permission issues. The visual impact of the quarry site as compared to a surface one was significantly reduced in the very scenic area by going underground. Over 1 million cubic meters of solid rock has been extracted. |
| Waste | Industrial waste | Odda, Norway | - | Gneiss | Aluminium smelter waste. |
| | Nuclear waste | Forsmark, Stripa, Sweden | 69 (W) × 30 (W) | Gneiss | Cylindrical. |
| | Refuse transfer | Stockholm, Sweden | - | - | - |
| Water | Water storage | Lyckebo, Sweden | 18 (W) × 30 (H) 35 m wide pillar | - | Annular cavern for storage of solar heated water. |
| | Water treatment | Oslo, Norway | 13 (W) × 16 (H) | Syenite | - |
| | Sewage treatment | Oslo, Norway | 16 (W) × 10 (H) 12 m thick pillar | Shale and limestone | - |

# Appendix B

## Features and Potential Issues of Potential Cavern Uses in Hong Kong

**Table B1 Features and Potential Issues of Potential Cavern Uses in Hong Kong (Sheet 1 of 5)**

| Land Use Type | Features and Potential Issues |
|---|---|
| Archives | (a) Can be combined with a data centre.<br>(b) Key issues:<br>• Ability to provide timely retrieval of documents.<br>• Environment appropriate for document preservation (e.g. free of damp, mold and pests).<br>• Fire engineering.<br>• Flood protection.<br>• Security.<br>• Temperature and humidity controls. |
| Columbarium/Mausoleum/ Mortuary | (a) Unlikely to be appropriate in the middle of urban areas.<br>(b) Key issues:<br>• Public acceptance.<br>• Architectural design to provide pleasant comfort for visitors.<br>• Fung shui issues - particularly with respect to the location and architecture of the entrance.<br>• Fire engineering.<br>• Traffic impact, especially crowd and traffic control during festive days.<br>• Ventilation, including burning of offerings in designated area of facility. |
| Civic centre<br>Cultural/Performance venue<br>Food and beverage<br>Recreational complex<br>Retail | (a) Usually located in or near urban areas with easy access by public transport.<br>(b) Key issues:<br>• Architectural design to enhance navigation and comfort.<br>• Provision of barrier free access.<br>• Fire engineering. |

**Table B1 Features and Potential Issues of Potential Cavern Uses in Hong Kong (Sheet 2 of 5)**

| Land Use Type | Features and Potential Issues |
|---|---|
| Data centre<br><br>Research/Testing laboratory | (a) Can be combined with archive facilities.<br><br>(b) Key issues:<br>• Excellent connections to the internet.<br>• Uninterruptable power supply.<br>• Controlled environment (temperature, humidity, dust, noise, vibration).<br>• Flood protection.<br>• Security.<br>• Temperature and humidity controls. |
| Explosives depot/magazine | (a) Key issues:<br>• Stringent procedures management and storage of explosives.<br>• Detailed consideration of effects and resistance to accidental detonation of explosives stored in the magazine.<br>• Unlikely to be accepted in the middle of urban areas.<br>• Fire engineering.<br>• Security. |
| Food/Wine storage | (a) Appropriate storage environment (e.g. room temperature for grains and cold/refrigerated conditions for meats, fruits, vegetables). |
| Incinerator | (a) Significant technical challenges for safety and fire engineering (e.g. ventilation, smoke extraction, means of escape, fire suppression and fighting). |
| Indoor swimming pool/ complex<br><br>Leisure/Sports centre | (a) Can be combined with civil defence shelters/emergency shelters.<br><br>(b) Similar to civic centre/cultural/performance venue/ recreational complex |

**Table B1 Features and Potential Issues of Potential Cavern Uses in Hong Kong (Sheet 3 of 5)**

| Land Use Type | Features and Potential Issues |
|---|---|
| Industry<br>Storage/Warehousing<br>Container storage | (a) Key issues:<br>• Controlled environment (temperature, humidity, dust, noise, vibration).<br>• Fire/explosion engineering for some applications.<br>• Flood protection.<br>• Traffic impacts. |
| LPG bulk storage<br>Oil bulk storage | (a) Key issues:<br>• Not acceptable in a residential or commercial area.<br>• Fire/explosion engineering.<br>• Potential contamination of ground and groundwater (not an issue for LPG). |
| Maintenance depot<br>Vehicle (including bus) depot | (a) Key issue:<br>• Traffic impacts. |
| Power station | (a) Key issues:<br>• Fire/explosion engineering.<br>• Flood protection.<br>• Security.<br>• Potential contamination of ground and groundwater (depending on fuel type).<br>• Connection to electricity network. |

**Table B1 Features and Potential Issues of Potential Cavern Uses in Hong Kong (Sheet 4 of 5)**

| Land Use Type | Features and Potential Issues |
|---|---|
| Refuse transfer facility | (a) Key issues:<br>• May not be appropriate in a residential or commercial area.<br>• Could be integrated with waste recycling facilities.<br>• Relocation of a "bad neighbour use", but consideration required for access and environmental issues.<br>• Needs to be as close to their source catchment area as practicable.<br>• Smaller facilities may have less potential to rehouse underground.<br>• Traffic impacts. |
| Service reservoir | (a) Accommodating service reservoir in cavern reduces the visual impact and releases surface land for higher value uses.<br>(b) Connection to water supply network. |
| Sewage/Water treatment plant | (a) Key issues:<br>• Requires adequate ventilation and careful siting of vents.<br>• Connection to sewage network. |
| Slaughterhouse | (a) Key issues:<br>• Hygiene issues.<br>• Traffic impacts. |
| Public utility installation | (a) Key issues:<br>• Fire/explosion protection.<br>• Flood protection.<br>• Security.<br>• Connection to utility network. |

**Table B1 Features and Potential Issues of Potential Cavern Uses in Hong Kong (Sheet 5 of 5)**

| Land Use Type | Features and Potential Issues |
|---|---|
| Transport connections and networks | (a) Key issues:<br>• Flood protection.<br>• Complicated to design and implement.<br>• Traffic impacts. |
| Underground quarrying | (a) Key issues:<br>• Access for heavy plant.<br>• Economic viability.<br>• Spatial requirements.<br>• Traffic impacts.<br>• May not be appropriate for a residential or commercial area. |
| Vehicle parking | (a) Underground parking reduces the visual impact and releases surface land for higher value uses.<br>(b) Key issues:<br>• Traffic impacts.<br>• Ventilation.<br>• Likely to be combined with other land uses. |

# Appendix C

# Planning and Technical Considerations for Cavern Development in Hong Kong

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 1 of 10)**

| Considerations | Comments |
|---|---|
| Economic | (a) An economic assessment should be carried out to ensure there is a sound case for cavern development. This should include:<br>• A comparative cost/benefit analysis of the cavern and non-cavern development options.<br>• Whole-life costs (e.g. capital costs, land rental, operating costs, maintenance costs, decommissioning costs, transportation).<br>• Opportunities to offset costs (e.g. sale of land released at the surface, sale of excavated material, sale of geothermal energy, reduced maintenance costs for some facilities, reduced security costs for some facilities).<br>• Considerations of other intangible benefits such as the provision of parks and recreational areas and low cost housing as these can outweigh quantified monetary benefits.<br>• Changes in land value.<br>• Possibility of future expansion of the development.<br>• Economic uncertainties and risks (e.g. construction time and cost, unforeseen and unforeseeable ground conditions).<br>• Contingency funds related to uncertainties and risks. Goel et al (2012) suggest that contingency funds of about 30% of the estimated construction cost need to be arranged for tackling unforeseen geological risks.<br>• Developer compensation.<br>• Length and terms of lease.<br>• Forms of contract.<br>• Sources of funding.<br>• Opportunities should be considered that would allow several users to share the costs of constructing, maintaining and operating the cavern. Such opportunities include joint ventures, multi-use facilities, connectivity between the cavern development with facilities at the surface and other underground space (e.g. MTR, shopping malls, etc.).<br>• Underground quarrying as part of the cavern development should be considered as win-win solution to generate profit from excavated materials, as well as create space for future use. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 2 of 10)**

| Considerations | Comments |
|---|---|
| Land ownership (including impacts on adjacent sites) | (a) The effects of the development on surrounding land uses (e.g. buildings, road, rail, tunnels, caverns, utilities, slopes), including physical effects (e.g. settlement) and property values should be considered.<br>(b) Enough land should be made available for the development to ensure sufficient space for construction (including cuttings at portals), operation, maintenance, upgrading and a protection zone. This is particularly the case around portals and shafts.<br>(c) Consideration should be given to acquiring (or reserving through the planning system) sufficient land for future expansion.<br>(d) If possible the legal and planning systems should allow for separate ownership of the surface and subsurface and the development of underground space beneath private land.<br>(e) The development content and relevant control on the cubic content of the development should ideally be specified in the form of a three-dimensional development envelope on the lease/engineering conditions. |
| Engineering | (a) The broad engineering feasibility of the proposed development must be secured during the planning phase, with reference to previous cavern developments in similar conditions, the ground conditions and the required span.<br>(b) It is necessary for sufficient suitably qualified planning, design, construction and other specialists (e.g. architecture, blasting, fire engineering, environmental, health and safety, quality control, etc.) to be available.<br>(c) Detailed information is required, including the neighbouring existing buildings, foundations, slopes, underground space and utilities, etc. Ideally this information will be incorporated into a 4D (three-dimensions and time) model and database for the project, which will also include ground conditions.<br>(d) Significant time and cost is involved in preparing, revising and gaining approval for the required design and construction submissions.<br>(e) The risk management system for the development should include technical considerations related to design and construction. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 3 of 10)**

| Considerations | Comments |
|---|---|
| Engineering (cont'd) | (f) Temporary storage of explosives on a site is not permitted other than in a surface or underground magazine (a Mode A Store) licensed by the regulatory authority. Without a Mode A Store, explosives must be used immediately after they are delivered to a site. Explosives delivery from the Government depot is limited to once a day (Monday to Saturday) at a time dictated by the delivery operation. This may constrain severely the production rate for excavation and result in time and cost implications for the project. There will also be cost and programme implications with respect to public consultations, design, construction and satisfying the stringent safety and security requirements for a site explosives magazine. Therefore, a detailed assessment should be made at the feasibility stage of a project on whether a site magazine will be required, and the cost and programme implications of proceeding with and without a site magazine should be fully documented.<br>(g) The requirement for monitoring before, during and following construction should be considered, including cost and time.<br>(h) The effects of construction on man-made slopes will need to be considered.<br>(i) Consideration should be given to the possibility of future expansion of the development, including the effects of excavating a new cavern (e.g. blasting, vibration, redistribution of stresses, induced movements, hydrogeological impacts, construction traffic) on the stability of the existing development and the operations of the existing facilities within.<br>(j) It is recommended that a 4D model (three-dimensions and time) is prepared. Features of this model may include a database (records of planning, design, construction, operation, maintenance, monitoring etc.) for the cavern and its associated infrastructure (portals, shafts, tunnels, adits etc.), its geology and the other features (nearby caverns, foundations tunnels, utilities). Such a model greatly facilitates planning, design, construction and subsequent operation, maintenance, upgrading and expansion and the progression from one stage of the project to the next. It is also invaluable for visualisation and public consultation. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 4 of 10)**

| Considerations | Comments |
|---|---|
| Ground conditions | (a) An understanding of the geological, hydrogeological and hydrological setting must be developed – this should be developed into geological, ground and design models which will ideally be 4D (three-dimensions and time) and integrated with the development layout and database.<br>(b) Good use should be made of existing site investigation information.<br>(c) It is likely that several phases of additional site investigation will be required and the cost and time required for this should be allowed for.<br>(d) Generally good quality rock is required, preferably close to the surface.<br>(e) The design and construction should accommodate the local geology – for instance the cavern should be situated in the best quality rock if possible.<br>(f) Particular attention should be paid to identifying areas with an absence of significant faults, weakness zones and weathering especially for portals, shafts and caverns.<br>(g) Consideration should be given to orientating the excavations, particularly the cavern, as favourably as possible with respect to the geological conditions and in particular the in-situ stress and structural trends of the discontinuities (e.g. faults and joints), if possible and providing structural geological data is available. This opportunity is unlikely to be realised later in the development if it is not taken at this point.<br>(h) The hazard, if any, posed by landslides, slopes failures and boulder/rock fall should be carefully considered for portals, shafts and access roads. In Hong Kong it may be necessary to carry out a Natural Terrain Hazard Assessment depending on the setting of the access points.<br>(i) If reusing excavated material and groundwater is a possibility then the investigation of this must be integrated throughout the entire project.<br>(j) Caverns, shafts and portals should be located to avoid areas of excessive groundwater inflow if possible.<br>(k) The possible effects of the development of hydrogeology and hydrology should be carefully considered and vice versa.<br>(l) The potential of contaminated ground and groundwater affecting the development should be considered, along with the potential for the development to cause contamination. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 5 of 10)**

| Considerations | Comments |
|---|---|
| Access and connectivity | (a) Sufficient access is required during construction to allow safe and efficient construction.<br>(b) Safe access and emergency vehicle access (EVA) are required during operation.<br>(c) Vehicle access for operation, maintenance and upgrading may be required.<br>(d) Users should be able to intuitively find their way into, out of and around the cavern development, particularly for public uses in which cases access for the physically disabled will also be required.<br>(e) It may be desirable for certain access points to be difficult to get to (e.g. secure facilities and ventilation shafts).<br>(f) Connection to nearby facilities or transport nodes may be desirable.<br>(g) Potential access points for Strategic Cavern Areas and for future expansion of other cavern developments should be protected through planning and land ownership where possible. |
| Public engagement | (a) Comprehensive and ongoing public engagement and consultations should be commenced from the outset of any cavern project, including public consultations, addressing all concerns of various stakeholders, depending on the location, use and ownership of the cavern development.<br>(b) It is important that the consultations address the psychological, environmental and safety concerns the community may have with respect to the development (e.g. change of landscape, construction disruption, negative perceptions related to ideas of underground space as dark, cold, damp, humid, stale, confined, claustrophobic, disorientating places, fears of entrapment or collapse, lack of natural environment).<br>(c) Public acceptance is more likely if the development can be shown to improve the environment (e.g. remove a bad-neighbour facility), provides other benefits to the community such as low cost housing, green spaces, parks or recreational areas and/or provides a useful service to the community (e.g. MTR station).<br>(d) If future expansion of the development is a possibility it would be beneficial to inform the public of this during the initial consultations. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 6 of 10)**

| Considerations | Comments |
|---|---|
| Statutory and regulatory requirements | (a) The following government departments shall be consulted at various stage of the cavern development. Some of these require statutory approval:<br>• Buildings Department – Existing Buildings and Structures Impact Assessment. All building works in caverns associated with private development, including fire safety and fire engineering design, are subject to the control of the Buildings Ordinance.<br>• Civil Engineering and Development Department – Geotechnical Assessments, Natural Terrain Hazard Assessments, Construction Material Management, etc.<br>• Mines Division, Civil Engineering and Development Department – Blasting Assessment, blasting licenses and permits, location of site storage facilities (Mode A Stores) of explosives, transportation of explosives.<br>• Environmental Protection Department – Underground rock caverns are subject to the control of the Environmental Impact Assessment (EIA).<br>• Environment, Transport and Work Bureau – Consultation may be required for high-level issues. Note that cavern developments will need to comply with the requirements of Environment, Transport & Works Bureau (ETWB) TC(W) No. 15/2005.<br>• Fire Services Department – Acceptance of fire engineering design.<br>• Lands Department – Land valuation, disposal and leases.<br>• Planning Department – Planning Assessment and land re-zoning.<br>• Transport Department – Traffic Impact Assessment.<br>• Other government departments that may require consultation, depending on the nature of the development, include: Agriculture Fisheries and Conservation Department, Architectural Services Department, Drainage Services Department, Electrical and Mechanical Services Department, Highways Department, Housing Department (including Housing Authority), Hong Kong Police Force, Leisure and Cultural Services Department (including Antiquities and Monuments Office) and Water Supplies Department.<br>• Consultation will also be required with all government departments, lease holders and licensees who own, manage or maintain facilities, land or registered slopes that may be affected by the cavern development (e.g. Housing Authority, MTRC, Drainage Services Department, Water Supplies Department, private utility companies, etc.) |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 7 of 10)**

| Considerations | Comments |
|---|---|
| Statutory and regulatory requirements (cont'd) | • MTR have railway protection boundary for underground works in Hong Kong. If any construction is planned within the protection area, the owner of the underground structure will be informed of the new planned development and will be given the opportunity to comment on the proposed scheme.<br>• Consultations with the relevant government departments and private utility companies to arrange integration of the cavern development into the electricity, water, sewage, telecommunication networks where necessary. |
| Safety | (a) Sufficient emergency exits must be provided.<br>(b) Access for emergency services including sufficient road access for emergency vehicles.<br>(c) Users should be able to intuitively find their way out during emergencies. Adequate means of escape and access for firefighting and rescue should be provided.<br>(d) Careful consideration should be given to fire safety (compartmentalisation, minimising combustible materials, fire prevention, fire resisting construction and materials, fire suppression, smoke control, ventilation) and design to minimise fire risk.<br>(e) Refuge areas should be considered for certain developments.<br>(f) The fire safety requirements of Buildings Ordinance and related regulations should be complied with.<br>(g) Safety management systems could include 24-hour management, emergency lighting, public address systems and surveillance.<br>(h) There should be sufficient redundancy in the various safety measures (e.g. backup generators, multiple exits).<br>(i) Avoidance of existing utilities (e.g. high-voltage power cables, high-pressure water mains).<br>(j) The delivery and storage of explosives during construction will require careful consideration.<br>(k) Health and safety risk registers should be prepared and maintained throughout the project.<br>(l) Consideration should be given to the location of emergency exits, emergency services access points and safety systems with respect to the possibility of future expansion of the development. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 8 of 10)**

| Considerations | Comments |
|---|---|
| Resilience | (a) Caverns are naturally earthquake resistant.<br>(b) Particular areas of concern for underground space include flooding, internal fire, explosion, chemical releases and terrorist attack.<br>(c) Cavern developments can be resistant to flooding providing adequate waterproofing and entrance access protection measures are put in place. The main vulnerable points are entrances, portals and shafts, which should be located above assessed flood levels where possible. Alternatively methods of sealing the access points should be installed and emergency shut up needs to be timely – once underground space has been flooded it can be difficult, costly and time consuming to put the facilities back into operation.<br>(d) Compartmentalisation of large caverns and networks can improve resilience of the cavern development.<br>(e) Backup generators should be available (e.g. for fire protection systems and flood sealing mechanisms). |
| Environmental | (a) Cavern development requires Environmental Impact Assessment (EIA).<br>(b) Consideration will need to be given to the positive and negative environmental effects of the development. This will include consideration of issues including aesthetics (e.g. utilising geology as part of the structure), air quality (including ventilation), carbon footprint, contaminated land, country parks, drainage, dust, ecology, embodied energy, ground gas (e.g. radon), groundwater, hazard to life, landscape, light, noise, odour, protected trees, re-use of material, scheduled sites, sustainability, user experience, visual impact (e.g. shaft and portal) and vibration.<br>(c) In general it will be necessary to minimise the environmental degradation caused by the construction and operation of the cavern and associated infrastructure and maximise the potential benefits (e.g. provision of green spaces / parks / recreational areas at the surface, geothermal linings, locating bad-neighbour facilities underground).<br>(d) The effects of traffic during construction and operation should be considered.<br>(e) The possible environmental effects due to future expansion of the development should be considered.<br>(f) Environmental risk registers should be prepared and maintained throughout the project. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 9 of 10)**

| Considerations | Comments |
|---|---|
| Sustainability | (a) It is necessary to take a long-term view for caverns as, unlike buildings, they cannot be easily removed or "taken down".<br>(b) The potential of preparing for and safeguarding future expansion of the facility should be considered (e.g. acquiring or reserving sufficient land for future expansion).<br>(c) Where possible caverns should be planned such that they are multi-use and can be adapted for other uses.<br>(d) Wherever possible caverns should be planned such that they can be connected to form part of coherent and interrelated complexes at the surface and subsurface and conversely do not unnecessarily conflict with existing and future surface and underground uses.<br>(e) Consider combined cavern uses such as cavern + groundwater source + re-use of excavated material + ground-source heat; conversely avoid negative combinations e.g. oil cavern + contamination of groundwater or cavern + dewatering of ground leading to settlement.<br>(f) Potential integration of underground quarrying with development of cavern land bank for future accommodation of suitable facilities could bring about benefits in terms of long-term land supply.<br>(g) Consideration should be given to considering the impacts of the development holistically, including the use of qualitative and quantitative assessments including whole-life costs, embodied energy and carbon footprints. |
| Decommissioning | (a) Decommissioning of cavern developments is a complex subject and a variety of factors must be considered including:<br>• Generally it is preferable that an alternate use for the facility is found as this maintains the value of the underground space, which generally increases with time.<br>• Failing this, the best way of decommissioning a cavern development is to fill the cavern with an inert material such as concrete, grout or expanding foam. Alternatively caverns have been filled with water or brine.<br>• If this is not possible then the entrances will need to be sealed and measures including stability assessment, installation of additional support, monitoring of instrumentation (both pre-existing and installed for post-decommissioning monitoring), surface level monitoring (to detect any settlement) and periodic inspections (to check the condition of the rock mass, the support and drainage) will likely be required. Requirements will be greatest for cavern developments underlying urban areas. However, there are significant health and safety risks associated with inspections of abandoned underground excavations, including rockfalls, inadequate ventilation, flooding and gases; and communities may not accept the possibility of future settlement or failures. |

**Table C1 Planning and Technical Considerations for Cavern Development in Hong Kong (Sheet 10 of 10)**

| Considerations | Comments |
|---|---|
| Decommissioning (cont'd) | • Areas that are rural at the time of decommissioning may become urban in the future. This must be considered with respect to decommissioning requirements.<br>• Other considerations include whether all of the equipment within the underground space will need to be removed prior to decommissioning, purging of liquid and gaseous contents (if applicable and potentially including liquid or gas trapped within the surrounding rock mass) and whether or not cavern developments below the water table should be allowed to flood following decommissioning (including consideration of changes to the groundwater table over time and particularly if the former cavern use was potentially contaminative).<br>• In all cases upon decommissioning detailed decommissioning/ abandonment plans for the development should be prepared and safely stored in a suitable public repository for future reference. |

# Appendix D

# Q-system

# Contents

## D.1 General

This appendix provides supplementary guidance for application of the Q-system to rock cavern developments in Hong Kong, which complements the relevant sections of Chapters 5 to 7. It is primarily based on Barton & Grimstad (2014) and NGI (2015), supplemented by other publications, and takes into account the design practice adopted by practitioners in Hong Kong in recent tunnel/cavern projects. The Q-system, like many other empirical methods, has been continuously updated to incorporate developments in rock support technology and experience in different ground conditions. Readers shall keep abreast of the latest development.

## D.2 Determination of Q-value

As described in Section 6.8.6, the empirical Q-system is based on the Rock Quality Designation, RQD (Deere, 1963) and five additional parameters, which account for the number of discontinuity sets, the discontinuity roughness and alteration (infilling), the amount of water and various adverse features associated with loosening, high stress, squeezing and swelling.

Guidance on determination of the six parameters is given by Barton & Grimstad (2014) and NGI (2015). The use of the Q-system requires detailed engineering geological mapping and an adequate account to be taken of all the geological features encountered.

The designer should be aware of the following when assessing the *Q* parameters, and when determining whether any factors should be applied for specific situations:

(a) Where Q-values are assessed on the basis of rock cores taken at the project planning stage, it is recommended that the actual cores are logged, rather than using photographs of the cores, whenever possible.

(b) Where the RQD value is not available, it is possible to use the correlation between volumetric joint count ($J_v$) and RQD proposed by Palmström & Stille (2010), although the correlation only provides an approximate average value. The equation is:

$$\mathrm{RQD} = 110 - 2.5J_v \quad \text{(D.1)}$$

where RQD = 0 for $J_v > 44$, RQD = 100 for $J_v < 4$, and $J_v$ is the number of joints per $m^3$.

(c) Fractures caused by blasting should not be included in the assessment of RQD as the blast damaged zone is unlikely to extend more than 2 m beyond the perimeter of the excavation and the natural discontinuities will likely control overall stability. However, these artificial fractures may need to be considered when assessing the stability of individual blocks.

(d) The value of the parameter $J_n$ is not the same as the actual number of discontinuity sets. The parameter, $J_n$, representing the number of joint sets will often be affected by foliation, schistosity, slatey cleavage or bedding, etc. If such parallel discontinuities are strongly developed, they should be counted as one complete joint set.

(e) The parameters, $J_r$ and $J_a$ (representing inter-block shear strength), should be based on the weakest significant joint set or clay-filled discontinuity in a given zone. However, if the joint set or discontinuity with the minimum value of ($J_r/J_a$) is favourably orientated for stability; then a second, less favourably orientated joint set or discontinuity may sometimes be of more significance for design, and its higher value of ($J_r/J_a$) should be used when evaluating $Q$.

(f) When a rock mass contains clay infill, the Stress Reduction Factor (SRF) pertaining to load release should be evaluated. In such cases, the strength of intact rock is of little interest. However, where jointing is minimal and clay is completely absent, the strength of the intact rock may become the controlling factor and stability will then depend on the rock stress / rock strength ratio. A strongly anisotropic stress field is unfavourable to stability and should be duly considered.

(g) In general, the compressive and tensile strengths ($\sigma_c$ and $\sigma_t$) of the intact rock should be evaluated with consideration given to the direction that is unfavourable for stability. This is especially important in the case of strongly anisotropic rocks. In addition, test samples should be saturated if this is appropriate to present or future in-situ ground conditions. A conservative estimate for strength should be made for those rocks that can deteriorate when exposed to moist or saturated ground conditions.

(h) Where the rock mass quality varies markedly from place to place, mapping to delineate different zones should be carried out. In general, the Q-value should be evaluated separately in two adjacent zones if it is considered that a change in support is likely to be needed. (A four-fold increase or reduction in $Q$, caused by a change in joint frequency, roughness or degree of alteration, etc., will normally qualify for a change in support requirements).

(i) For the determination of the mean values of $Q$ for narrow (i.e. 0.5 to 3 m wide) weakness zones and the surrounding rock mass, the empirical equation D.2 (Løset, 1997) can be used. Further guidance on the use of the equation is given in NGI (2015). It should be noted that the actual supported areas should include at least an additional metre on either side of the weakness zone.

$$\log Q_{\mathrm{m}} = \frac{b \log_{10} Q_{\mathrm{zone}} + \log_{10} Q_{\mathrm{sr}}}{b+1} \quad \text{................} \quad \text{(D.2)}$$

where $Q_{\mathrm{m}}$ = mean Q-value for weakness zone / surrounding rock mass

$Q_{\mathrm{zone}}$ = Q-value for the weakness zone

$Q_{\mathrm{sr}}$ = Q-value for the surrounding rock mass

$b$ = width of the weakness zone measured along the length of the excavation (in metre).

## D.3 Using the Q-system to Evaluate the Support Requirements for Cavern Development in Hong Kong

The Q-system incorporates a rock mass quality and rock support chart from which the required rock support measures based on the Q-value can be derived.

The Excavation Support Ratio (ESR) represents the safety requirement for the use of the underground space and includes considerations for both temporary and permanent openings.

The following points highlight items of particular relevance to the applicability and use of the Q-system for cavern developments in Hong Kong.

(a) The igneous rocks in Hong Kong are generally suitable for the use of the Q-system. The system may also be applicable to the sedimentary and metamorphic rocks subject to an assessment on a case-by-case basis.

(b) Igneous rock around the excavation should be fresh to moderately decomposed (i.e. Grades I to III) and of sufficient strength for the opening to be stable under the in-situ stress without adverse deformation. The Q-system is not considered applicable where the rock mass is highly or more decomposed (with the exception of discrete weakness zones).

(c) The minimum rock cover should be half of the greater of the span or the height of the excavation above the crown of the opening if the Q-system is to be used without verification by additional numerical analysis (see also Section 6.4.2).

(d) The following guidance refers to numerical analysis requirements for temporary support design, considering the design dimension $D$ (rock adit / tunnel / cavern span or height, or rock shaft diameter / largest dimension):

(i) for $D$ < 16 m, the design may be carried out without additional numerical analysis provided that there is no suspected/confirmed adverse geological conditions in the ground which could lead to an inadequacy of design by the Q-system;

(ii) for $D$ = 16-25 m, additional numerical analysis is optional but may need to be carried out depending on the complexity and range of the rock mass conditions expected to be encountered; and

(iii) for $D$ > 25 m, the design should be based on / verified by numerical analysis.

Regardless of $D$, numerical analysis should be carried out where there are complex geometries, sensitive structures and/or close interaction of various openings to ensure that the interaction of the ground and the nearby openings and/or sensitive structures are analysed and assessed for their stability, operability and safety.

(e) For portals/intersections, the Q-values should be adjusted accordingly ($J_n$ × 2 for portals; $J_n$ × 3 for tunnel intersections) and the specific temporary support should cover at least one adit/tunnel/shaft diameter from the outer edge of the feature (using the diameter of the smallest of the intersection openings). Additional numerical analysis may be required depending on the nature of the geological setting and the geometry of the excavations. The design should carefully examine the adequacy of support at overlapping stressed zones.

(f) The Q-system is not applicable where stability is controlled by isolated blocks formed by particular combinations of discontinuities. The joint geometry should be considered specifically for each case when designing the rock support. NGI (2015) gives some examples of unfavourable joint geometries that require special attention. The design must include provisions to ensure that kinematically permissible rock wedges are identified, assessed and supported individually in both temporary and permanent conditions.

(g) The use of the Q-system alone is insufficient for assessment of pillar support (see also Section 6.8 for guidance on requirements for analysis).

(h) In the SRF table, items for “high stress, very tight structure” to “heavy swelling rock pressure” are unlikely to be applicable to the majority of cavern developments in Hong Kong.

(i) Based on experience from recent local tunnel projects, for temporary support, an ESR value of 1.6 can generally be adopted for all types of excavations, provided that the support elements are of adequate durability for their required design life. An ESR value of 1 has been used for rock bolt length determination for both the temporary and permanent conditions, unless the designer is satisfied that a different design length is appropriate as supported by sufficient numerical and/or other analytical analysis for the scale of excavation and variety of rock mass conditions anticipated (see also Equation 6.12).

(j) Barton & Grimstad (1994) has suggested a number of factors which can be applied to the basic Q-value to relax the temporary support requirements. In practice, it is recommended that the applicability of these factors should be carefully considered on a case-by-case basis.

## D.4 References

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# Glossary of Terms

## Glossary of Terms

**Discontinuities.** See *faults, shear fractures* and *joints*.

**Embodied energy.** The sum of the energy requirements associated, directly or indirectly, with the delivery of a good or service.

**Fault.** A discontinuity surface across which there has been a shear displacement.

**Gravitational stress.** The stress state due to the weight of the super-incumbent rock mass. A synonym is *overburden stress*.

**Hard rock.** Generally rock of decomposition grade I to II (i.e. fresh and slightly decomposed) and with strength of strong or above in accordance with Geoguide 3. However, definition may include areas of rock of decomposition grade III and moderately strong in appropriate circumstances.

**Joint.** A fracture formed in tension in which any displacement is too small to be visible to the unaided eye.

**Maximum Instantaneous Charge (MIC).** The maximum charge, in kilograms, initiated at any instant in time. The "8 millisecond rule" is typically applied for non-electric detonators in Hong Kong. This means that charges detonated less than 8 ms apart are considered to initiate simultaneously; thus, the quantity of explosives from all such charges is counted as part of the MIC. For electronic detonators, a shorter time frame may be applicable, depending on the accuracy of the detonators being used.

**Natural stress.** The stress state that exists in a rock prior to any artificial disturbance. The stress state is the result of various events in the geological history of the rock mass. Therefore, the natural stresses present could be the result of the application of many earlier states of stress. Synonyms include *virgin*, *primitive, field* and *active*.

**Look-out angle.** The angle of perimeter holes inclining outward from the excavation direction to allow sufficient space for the drilling of the next round of blast holes.

**Powder factor.** The quantity of explosives in kilograms per cubic metre of rock to be excavated in the blast.

**Rock anchor.** Tendon (usually a high tensile steel bar, strand or wire acting as a tension member) and its associated components that transmit load into rock. In contrast to a rock bolt, it is often pre-stressed to a high tension to exert a high active force on the rock mass.

**Rock bolt.** Both tensioned bolts and fully grouted untensioned bolts in the context of this Geoguide. The latter type of bolt is termed 'dowel' in the Geotechnical Manual for Slopes (GCO, 1984). Pre-tensioned is required for some types of bolt to activate their anchorage and to ensure they are effective. However, the pre-stressing of the rock is essentially incidental.

**Shallow depth.** Cavern developments located at a depth less than approximately 100 to 150 m**.**

**Shear fracture.** A fracture formed by shear at an acute angle to the maximum principal stress. Essentially a type of fault with millimetre-scale displacement that can be difficult to distinguish from a joint.

**Sustainability.** Defined here as development that "*meets the needs of the present without compromising the ability of future generations to meet their own needs*".

**Weakness zone.** Weakness zones are defined as zones that are weaker than the surrounding rock. The width of a weakness zone can range from a few centimetres to several hundred metres. Weakness zones have many origins and can be weak rocks, faults, heavily fissured zones, hydrothermally altered rocks, deeply weathered zones and combinations of these.

# Glossary of Abbreviations

# Glossary of Abbreviations

| | |
|---|---|
| 2D | Two-dimensional |
| 3D | Three-dimensional |
| 4D | Four-dimensional |
| AAV | Aggregate abbrasion value |
| ACV | Aggregate crushing value |
| AIV | Aggregate impact value |
| ANFO | Ammonium nitrate fuel oil |
| API | Aerial photograph interpretation |
| ATV | Acoustic televiewer |
| BIM | Building information modelling |
| BS | British Standard |
| BWI | Bit wear index |
| CAI | Abrasiveness index test |
| CAPRO | Cavern Projects Study |
| CAS | Cavern Area Studies |
| CCTV | Closed-circuit television |
| CEDD | Civil Engineering and Development Department |
| CEL | Civil Engineering Library |
| CIRIA | Construction Industry Research and Information Association |
| CLP | CLP Power Hong Kong Limited (formerly China Light and Power) |
| CMP | Cavern Master Plan |
| CSIRO | Commonwealth Scientific and Industrial Research Organization |
| CSM | Cavern Suitability Map |

| | |
|---|---|
| dBL | Decibel linear |
| DCD | Directional core drilling |
| DGIU | Digital Geotechnical Information Unit |
| DRI | Drilling rate index |
| DSD | Drainage Services Department |
| E&M | Electrical and mechanical |
| EIA | Environmental Impact Assessment |
| EPD | Environmental Protection Department |
| ESR | Excavation support ratio |
| ETWB | Environment, Transport and Works Bureau |
| EVA | Emergency vehicle access |
| FRC | Fibre-reinforced concrete |
| FRS | Fibre-reinforced shotcrete |
| FSD | Fire Services Department |
| GBR | Geotechnical Baseline Report |
| GCO | The Geotechnical Control Office (now the Geotechnical Engineering Office) |
| GEO | The Geotechnical Engineering Office |
| GFA | Gross floor area |
| GIS | Geographical information system |
| GIU | Geotechnical Information Unit |
| GRP | Glass reinforced plastic |
| GSI | Geological Strength Index |
| HDC | Horizontal directional coring |
| HDPE | High-density polyethylene |
| HKGS | Hong Kong Geological Survey |

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|---|---|
| HKPSG | Hong Kong Planning Standards and Guidelines |
| HKSAR | Hong Kong Special Administrative Region |
| ISO | International Organization for Standardization |
| ITA-AITES | International Tunnelling and Underground Space Association |
| JCS | Joint compressive strength |
| JRC | Joint roughness coefficient |
| LAV | Los Angeles value |
| LHD | Load-haul-dump |
| LiDAR | Light detection and ranging |
| LPG | Liquefied petroleum gas |
| M&W | Materials and workmanship |
| MIC | Maximum instantaneous charge |
| mPD | Metres above Principal Datum |
| MTR | Mass Transit Railway |
| MTRC | MTR Corporation Limited (formerly Mass Transit Railway Corporation) |
| NEQ | Net explosive quantity |
| NGI | Norwegian Geotechnical Institute |
| NIMBY | “Not in my back yard” |
| OTV | Optical televiewer |
| PAH | Project Administration Handbook |
| PCCW | Pacific Century CyberWorks |
| PEG | Pre-excavation grouting |
| PEGS | Preliminary Engineering Geological Studies |
| PPV | Peak particle velocity |
| PSV | Polished stone value |

| | |
|---|---|
| PVC | Polyvinyl chloride |
| REV | Representative elemental volume |
| RMi | Rock mass index |
| RMR | Rock mass rating |
| RQD | Rock quality designation |
| RRS | Reinforced ribs of shotcrete |
| RSR | Raisebore stability ratio |
| SPUN | Study of the Potential Use of Underground Space |
| SRF | Stress reduction factor |
| SSE | Site sensitised explosives |
| SSPB | Swedish State Power Board |
| TBM | Tunnel boring machine |
| TC(W) | Technical Circular (Works) |
| TCR | Total core recovery |
| TFV | Ten-percent fines value |
| UCS | Uniaxial compressive strength |
| USACE | United States Army Corps of Engineers |
| USBM | United States Bureau of Mines |
| WHO | World Health Organization |
| WSD | Water Supplies Department |
| XRD | X-ray diffraction |

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