

GUIDE TO CAVERN ENGINEERING

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

GEOGUIDE 4

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**GEOTECHNICAL ENGINEERING OFFICE
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Special Administrative Region**

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FOREWORD

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

The contents of this Geoguide are derived from Scandinavian practice but suitable modifications have been included to make it applicable to conditions in Hong Kong. Whilst the scope of the document is defined as the design and construction of rock caverns, much of the material presented here will be applicable to tunnelling works. As with other Geoguides, this document gives guidance on good engineering practice, and its recommendations are not intended to be mandatory. It is recognised that experienced practitioners, on whose judgement the success of any underground excavation depends, may wish to use alternative methods to those recommended herein.

The Geoguide was drafted by Mr O. J. Berthelsen of Berdal Strømme Consulting Engineers as sub-consultants to Ove Arup and Partners under a consultancy agreement with the Geotechnical Engineering Office. The membership of the Steering Committee and Working Group are given on the opposite page. The consultancy agreement was managed by Mr D. J. Howells and his staff in the Advisory Division of the Office.

To ensure that the Geoguide would be considered a consensus document by the various interested parties in Hong Kong, a draft version was circulated locally and abroad for comment in early 1991 to contractors, consulting engineers, academic institutions and Government departments. Many individuals and organisations made very useful comments, which have been taken into account in finalising the Geoguide, and their contributions are gratefully acknowledged.

Practitioners are encouraged to comment at any time to the Geotechnical Engineering Office on the contents of this Geoguide, so that improvements can be made to future editions.



A. W. Malone
Principal Government Geotechnical Engineer
March 1992

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CONTENTS

	Page No.
TITLE PAGE	1
FOREWORD	3
CONTENTS	5
1. INTRODUCTION	11
1.1 SCOPE AND OBJECTIVE	11
1.2 BACKGROUND	12
1.3 REVIEW OF CAVERN USAGE	12
2. GEOLOGICAL INPUT TO CAVERN ENGINEERING	15
2.1 INTRODUCTION	15
2.2 GEOLOGY OF HONG KONG	15
2.2.1 Solid Geology	15
2.2.2 Structure	16
2.2.3 Weathering	17
2.2.4 Soils	17
2.3 INSITU STRESSES	17
2.4 HYDROGEOLOGY	17
3. SITE INVESTIGATION	19
3.1 INTRODUCTION	19
3.2 OBJECTIVE OF SITE INVESTIGATION	19
3.3 EVALUATION OF EXISTING DATA	20
3.3.1 Objective	20
3.3.2 Sources of Information	20
3.3.3 Aerial Photograph Interpretation (API)	21
3.4 PLANNING OF GROUND INVESTIGATION	22
3.4.1 Approach	22
3.4.2 Geological Models	22
3.4.3 Ground Investigation Methods	23
3.4.4 Information to be Obtained	23

3.5	ENGINEERING GEOLOGICAL FIELD MAPPING	24
3.6	FIELD INVESTIGATIONS	25
3.6.1	Extent of Ground Investigation	25
3.6.2	Geophysical Investigation Methods	25
3.6.3	Drillhole Investigation	28
3.6.4	Hydrogeological Investigation	29
3.6.5	Trial Shafts and Adits	29
3.7	FIELD AND LABORATORY TESTING	30
3.7.1	General	30
3.7.2	Permeability Testing	30
3.7.3	Joint Orientation	30
3.7.4	Borehole Surveying	31
3.7.5	Rock Stress Measurement	31
3.7.6	Mechanical Strength	33
3.7.7	Deformation Parameters	33
3.7.8	Chemical Properties	34
3.7.9	Mineralogy and Swelling Properties	34
3.7.10	Rock Temperature	34
3.7.11	Blasting Trials	34
3.7.12	Tests for Drillability	35
4.	CAVERN DESIGN	37
4.1	DESIGN APPROACH	37
4.1.1	Basis of Cavern Design	37
4.1.2	Design Sequence	37
4.1.3	Rock as a Structural Material	38
4.1.4	Rock Classification Systems for Determining Rock Support Requirements	38
4.2	CAVERN LOCATION AND ORIENTATION	38
4.2.1	Factors in Optimising Cavern Location and Orientation	38
4.2.2	Minimum Rock Cover	39
4.2.3	Weakness Zones	40
4.2.4	Jointing	40
4.2.5	Groundwater	41
4.2.6	Stress Conditions	41
4.3	CAVERN LAYOUT AND SHAPE	42
4.3.1	Introduction	42
4.3.2	Cavern Shape	42
4.3.3	Spacing between Caverns	43
4.4	INFLUENCE OF CONSTRUCTION METHODS AND COSTS ON DESIGN	44
4.5	EVALUATION OF ROCK MASS QUALITY AND ROCK SUPPORT	45
4.5.1	Approach	45
4.5.2	Q-system for Rock Classification and Support Estimation	45

4.5.3	RMR Method of Rock Classification	47
4.5.4	Rock Bolt Lengths and Spacings	48
4.5.5	Design of Shotcrete and Fibrecrete Support	50
4.5.6	Design of Cast Concrete Support	50
4.6	ANALYTICAL AND NUMERICAL METHODS	50
4.6.1	Introduction	50
4.6.2	Stability Analyses	51
4.6.3	Numerical Models	51
5.	CAVERN CONSTRUCTION	55
5.1	INTRODUCTION	55
5.1.1	Purpose and Scope	55
5.1.2	Methods of Excavation	55
5.1.3	Working Cycle	55
5.1.4	Initial and Final Support	56
5.2	PLANNING THE EXCAVATION	56
5.2.1	Top Heading	56
5.2.2	Benching	57
5.2.3	Access Tunnels	58
5.2.4	Ventilation	59
5.3	TOP HEADING EXCAVATION	59
5.3.1	Drilling Blast Holes	59
5.3.2	Explosives and Charging	60
5.3.3	Scaling	60
5.3.4	Mucking-out	60
5.3.5	Rock Support	61
5.4	BENCH EXCAVATION	61
5.4.1	Drilling Blast Holes	61
5.4.2	Control of Over-break	61
5.4.3	Explosives and Charging	61
5.4.4	Scaling	61
5.4.5	Mucking-out	62
5.4.6	Rock Support	62
5.5	SHAFT EXCAVATION	62
5.5.1	Need for Shafts	62
5.5.2	Choice of Method of Shaft Construction	62
5.5.3	Long-hole Drilling	63
5.5.4	Shaft Excavation by Raise Boring	63
5.5.5	Alimak Method	64
5.5.6	Raise Building	64
5.5.7	Pilot Shaft Method	64
5.5.8	Drilling	64
5.6	ROCK SUPPORT METHODS	64
5.6.1	Design Philosophy	64

5.6.2	Short-term and Long-term Safety Requirements	65
5.6.3	Selection of Support Methods	65
5.6.4	Rock Bolts	66
5.6.5	Long Bolts Ahead of the Face: Spiling	70
5.6.6	Shotcrete	70
5.6.7	Rock Anchors	75
5.6.8	Concrete Lining	75
5.7	BLAST VIBRATIONS AND BLAST DESIGN	76
5.7.1	Introduction	76
5.7.2	Blast Vibration Acceptance Criteria	76
5.7.3	Energy Propagation	77
5.7.4	Blast Design	78
5.8	EXPLORATORY DRILLING AND GROUTING	80
5.8.1	Need for Exploratory Drilling and Grouting	80
5.8.2	Drilling Exploratory Holes and Grout Holes	80
5.8.3	Grouts and Grouting	81
5.9	DISPOSAL OF SPOIL	82
5.9.1	Rates of Spoil Production	82
5.9.2	Options for Spoil Disposal	82
5.10	WATER PROOFING AND OTHER MITIGATION MEASURES	82
5.11	CONSTRUCTION TIME	83
5.11.1	Rates of Excavation	83
5.11.2	Rates of Support Installation	83
5.12	CONSTRUCTION RECORDS	84
6.	QUALITY CONTROL, MONITORING AND MAINTENANCE	85
6.1	INTRODUCTION	85
6.2	QUALITY CONTROL	85
6.3	MONITORING	86
6.3.1	Need for Monitoring	86
6.3.2	Exposed Rock Surfaces	86
6.3.3	Shotcrete and Fibrecrete	87
6.3.4	Displacement Measurements	87
6.3.5	Rock Bolts	87
6.3.6	Rock Anchors	88
6.2.7	Groundwater	88
6.2.8	Rock Temperature	88
6.2.9	Stress Measurement	88
6.2.10	Radon	88
6.4	MAINTENANCE	88
7.	REFERENCES	91

TABLES	101
LIST OF TABLES	103
TABLES	105
FIGURES	119
LIST OF FIGURES	121
FIGURES	123

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1. INTRODUCTION

1.1 SCOPE AND OBJECTIVE

The purpose of this Geoguide is to provide designers and regulatory authorities in Hong Kong with a guide to good cavern engineering practice. The Geoguide concentrates on the geotechnical aspects of cavern engineering and aims to provide guidance on how to produce cost-effective and safe integrated designs.

This Geoguide is aimed primarily at engineers engaged in the design and supervision of construction of cavern installations. These engineers are likely to be specialists in rock engineering and cavern design as well as those engaged with the fitting-out of the cavern including civil, mechanical and electrical works. Non-specialist professionals, including planners and developers, will also find this Geoguide of use as it indicates the possibilities and advantages of the use of underground space as well as the limitations.

Caverns and tunnels differ from each other primarily in their physical dimensions and uses to which they are put. Tunnels are used essentially for transport and access, whereas caverns are used for storage or a process, including commercial activities, and possibly habitation. The engineering principles for tunnels and caverns are the same. Some caverns are, however, very large in cross-section compared to most tunnels and special design and construction considerations may apply.

The Geoguide covers Hong Kong ground conditions and their implications for cavern planning and design, site investigation, design concepts and practices, descriptions of geotechnical processes and installations commonly used in cavern construction, methods of excavation, monitoring and maintenance. The chapter on cavern construction gives information relevant to cavern design, but is not an exhaustive description of the construction techniques or construction planning.

A brief review of current cavern usage is included to demonstrate the considerable scope of successful cavern installations. Precedents are particularly important in cavern engineering as most of the body of design rules are empirical. The design methods described in this Geoguide are the ones most commonly used and are generally preferred, but other methods may be considered.

The granites and volcanic rocks of Hong Kong are generally suited to cavern construction. The specific problems that may be related to design of caverns for the other rocks in Hong Kong, the meta-sediments, are not covered by this Geoguide, but the principles of design still apply.

Special design aspects of caverns for particular applications are not covered, for example radio-active waste repositories, fortification, heat flow problems and de-humidification. Reference may be made to studies such as SPUN (Ove Arup and Partners, 1990) and other references for more information.

In using this Geoguide it must be realized that the information contained herein, and the design procedures described, presuppose the engineering judgement of a skilled and

experienced practitioner; the Geoguide can never be a substitute for the experienced cavern designer.

1.2 BACKGROUND

There has been an increasing interest in Hong Kong in placing facilities underground. This interest has arisen because of the considerable planning and cost advantages that can be realized and because of an increased awareness of the cavern technology developed elsewhere for rock conditions similar to those found in Hong Kong.

This interest was the background to the SPUN (A Study of the Potential Use of Underground Space) study carried out for the Geotechnical Control Office (Ove Arup and Partners, 1990). SPUN explored the feasibility of placing various types of facilities in Hong Kong underground and included a report on the cavern engineering for the schemes, which is the immediate predecessor to this Geoguide.

The driving forces which have brought cavern solutions to the fore in Hong Kong are environmental and fiscal. Planning and technical considerations make it necessary to place potentially noxious operations within the urban areas. These can be located in rock caverns without significant effect on the environment. Examples of such operations are refuse transfer stations and sewage treatment works. Placing certain activities underground releases land for other uses and also yields potential planning benefits.

1.3 REVIEW OF CAVERN USAGE

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 man 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 civilizations. Many of these uses were funerary or ceremonial as in dynastic Egypt. Extensive use of underground space can be seen in the Nabatean city of Petra in 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 space, 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 nineteenth century. At that time the cross-sectional area of caverns was rarely more than 30 m² and the excavation depended heavily on manual labour. Today's methods of excavation have made cross-sectional areas in excess of 800 m² technically feasible and economic.

The variety of uses of underground space is now considerable. Some uses are common, with many installations in operation, e.g. oil products stores, and some are specialist or unique, e.g. submarine docks. Many are associated with mining operations and with transport projects, e.g. for railway stations and for storage and various industrial processes. Tables 1 and 2 illustrate the extent and variety of underground schemes currently in operation. Table 3 gives a summary of large span tunnels and caverns constructed in Hong Kong.

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2. GEOLOGICAL INPUT TO CAVERN ENGINEERING

2.1 INTRODUCTION

Knowledge of the geology is the starting point for all geotechnical investigations for cavern schemes. This knowledge is useful for the early assessment of the suitability of a particular cavern site and as a basis for the development of the geological models that are used to plan detailed investigations.

The following aspects of the geology of a site are of fundamental importance to cavern design and construction:

- (a) The *solid geology*, or rock type, gives an indication of the mechanical properties, joint spacing and the pattern of weathering that might be expected. The rock type also influences the cost of drilling and blasting. In the strong, igneous rocks of Hong Kong the discontinuities govern engineering behaviour.
- (b) The *geological structure* gives a guide to the orientation of major discontinuities that might be encountered, such as faults; joint orientations tend to follow regional patterns (GEO, 1986 onwards; Whiteside & Bracegirdle, 1985).
- (c) Typical *weathering* patterns give an indication of the total cover required for a cavern and the works that might be required in establishing portals and access tunnel.
- (d) The *hydrogeology* can be important for some sites and some types of cavern installation and an initial evaluation can be made on the basis of generalized data.

2.2 GEOLOGY OF HONG KONG

2.2.1 Solid Geology

A variety of rock types of igneous, sedimentary and metamorphic origin occur in Hong Kong. Of these, the rocks of igneous origin, principally granite and the various volcanic rock types, have the greatest potential for cavern development. 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. **Figure 1** shows a generalised geological map of Hong Kong indicating the distribution of the principal rock types. 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-eastern New Territories. In north-east Lantau and Tsing Yi, a major swarm of felsparphyric and quartzphyric rhyolite and rhyodacite dykes has intruded both the granitic and volcanic rocks.

Sedimentary rocks outcrop in two main areas in the north-west and north-east New Territories (Figure 1). These have undergone regional metamorphism and display metamorphic effects to varying degrees.

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.

A detailed geological description of the rocks and stratigraphy of Hong Kong is given in a series of six memoirs (Addison, 1986; Langford et al, 1989; Strange & Shaw, 1986; Strange et al, 1990; two in preparation). These are accompanied by fifteen geological maps at a scale of 1:20 000 (GEO, 1986 onwards). Geological maps at a scale of 1:5 000 are available for selected development areas.

The granites and volcanic suite of rocks ranges in age from Middle Jurassic to Early Cretaceous. The granites are generally widely jointed with typical joint spacings between 0.5 m and 2 m. Sheeting joints are often present near the surface.

The granitic rocks are normally composed of feldspars, 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, quartz monzonite and syenite, in the form of sheet-like plutons, outcrop in many areas.

The volcanic suite of rocks has a varied lithology and includes fine ash tuff, coarse ash tuff and trachydacitic and rhyolitic lava flows. Tuff is the most common rock type. The rocks are generally closely jointed with typical joint spacings between 50 mm and 200 mm; however, in the coarse grained varieties, joint spacings up to 3 m may be found.

Basalt 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.

2.2.2 Structure

Many of the rocks mentioned above have been faulted and sheared. The faults follow three dominant regional trends: north-west, north-east and east-north-east, with regional dyke swarms often following the latter trend. Fault zones, which vary in width and which can be many metres wide, can be vertical or inclined and may be weathered to considerable depths. 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 more prominent faults are shown on the 1:20 000 scale geological maps but smaller faults can be commonly identified in the field. Hydrothermal alteration is frequently associated with fault zones and infilling by quartz mineralisation is common.

2.2.3 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 its thickness can be several tens of metres (GCO, 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 also occurs which can be difficult to distinguish from extreme cases of sub-aerial weathering. The weathering of granites in Hong Kong is described in the Hong Kong Geological Survey Memoir No.2 (Strange & Shaw, 1986), which provides an excellent introduction to the formation and nature of these important but complex materials. Recommended standards for describing weathered rocks and weathering profiles are given in Geoguide 3 (GCO, 1988).

2.2.4 Soils

In Hong Kong, materials on natural slopes that may be regarded and treated as soils for engineering purposes, are products of chemical and physical weathering. Soils may thus be insitu weathered rock or transported weathering products that have moved down slope and covered the insitu materials. The thickness of transported deposits rarely exceeds 30 m, but the thickness of the weathered rock mantle can be 60 m and more (GCO, 1984).

In many low-lying areas, recent marine and estuarine deposits are common. These might entail soft ground excavation techniques for access tunnels should caverns be constructed in the rock beneath them.

2.3 INSITU STRESSES

From the limited amount of test data there is no evidence of high tectonic stresses in the rocks of Hong Kong. All indications are that high stresses will not be a problem for cavern construction at modest depths given the high strength of most of the rocks encountered. Insitu stress measurements have shown that the stresses are mostly low with the horizontal stresses higher than the vertical stresses, at least at shallow depth. Near major geological structures, such as faults, the stresses may deviate from the normal pattern.

Low insitu stresses can be important, as increased overbreak and increased water seepages can result.

Details of tests carried out in Hong Kong are given in Section 3.7.5 and the effects of insitu stresses on caverns are discussed in Section 4.2.6.

2.4 HYDROGEOLOGY

The groundwater regime can have an influence on the siting and engineering of a cavern. Some types of cavern installations, such as oil products stores and sewage treatment works, rely on groundwater seepage towards the cavern for containment of liquids and gases.

The groundwater table can also be affected by caverns. The groundwater regime can be complex and may be difficult to elucidate.

The groundwater surface in the hills of Hong Kong is commonly 10 to 30 m below ground level. The depth of rock cover required for most caverns will be greater than this and most of them will therefore be located below the groundwater table.

Groundwater also occurs as ephemeral groundwater perched in the weathered rock or superficial deposits. This groundwater would normally have little consequence for cavern schemes.

The permeability of the rock mass in Hong Kong is generally low (of the order of 10^{-8} m/s), particularly in the volcanic rocks, and is dominated by fissure flow. The low permeability is reflected in the steeply sloping groundwater surfaces that are often observed and in the relatively low seepage rates experienced in tunnel excavations. However, higher permeabilities occur in some zones, and substantial seepages from heavily fissured zones have been observed in tunnel excavations, particularly in the granitic rocks. Heavy water inflows can affect the local stability of underground excavations.

3. SITE INVESTIGATION

3.1 INTRODUCTION

The geology of Hong Kong outlined in Chapter 2 forms the framework for more detailed geotechnical considerations of specific sites. This chapter describes the objectives of site investigation for caverns, its methodology and scope, as well as ground investigation techniques of particular importance for cavern schemes. The methods of ground investigation are not described in detail; Geoguide 2: Guide to Site Investigation (GCO, 1987) should be referred to for details of these methods. The guidance in Geoguide 2 applies in general to cavern schemes. In the interest of continuity, some information contained in Geoguide 2 has been included in this Geoguide.

3.2 OBJECTIVE OF SITE INVESTIGATION

The primary objective of site investigation for a cavern project, in common with those for most other civil works, is to provide:

- (a) data for the assessment of the suitability of a site for cavern construction and the making of choices where more than one site is considered,
- (b) data for the design of the caverns including optimisation of size, shape, orientation and rock support (which includes consideration of construction methods),
- (c) data that allows a realistic assessment of construction time and associated cost and the implications of any geological hazards and
- (d) the basis for assessing the effects on the surroundings as a result of the scheme and engineering data for an environmental impact study in accordance with the requirements of the Environmental Protection Department.

The required quality and quantity of design data increases as the project develops from its inception stage to implementation. The nature and extent of the site investigation must be appropriate to the stage of project development. Improved cost estimates require improved geotechnical data.

The amount of ground investigation that should be done for a particular stage of project development depends on many factors including the availability of existing data, the availability of good rock exposure, the complexity of the geology and the skill and experience of the engineering geologist.

The environmental impact assessment (EIA) for the project requires a systematic approach and should be in accordance with Lands and Works Branch Technical Circular 9/88,

Advice Note 2/90 issued by the Environmental Protection Department and guidelines in Hong Kong Planning Standards and Guidelines Chapter 9.

3.3 EVALUATION OF EXISTING DATA

3.3.1 Objective

The objective of the desk study is to obtain as much available information as possible on the ground conditions and other factors that might influence the siting of a project 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: rock type; structure (including discontinuities); nature and depth of weathering; the presence of landslips; the nature, thickness and extent of natural soils and fill material; and the hydrogeology; all leading to a preliminary assessment of the geological implications for the scheme including development of a tentative geological model to be used for further investigations and a preliminary assessment of engineering properties of the rocks and soils,
- (b) previous and current land uses that might affect the scheme, particularly underground works such as mines and tunnels which may affect implementation,
- (c) identification of land uses, including buildings and their occupants, above or adjacent to the caverns, that might be affected by the works,
- (d) the location of services and utilities which may affect or be affected by the scheme,
- (e) suitable means of providing access to the caverns, including portal and shaft locations, that will satisfy environmental, traffic and engineering considerations,
- (f) environmental conditions that might impose restrictions on scheme implementation and
- (g) lease and engineering conditions imposed by the Hong Kong Government, which include requirements, restrictions and responsibilities in relation to the scheme.

3.3.2 Sources of Information

Geoguide 2 (GCO, 1987) gives comprehensive guidance on desk studies and information to be collected for civil engineering schemes in general, and this applies equally to cavern projects. Geoguide 2 also gives the principal sources of information. For the

underground components of cavern schemes, the access tunnels and the caverns themselves, the following are commonly the most important for the preliminary assessment of a site:

- (a) topographic maps at scales of 1:20 000 down to 1:500,
- (b) geological maps at a scale of 1:20 000 with Memoirs (1:5 000 maps are available for some areas),
- (c) aerial photographs at various scales depending on the scale of the features being studied and
- (d) records of past works including ground investigation data.

The Hong Kong Geological Survey is responsible for maintaining up-to-date maps of the Territory and the Geoscience Database, and giving geological information and consultative services to the public and private sectors. Since its establishment in 1982 as a unit of the Geotechnical Engineering Office (GEO) (then the Geotechnical Control Office), the Hong Kong Geological Survey has published over forty maps and memoirs. Its Geoscience Database includes a bibliography of Hong Kong geology, geological and photographic collections. The Geotechnical Information Unit, located in the Civil Engineering Library, is a data bank for all geotechnical information collected by the GEO and is open to members of the public. The collection includes ground investigation reports for over 10 000 projects with logs for over 200 000 boreholes.

At the time of data collection, a brief site reconnaissance should be made to record major features, whether geological or geographical, and human activity.

3.3.3 Aerial Photograph Interpretation (API)

Aerial photographs, particularly when examined stereoscopically, can often be used to identify and delineate specific ground features such as the distribution of soil types, soil thickness, bedrock types, depth to bedrock, fracture patterns and spacings as well as local relief. API is of particular value in the mapping of 'photolineaments'. This term refers to straight line or gently curving features on aerial photographs that are usually the surface expression of variation in the structure or materials 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 (GCO, 1987).

The hillsides of Hong Kong often have a dense cover of vegetation and a mantle of weathered rock and colluvium both of which tend to obscure features of interest. Although major zones of weakness (defined in Section 4.2.3), 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.

Aerial photographs may be obtained from the Survey and Mapping Office of the Buildings and Lands Department. Good quality photographs are available for most areas and for various years from 1963 onwards, at scales of 1:2 000 to 1:40 000.

3.4 PLANNING OF GROUND INVESTIGATION

3.4.1 Approach

Ground investigation for a typical cavern scheme consists of investigation for the caverns themselves and for accesses and portals. The latter follows procedures and patterns common to ground investigation for surface structures and they are normally aimed at specific localities. The investigation for the caverns is different in that it is aimed at establishing the geology and hydrogeology of the proposed site, including the location of major discontinuities, in particular zones of weakness, commonly with only slight reference to a specific position, elevation or orientation of the caverns.

Ground investigation is used to develop and refine a geological model which is given its first tentative shape during the evaluation of existing data (see Section 3.3). Two or more stages of ground investigation are required for most schemes. Each stage should be designed on the basis of the model and provides, in turn, data for refinement of the model.

3.4.2 Geological Model

The strategy for ground investigation is to proceed from the simple, low cost investigation to the more costly and complex. For the investigation to be effective, it must be based on a geological model, a working hypothesis, which is improved as data is collected.

As indicated in Section 3.3, the initial working hypothesis is made on the basis of an assessment of existing data and a preliminary visit. This is followed by a detailed engineering geological mapping of the cavern site and environs. The area to be covered must be sufficient to give a good understanding of the geology, in particular the larger scale structures that might affect the cavern scheme.

The results of the engineering geological mapping are used to develop the geological model, or a series of models if the data allows alternative interpretations, particularly in respect of the location of zones of weakness.

Ground investigation, using the methods described below, is required to develop the geological model and to give other geotechnical information, including information needed for the development of a hydrogeological model. Engineering geological mapping alone rarely yields sufficient data to define an unambiguous and complete geological model of a site. In particular, the zones of weakness critical to the scheme usually cannot be confidently located without carrying out ground investigation.

3.4.3 Ground Investigation Methods

Ground investigation for cavern schemes progresses, as with all ground investigation, from low cost, simple surface observations to more costly deep drillholes and specialist testing. Geoguide 2 (GCO, 1987) describes a range of techniques which can be considered for investigating a cavern site. Of these, only a few are normally employed, the common methods and sequence being:

- (a) engineering geological field mapping,
- (b) surface seismic refraction surveying,
- (c) rotary drilling and
- (d) insitu and laboratory testing.

In some cases, other methods such as cross-hole seismic surveying with tomography and trial adits and shafts with large-scale testing, are justified by their cost-effectiveness (see Sections 3.6.1 and 3.6.5).

3.4.4 Information to be Obtained

The ground investigation should proceed from investigation of the large-scale features that may dominate engineering and economic feasibility to the factors that would normally have only a marginal effect. First priority must be given to obtaining information on:

- (a) rock type,
- (b) joint characteristics including joint frequencies and orientations,
- (c) location and orientation of weakness zones,
- (d) depth of weathering and
- (e) groundwater conditions and field permeability.

At later stages of project development, it may be desirable or necessary to obtain site specific data on some or all of:

- (f) rock strength,
- (g) insitu stresses,
- (h) rock deformation characteristics and
- (i) chemical properties of rock and groundwater.

Rock strength is only important for design when it is low compared with the applied stresses. Most Hong Kong rocks are strong when fresh and strength measurement for intact rock therefore has a low priority in ground investigation. Typical uniaxial compressive strengths for common Hong Kong rocks are given in Table 4. Rock strength may, however, be of importance from the point of view of drillability, and testing for this purpose is discussed in Section 3.7.12.

In addition to the factors listed above, rock temperature can be important in the design of air conditioning and ventilation plant and for the design of heated, chilled and deep freeze stores. Unpublished data from sites at Cape Collinson and Mount Davis suggest that the rock

temperature at moderate depth is about 23° which is the annual average of the mean daily temperatures for Hong Kong. The ambient rock temperature should be measured for each cavern scheme investigated.

3.5 ENGINEERING GEOLOGICAL FIELD MAPPING

Chapter 9 of Geoguide 2 (GCO, 1987) sets out the principles and requirements for geological mapping. For cavern and tunnel projects, more emphasis on the rock structure than is indicated by Geoguide 2 may be appropriate. Particular attention should be paid to locating and describing zones of weakness, such as faults and heavily jointed or crushed zones. The mapping should be concentrated primarily on information useful for estimating the rock conditions at depth and on surface features that might 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 aid the understanding of the solid geology.

The area covered by the field mapping should be considerably larger than the proposed site in order to obtain an impression of the main geological structures of the area.

Guidance on the description of rocks, rock masses and discontinuities is given in detail in Chapter 2 of Geoguide 3: Guide to Rock and Soil Descriptions (GCO, 1988).

The starting point for geological mapping of the site and environs is 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 type including strength, colour, texture, weathering and alteration, structure and discontinuities including location, orientation, spacing, persistence, roughness, aperture, infilling and seepage. Where fresh rock is exposed, rock classification systems such as the Q-system or RMR method may be applied (see Sections 4.5.2 and 4.5.3). Rock classification from surface exposures of weathered rock should be used with caution as such rock may not be representative of the conditions at depth.

The data collected should be assembled on plans and sections. Joint data may be represented graphically as joint rosettes or polar diagrams (Figure 2). Joint rosettes lend themselves to easy visual interpretation, but polar diagrams give more complete information and can be used analytically (Hoek & Brown, 1980). A tentative model of the geology can be developed on the basis of the desk study and this forms the basis for planning subsequent investigations.

The engineering geological 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.

3.6 FIELD INVESTIGATION

3.6.1 Extent of Ground Investigation

(1) *General.* Chapter 10 of Geoguide 2 (GCO, 1987) gives general guidance on the extent of 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 the 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.

Seismic refraction survey lines should be connected to form a closed loop or grid. The area covered should always be greater than the proposed caverns. The number of survey lines and the amount of data collected should be sufficient for confident interpretation of the results, i.e. low velocity zones should be located with confidence. Sufficient data should also be collected to give a firm indication of the depth of weathering for the site as a whole, with appropriate details along the proposed access tunnels and portals. The extent of the survey 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. Section 3.6.2 gives a brief consideration of other geophysical methods.

(3) *Drillholes.* The number of drillholes required depends on the geological model developed from the mapping data and the geophysical surveys. Their number must be sufficient to resolve uncertainties in the geological model, 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.

Investigation holes should be deep enough to investigate the rock that might affect or be affected by the caverns. The depth should be sufficient to reveal the structure of the rock and to locate discontinuities and weakness zones of importance to cavern design. Some holes should penetrate below lowest probable floor level of the proposed installation by at least 5 m and maybe half the cavern span or more, depending on the uncertainties in cavern location at the time of investigation and the geological factors. Section 10.7.8 of Geoguide 2 (GCO, 1987) indicates that investigation holes for tunnels should be taken to a generous depth below the proposed invert level because changes in design may result in a lowering of the tunnel and because the zone of influence of the tunnel may be extended by the nature of the ground at greater depth. This has some validity also for access tunnels, although the possibility of substantial changes in level is limited by the portal levels and the normally relatively short access tunnel lengths.

3.6.2 Geophysical Investigation Methods

(1) *General.* Chapter 33 of Geoguide 2 (GCO, 1987) gives an outline of the capabilities of various geophysical survey techniques used in site investigation. 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.

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 methods have been employed from time to time. Electrical resistivity measurements can reveal weakness zones and details of superficial deposits. 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.

(2) *Seismic Refraction Surveying.* The geophysical 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 the 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 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 in essence of placing geophones at fixed spacings 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 230 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 seismic velocity increases with the density of the material. 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 3. 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.

The survey results are presented as cross-sections showing the various layers and zones with the velocities indicated (Figure 4). 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 (Figure 4).

The depth 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. Explosives can be used in urban areas providing proper care and control is exercised. The explosives must be discharged in a borehole of sufficient depth to avoid damage to surface facilities or hazard to the public. 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.

Permits for the use of explosives must be obtained from the Mines and Quarries Division of the Civil Engineering Department for each survey undertaken. The contractor who carries out the survey is required to submit an application to the Mines and Quarries Division, attaching a method statement and an up-to-date plan showing the area where the proposed seismic survey is required, the survey lines and hole positions, nearby structures, power lines and services. The staff of the Mines and Quarries Division will then visit the site and will advise on the pre-licensing requirements, such as the types of protective covering required (if any), allowable charge weight per hole, employment of a registered shotfirer, registration of his firing equipment and taking delivery of explosives. If site access is difficult, the contractor may be required to clear the survey lines first. A licence will be issued when the pre-licensing conditions have been complied with.

At noisy locations, the 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.

(3) *Cross-hole Seismic Investigation.* The cross-hole seismic method can provide information on the structure of large volumes of the 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 holes drilled in the same plane. As this requirement is frequently at variance with the hole locations required to discover the geology of the site, additional holes 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. The expenditure on this investigation (as for all investigations) 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 structure where this is critical for successful construction.

Traditional cross-hole seismic investigation commonly gives only average seismic velocities between holes and variations with depth, in igneous geological settings. Computerised acoustic tomography uses the cross-hole technique and it can produce a detailed picture of the rock structure (By, 1987). **Figure 5** shows the principles of the cross-hole seismic method 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.

3.6.3 Drillhole Investigation

Drillhole investigation is the most common and direct way of examining the rock mass at depth. Because of the limitations of field mapping and aerial photograph interpretation, a drilling programme, with or without preceding geophysics, will generally be necessary for 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 (GCO, 1987).

Problems can occur with the rock collapsing into the drillholes and procedures to stabilise such holes should be included in the site investigation contract. The stabilisation of the 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.

High quality core recovery is essential. This means that an appropriate drillhole diameter and core barrel should be chosen. Likely rock strength and rock structure, in addition to the depth to be attained, will influence the choice.

Drillhole diameters are governed by the depth of hole, 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.

The drillhole log must give comprehensive information as described in Geoguide 2 (GCO, 1987). The core logs must contain sufficient data for use in the recommended rock classification systems. Colour photographs of the cores should also be taken as a permanent record, even though the cores are normally stored at least until completion of construction.

All drillholes must be fully grouted upon completion to avoid water problems should the holes be encountered during tunnel or cavern construction. This applies also to the section of hole outside the reaction length of piezometers.

3.6.4 Hydrogeological Investigation

Investigations are required to establish the groundwater regime. An understanding of the water tables is required to assess the effects of drainage into underground openings and to design groundwater containment for certain cavern uses. Predictions of groundwater changes are considered on the basis of rock mass permeability, water table levels, recharge and drainage into the excavations. This section deals with water table and piezometric observations. Section 3.7.2 deals with permeability measurements.

Geoguide 2 (GCO, 1987) describes methods of determining groundwater pressure. Water level observations in investigation holes give the first information on the groundwater and this forms 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.

3.6.5 Trial Shafts and Adits

Trial shafts and adits provide access to rock exposures that do not naturally exist, and are used to resolve particular problems of design or of construction planning. However, they are expensive and should only be used when the cost of the 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 the rock mass in critical areas, such as the locations, orientations and strength and deformation properties of significant discontinuities and zones of weakness.

The main objectives of making trial shafts and adits are:

- (a) investigating the rock mass structure,
- (b) obtaining detailed information on particular zones in the rock mass,
- (c) insitu 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.

ISRM (1981) gives detailed descriptions of sampling procedures and field tests for rock.

3.7 FIELD AND LABORATORY TESTING

3.7.1 General

Section 3.4.4 gives a list of priorities for data collection. The testing requirements must be evaluated for each scheme in terms of technical requirements, budgets and stage of project development. Items (a) to (e) of the list in Section 3.4.4 are covered by the drilling programme, geophysical surveys and field mapping. The remaining items, including rock temperature, are discussed in this section.

3.7.2 Permeability Testing

Permeability testing is normally carried out as packer (water absorption) tests as described in Section 21.5 of Geoguide 2 (GCO, 1987). 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 the proposed cavern construction. Correlation of test results with the core logs can give an indication of the openness of fissures and thus a general idea of the state of stress and stress variations.

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

3.7.3 Joint Orientation

(1) *Impression Packers and Core Orientators.* The presence and character of discontinuities within the rock mass are the major influence on the stability of caverns and they need to be examined in particular detail. To obtain information on the orientation and inclination of these, core orientation devices are often used. In Hong Kong, an inflatable packer covered with parafilm (the drillhole impression packer test) is the most common investigation method. Various other devices for orienting the rock core are manufactured. Section 21.8 of Geoguide 2 (GCO, 1987) gives details of the methods.

(2) *Closed Circuit Television Surveying.* Closed circuit television surveying is a useful means of observing and recording joints in boreholes. Connected to a video recorder, the video camera will provide a permanent and detailed record of the joints. The method has advantages over impression packers and core orientation devices in that records of heavily

fissured and broken rock zones can also be made. Mechanical methods commonly fail in these circumstances.

3.7.4 Borehole Surveying

Deep boreholes, particularly when inclined, can 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 the hole. The boreholes may therefore have to be surveyed.

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

- (a) *Photographic probes*, where the direction (measured by magnetic compass) and inclination of the holes is recorded on film. Single-shot and multiple-shot instruments are available, the latter being quicker in use as they do not have to be lifted to the surface between records.
- (b) *Gyroscope instruments*, with photographic or electronic data collection. These instruments can be used in drillholes subjected to magnetic disturbance, for example in cased holes.
- (c) *Inclinometers*, where the inclination of the hole is measured by electronic accelerometers.

The choice of method is dependent on availability and suitability of instruments, cost and the accuracy required. For most purposes a multiple-shot photographic instrument (an Eastman multiple-shot instrument or equivalent) is satisfactory, but single-shot instruments may also be used.

3.7.5 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 insitu stress testing is to check that the stresses will not cause undue problems in cavern excavation or support 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 caverns at shallow depth. Further discussion on the influence of insitu rock stresses on cavern design is given in [Section 4.2.6](#).

Three methods of insitu stress measurement can be considered:

- (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 following ISRM (1986).

In the over-coring method, the triaxial state of stresses can be assessed for every point of measurement in a drillhole. One method of over-coring, the SSPB method, is illustrated in Figure 6 (Hiltscher et al, 1979). With this method, tests can be performed in drillholes at any depth down to several hundred metres. However, the typical instrumental error dictates that tests should not be attempted at depths of less than 100 m (unless a large and statistically valid number of tests are made). Other methods such as the USBM and CSIRO may be limited in their application to depths of less than 50 m (ISRM, 1986). 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 may be required, perhaps 20 or more, to obtain a statistically significant result. However, even a few test results can indicate if stress-related problems are likely to occur in cavern construction.

Hydraulic fracture tests can be done in both shallow and deep drillholes as shown in the schematic layout in Figure 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 open. 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 3.7.3. A common assumption in interpretation is that the axis of the drillhole corresponds to an axis of principal stress and that this stress is known. Normally the tests are done 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 interpretation of the triaxial state of stress may be made even when existing fissures are present (Cornet, 1986).

Flat-jack tests can be done in accordance with ISRM (1986), but are not commonly used for ground investigation as they require the exposure of an exposed rock surface at depth. Twist & Tonge (1979) report successful tests using flat-jacks during the construction of the Aberdeen road tunnels.

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

Experience from stress measurements by over-coring using the SSPB method, in volcanic rocks in Hong Kong (Söder, 1990; Ingevall & Strindell, 1990), shows that up to half of the measurement attempts failed because of close fissuring. The test results indicated that the insitu stresses were unexceptional with an indication that the horizontal stresses were higher than the vertical stresses. No other firm conclusions could be made. The Leeman Cell was successfully used as reported by Twist & Tonge (1969) during construction of the Aberdeen tunnels, but there has been no report on its use in Hong Kong in recent years.

Hydraulic fracture tests have been carried out at Tsing Yi Island (Rummel & Konietzky, 1990). Some 70 tests were done and ten fissures were induced. The remaining tests were done by jacking existing fissures. The tests indicated minimum principal stresses in the range 0.8 to 2.4 MPa.

The tests at the various locations were influenced by topography and were all carried out at relatively shallow depth. Adjacent fissures also affected the results. Generally valid conclusions on stresses cannot be given. The data, however, suggest that the minimum horizontal stresses may be twice the theoretical overburden pressure or higher, and that the vertical stress is commonly less than the theoretical overburden pressure. The fissures induced at Tsing Yi suggest a minimum horizontal stress direction between N50°W and N10°E.

3.7.6 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 and the point load test (ISRM, 1981). These tests should be performed in limited numbers on fresh rock to confirm the high strength expected at most cavern sites. The tests on weathered rock may have more significance because of the potential problems of excavation in such materials.

(2) *Joint Shear Strength.* The rock classification methods referred to in [Section 3.5](#) do not require experimentally determined joint shear strengths as an input. For particular stability problems, such as sliding wedges in cavern walls, joint shear strength may be important. Experience has shown that visual assessments of joints are generally adequate to assess joint 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 insitu tests be justified. The tests may be shear box tests following ISRM (1981) or tilt tests following Barton & Choubey (1977) and Barton & Bandis (1990). Hoek & Brown (1980) gives recommendations on how to treat stability problems, including determination of joint properties.

3.7.7 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 it, the deformation parameters can be measured by laboratory

uniaxial or triaxial tests (ISRM, 1981) or by using a high pressure pressure-meter or Goodman Jack (ISRM, 1986). Shear and normal stiffness of rock joints can be obtained by shear box tests (ISRM, 1981) or tilt tests (Barton & Choubey, 1977).

Any mathematical modelling that may be required demands data on the properties of joints and intact rock and tests have to be performed that are appropriate for the model.

3.7.8 Chemical Properties

The chemical properties of the rock that are of interest are those related to re-use of the rock as aggregate and the effects of any leachates in the groundwater that may be detrimental to constructions within caverns, particularly concrete, or minerals that might react with fluids stored in the caverns.

In situ weathered rocks and their associated soils in Hong Kong are generally not chemically aggressive (GCO, 1987), and the same applies to the groundwater derived from these materials. Nevertheless, routine tests should be carried out on soil and groundwater for sulphate content and pH-value in accordance with BS 1377: Part 3: 1990, Methods 5 and 9 (BSI, 1990). Other chemicals that have caused problems elsewhere are sulphuric acid produced by alum shales upon exposure, and sulphates that react with Ordinary Portland Cement. Where salt water is present, it can cause severe corrosion of construction steel including concrete reinforcement. Sulphur minerals can react detrimentally with oil products. Appropriate tests should be carried out for these and any other special cases.

3.7.9 Mineralogy and Swelling Properties

It is prudent to carry out mineralogical analysis of fault gouge material 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.

3.7.10 Rock Temperature

Rock temperature may be recorded in boreholes 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.

3.7.11 Blasting Trials

Blasting trials in boreholes for the purpose of verifying the attenuation coefficients in the vibration attenuation equations have been attempted from time to time but have generally not improved the predictability of the equations. This is because of the differences in confinement of the charges in the trial borehole blast and the production blast-holes, and because a single detonation in a borehole cannot simulate the delays in detonation between

different blast holes. At the design stage, vibrations are best predicted using available empirical equations (see Section 5.7.2).

Blasting trials are best done during the first stages of construction. Charge weights per delay can be correlated with measured vibrations and any control measures required to attain safe vibration limits can then be set.

3.7.12 Tests for Drillability

The drillability of a rock is defined as the rate of drill bit penetration into the rock. The Drilling Rate Index (DRI), also known as the drillability index, gives an indication of the drillability of a rock. The DRI is calculated from parameters: the S_{20} value from the Brittleness Test and the S_j value of the Sievers Miniature Drill Test. The tests are described by Tamrock (1986) and Selmer-Olsen & Blindheim (1970).

The Bit Wear Index (BWI) is a measure of expected drill bit wear and it can be determined from the DRI and the Abrasion Value (AV). The latter value is derived from a laboratory test (Tamrock, Selmer-Olsen & Blindheim). BWI can also be calculated directly from AV.

The DRI and BWI are related to the quartz content of the rock as shown in Figure 8. The correlation may be used as a basis for estimating the drillability of the rock and this procedure may be sufficient without the need to carry out any test other than quartz content analyses.

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4. CAVERN DESIGN

4.1 DESIGN APPROACH

4.1.1 Basis of Cavern Design

In common with other complex constructions, the design of a large underground space is an iterative process where a series of factors influence the final result. The nature of the factors and the constraints on design are different for the above- and below-ground schemes and give rise to a slightly different approach. There are four major elements that underlie the approach to the engineering of caverns:

- (a) The rock is used as a structural material. Strong to extremely strong rock has a compressive strength that is typically greater than that of concrete. The uniaxial compressive strength, σ_c , for such rock is typically in the range 100 MPa to 200 MPa and more, see Table 4.
- (b) Geotechnical design is based primarily on precedents, i.e. empirical methods.
- (c) The design is related to construction procedures.
- (d) The design is optimised on the basis of geotechnics, usage, construction methods and external factors.

In addition to these elements, numerical methods can be used to predict problem areas and to extrapolate experience, with monitoring used to verify the design as necessary.

4.1.2 Design Sequence

Cavern design is, as indicated above, an iterative procedure where a series of elements in the design are optimised to produce a cost-effective cavern installation. The design commonly progresses in the following order, but with numerous iterative loops:

- (a) identification of the geometrical and physical requirements for the cavern,
- (b) identification of areas with geology suitable for cavern construction,
- (c) evaluation of the topography in relation to the geometrical requirements,
- (d) location of suitable access to the underground facility,
- (e) evaluation of geological and hydrogeological data,

- (f) determination of optimal location, orientation, lay-out and geometry for the cavern or cavern system based on the above factors,
- (g) optimisation of the design with respect to cavern use and construction methods, which may include modification of the cavern use or process, and
- (h) evaluation of rock support measures.

4.1.3 Rock as a Structural Material

The basic aim of underground excavation design is to utilise the rock itself as a structural material. The support works are normally aimed at improving the local stability of the rock.

The key to economical design of underground space lies in the recognition of weaknesses in the rock structure in the design of the rock support system. Experience from several hundred caverns and tunnels constructed in recent decades has added considerably to our understanding of how effective rock support may be achieved. These empirical data are the basis for the two most common methods of predicting rock support requirements from geotechnical data.

4.1.4 Rock Classification Systems for Determining Rock Support Requirements

Rock support assessments may be based on systems of rock mass classification which also give support recommendations in accordance with geological conditions and the intended use of the underground space. The Q-system and the RMR method are empirical methods based on case records. They are used widely internationally. Both systems are suited to Hong Kong conditions. Outlines of the methods are given in Sections 4.5.2 and 4.5.3, respectively.

4.2 CAVERN LOCATION AND ORIENTATION

4.2.1 Factors in Optimising Cavern Location and Orientation

The choice of location for a scheme is the most important single decision in the whole design and construction process, as it determines the quality of the rock in which the underground space will be excavated. Factors other than engineering geology can constrain the choice of location and a favourable site may have to be found within a limited area.

The constraints on cavern location that should be considered are frequently related to the location of portals, access tunnels, 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. The important factors in this optimisation are:

- (a) adequacy of rock cover,
- (b) avoidance of weakness zones or the crossing of them in the shortest possible distance,
- (c) avoidance of adverse orientation relative to major joint sets,
- (d) sufficient depth below the groundwater table, for some uses,
- (e) avoidance of rock with abnormally low stresses, giving reduced confinement and
- (f) avoidance of rock with very high stresses.

4.2.2 Minimum Rock Cover

For shallow caverns the minimum rock cover must be defined. The rock cover should be sufficient to give adequate normal stresses on joints such that the roof and walls will be, as far as possible, self-supporting. The minimum rock cover required can be a major constraint on cavern location. The minimum rock cover is determined from an assessment of many factors, which may include:

- (a) the quality of the geological information,
- (b) the rock properties,
- (c) thickness of superficial deposits,
- (d) depth of weathering,
- (e) the cavern span and
- (f) cost implications.

As a general rule, the minimum cover of strong rock should be not less than half the cavern span. In feasibility studies, based on limited information, it is prudent to assume a cover of strong rock of not less than the cavern span.

Large span tunnels and caverns have been constructed with rock cover down to a quarter of the cavern or tunnel span. In the Vålerenga road tunnel in Oslo, Norway, rock cover down to 3.5 m was adopted for a span of 12.6 m (Berdal Strømme, 1988). In general, reduced cover increases the amount and cost of ground investigation and rock support work and this cost must be offset by advantages in adopting reduced cover. Reduced rock cover is normally limited to small areas, such as the section of cavern closest to the portal. Design

of such low cover is only acceptable where the fresh rock surface is well defined by detailed site investigations.

4.2.3 Weakness Zones

Weakness zones are defined as zones that are weaker than the surrounding rock. The thickness of a weakness zone can range from a few centimetres to several hundred metres. The material in these zones may be described, as for all rock, in accordance with the principles described in Geoguide 2 (GCO, 1987). Weakness zones have many origins and can be weak rocks, faults, heavily fissured zones, hydrothermally altered rocks and deeply weathered zones.

The cost of cavern construction can be strongly influenced by the presence and nature of the weakness zones encountered. Many major problems in underground construction are related to such zones and the cost implications can be considerable. These problems commonly result in poor stability of unsupported rock and heavy inflows of water from fissured zones. It is therefore important that weakness zones are identified and, if possible, avoided. If the space between faults, gouges, crushed zones or weathered seams is too small for the cavern space, the different zones have to be evaluated and a location chosen that gives the minimum excavation in these difficult materials. Alternatively, a different cavern geometry that fits between the weakness zones may be considered.

Consideration should be given to the orientation of the weakness zones in the rock mass, as steeply dipping features can have a major influence on the stability of walls, while flat lying ones can be a threat to roof stability. Figure 9 shows typical sections with heavy rock support measures located according to the orientation of major weakness zones.

4.2.4 Jointing

The orientation of joints with respect to the axis of the excavation has an influence on the stability of a cavern and the amount of overbreak. The two main methods of rock classification (the Q-system and the RMR method described in Sections 4.5.2 and 4.5.3 respectively) take into account the influence of joint orientation on rock classification and support requirements in different ways. The amount of overbreak has been shown to increase if the angle between the excavation axis and the strike of a major joint set is small, say less than 30° (Thidemann, 1976). The character of the joints can be a major influence on the orientation of caverns. 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).

When other constraints can be satisfied, optimisation of the direction of the excavation axis with respect to joint orientation should be achieved. It is therefore necessary to carry out a detailed survey of the bedding or foliation and the jointing of the rock mass. For openings situated at shallow or intermediate depths, the longitudinal axis of the cavern is ideally oriented along the bisection line of the largest intersection angle of the strikes of the two dominant sets of discontinuities, be they joints, bedding or foliation. Close alignment

with any further joint sets should be avoided, so as to reduce the extent of potentially unstable rock.

4.2.5 Groundwater

The location of the groundwater surface and predictions of changes created by the underground openings can be important considerations in determining the elevation of a cavern scheme. Some schemes require a constant seepage towards the caverns which ensures that the groundwater will not be polluted by materials and activities in the caverns. For some uses, such as oil products stores and gas stores, the groundwater pressure is used to confine the product or gas and is thus a prerequisite for successful implementation. For such cases, the groundwater surface has to be maintained with impressed water curtains. For other applications the location of the groundwater is less important, and some caverns should ideally be placed above the groundwater surface to obtain as dry a cavern as possible. The need to minimise the effects on the environment or existing structures and facilities due to any changes in groundwater surface may be a constraint on the cavern location.

4.2.6 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, can be significant. Rock stresses are also influenced by structural heterogeneities, such as major weakness zones where highly anisotropic stress conditions can occur.

Stresses within the rock influence the stability of excavations and, within limitations, increased stresses give increased stability. Increased stresses in the rock give rise to greater friction forces on joints and thus greater rock mass strength. The stress-related problems that may occur in caverns constructed in Hong Kong are more likely to be caused by low stresses. For most sites excavation stability can be expected to improve with increased thickness of cover.

Stresses in hard rocks are normally anisotropic. The degree of anisotropy can influence cavern stability and therefore optimal shape. Anisotropic and high stresses and accompanying stability problems can be associated with caverns constructed in high valley sides. There is no clear evidence that such conditions occur in Hong Kong, although the height and steepness of some mountains might suggest these conditions. Investigations for caverns in such locations must therefore establish if these conditions exist and appropriate designs must be adopted.

High tectonic and residual stresses could have an influence on the location and shape of shallow caverns, but there is no evidence to suggest that this might occur in Hong Kong (see Section 3.7.5).

4.3 CAVERN LAYOUT AND SHAPE

4.3.1 Introduction

The design of cavern geometry and layout of a system of caverns is 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 opening, i.e. the total height and arch shape, influences the cost of excavation and support.

The main parameters defining cavern layout and geometry are the cavern size and shape and the spacing between caverns.

The empirical approach to design of underground space in hard rock conditions is based on experience from numerous caverns and tunnels constructed in various types of rock mass under varying stress conditions as exemplified in [Tables 1 and 2](#). This approach is used for tunnels and caverns of conventional size, say up to 25 m span, constructed in good rock. Factors other than rock stresses normally govern design and sophisticated design tools are therefore not in common use (see [Section 4.6](#)).

However, large span caverns, caverns in difficult ground conditions and multi-cavern schemes are commonly the subject of stability and stress distribution analyses using various methods. Such analyses are used primarily to extrapolate experience (i.e. the empirical design rules) to cover conditions not previously encountered. Under no circumstances is mathematical analysis a substitute for experience.

4.3.2 Cavern Shape

(1) *Fundamentals*. The rock mass is a discontinuous material of low tensile strength. The basic design concept for an underground cavern is to aim at evenly distributed compressive stresses in the rock mass bounding the excavation. This is best obtained by giving the space a simple form with an arched roof. Intruding corners should be avoided as the rock will be locally de-stressed, resulting in greater overbreak during blasting or an unstable situation after blasting ([Figure 10](#)).

The cross-section of a cavern should be optimised, within given restraints, 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. All these costs must be quantified, at least relative to each other, for the prevailing rock conditions before the cavern cross-section can be optimised. [Figure 11](#) shows typical curves that illustrate the relationships of these costs to cavern dimensions.

(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 12](#)). This roof shape is not commonly altered to suit particular geological structures. Reducing the roof arch height increases stability problems and fall-out during blasting but may be justified if the dominant joints have a shallow dip. However, improved stability that might arise because of

increasing the roof arch height, with a corresponding reduction in roof support costs, is normally inadequate to offset the additional rock excavation cost. Justification for increasing the roof arch height is commonly the use of the space under the arch for ducts and services.

(3) *Wall Height.* Cavern walls are normally vertical. This suits the method of excavation and yields little unusable space. Wall stability is a function of wall height, the insitu stresses and the orientation and engineering properties of the principal joint sets. The flat wall surface precludes any substantial arching action and high walls tend to be unstable.

Major joints 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.

Joints with shallow dip favour wall stability as the dominating vertical stresses in the walls increase joint friction. Conversely, steeply dipping joints with strikes parallel to the wall reduce stability as the horizontal stresses on the joints 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 other parts.

From the data collected to date, the insitu stresses at depths down to 100 m or so are modest in Hong Kong, with no evidence of high horizontal stresses although these can be higher than the vertical stresses (see Section 3.7.5). These stress conditions would not normally cause a general problem with wall stability.

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

4.3.3 Spacing between Caverns

(1) *Pillars.* The width of pillars depends primarily on the rock quality, the orientation of the discontinuities, the cavern spans and heights and any openings formed in the pillars. Insitu stresses can also affect pillar widths, especially for deep caverns. Pillar widths are normally equal to between half and the full cavern span or height, whichever is the greater. At the preliminary planning stage pillar widths should be conservative. As planning progresses on the basis of improved geological data, narrower pillars may be considered.

Pillar widths are normally determined on the basis of judgement and simple analysis. Estimates of acceptable pillar width can be made on the basis of assuming kinematically possible sliding on unfavourable joints and calculating the factors of safety. Figure 13 illustrates such a procedure. Estimates of vertical stresses and joint shear strengths in the pillar are required for this type of analysis.

Pillar widths should in general be designed so generously that mathematical modelling is not necessary. The cost of collecting the rock data and the analysis normally exceeds the potential savings in construction costs. Pillar widths designed to the theoretically defensible

limit will commonly also give rise to additional excavation and support problems. However, at some sites narrow pillars may be necessary because of site availability, faults and other restrictions and the additional complications and costs will have to be accepted.

(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 upon the stresses in the surrounding rock mass decreases with increasing distance from the opening. If isotropic virgin stresses and a circular opening are assumed, the induced stresses are insignificant at a distance from the opening of 0.5 to 1 times the diameter of the opening. For caverns at modest depth, which will be the common case in Hong Kong applications, the induced stresses will be very low compared to the rock strength. Hence, this problem has to be considered from the viewpoint of joint geometry relative to the 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, e.g. Franzefoss Bruk, Norway, with cavern span, height and pillar width of 13 m, 17 m and 8.5 m, respectively, with a vertical separation of only 8 m.

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

- (a) increased blasting and support costs,
- (b) the overbreak and loosening of rock beyond the rock face in both the lower and upper caverns and
- (c) the risk of outfall of rock that might impair the stability of the floor of the upper cavern.

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

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

4.4 INFLUENCE OF CONSTRUCTION METHODS AND COSTS ON DESIGN

Construction methods and the costs of the various operations involved have to be taken into account in the design. Sections 4.2 and 4.3 discuss these factors with respect to cavern

location, orientation and shape. The construction methods in a properly optimised design, also influence the layout of the scheme. The total design has to take into account:

- (a) construction sequence,
- (b) access to different levels of excavation,
- (c) drill and blast cycle, including different costs for top heading and bench excavation,
- (d) mucking-out and spoil transport,
- (e) ventilation of blast gases,
- (f) work on multiple faces,
- (g) over-break,
- (h) stability problems and support requirements and
- (i) drainage and pumping of seepage water.

There is a cost and a time element attached to each of the above items. The construction methods are described in [Chapter 5](#).

4.5 EVALUATION OF ROCK MASS QUALITY AND ROCK SUPPORT

4.5.1 Approach

Preliminary design of rock support may be made on the basis of rock classifications, using either the Q-system or the RMR (Rock Mass Rating) method (see [Sections 4.5.2 and 4.5.3](#)). These methods allow the most suitable type of support to be determined for the various rock classes that have been identified. The methods may also be used for the final design which necessarily involves their use during construction. Both methods may be used in parallel for complex and difficult ground conditions and the results compared. Rock bolt lengths are estimated on an empirical basis taking into account block size in the case of spot bolting, and cavern span in the case of systematic bolting. Design methods for bolts, shotcrete and cast concrete support are described in [Sections 4.5.4, 4.5.5 and 4.5.6](#), respectively.

4.5.2 Q-system for Rock Classification and Support Estimation

The Q-system was developed by NGI (Norwegian Geotechnical Institute) (Barton et al, 1974) and later updated to incorporate developments in rock support technology (Grimstad et al, 1986). The method is empirical and is based on the Rock Quality Designation, RQD (Deere, 1964), and five additional parameters, which modify the RQD value for the number

of joint sets, the joint roughness and alteration (infiling), the amount of water and various adverse features associated with loosening, high stress, squeezing and swelling. This classification system is based on support installed for 212 case histories. The Q-system also takes account of the intended use of the excavated space.

Figure 14 shows a simplified diagram of rock support measures based on the Q-system. The Q-value is expressed by the formula:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where

- RQD = Rock Quality Designation
- J_n = number of joint sets
- J_r = joint roughness parameter
- J_a = joint alteration or filling parameter
- J_w = water pressure or leakage parameter
- SRF = stress reduction factor.

The numerical value of Q ranges from 0.001 for exceptionally poor quality squeezing ground up to 1 000 for exceptionally good quality rock which is practically unjointed. The six parameters, each of which has an importance rating given in Tables 5 to 10, can be estimated from site investigation results and verified during excavation. In combination they represent:

- (a) block size by the quotient RQD/J_n ,
- (b) inter-block shear strength by the quotient J_r/J_a and
- (c) active stress by the quotient J_w/SRF

The equivalent span or height in Figure 14 is found by dividing the actual excavation dimension by a factor, the Excavation Support Ratio (ESR value), representing the safety requirement for the use of the space. For underground power stations the factor is 1.0, while it is 1.0 to 1.3 for road tunnels and sewage treatment works and 1.6 for water tunnels. A detailed listing of ESR values is given by Barton et al (1974) and is reproduced in Table 11.

The support derived from Figure 14 is the permanent roof support. Evaluation of permanent wall support by this method, requires a modification of the Q-value as shown below:

Q-value for roof, Q_r	Q-value for wall, Q_{wall}
> 10	5.0 x Q_r
0.1 < to < 10	2.5 x Q_r
< 0.1	1.0 x Q_r

In order to calculate the Q-value for tunnel or cavern intersections, a joint set number value of $3 \times J_n$ is normally used.

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 gives only probable amounts and support types to be used. During construction of the underground opening, the rock support types and quantities should be adapted to the observed rock conditions. The heterogenous nature of rock masses precludes the design of definitive, cost-effective support systems prior to excavation.

To simplify the classification of the rock mass quality and the rock support evaluation, it is common practice to divide the Q-value range into classes as indicated on **Figure 14**.

Apart from the engineering requirements for support, there is a strong psychological aspect that affects the minimum support installed, particularly in cavern roofs. In caverns with large spans and height, the construction workers often require some bolting and full shotcrete support to the roof to feel safe. This practice frequently results in good economy as the alternative may be substantial and costly scaling work to prevent the later downfall of small stone.

Useful information on the use of the Q-system for cavern design in Hong Kong is given by Barton (1989).

4.5.3 RMR Method of Rock Classification

The RMR method of rock classification was developed by Bieniawski (1974,1976, 1979). The classification in its developed form (Bieniawski, 1976) is based on the assessment of five parameters:

- (a) uniaxial compressive strength of intact rock,
- (b) rock quality designation, RQD,
- (c) spacing of discontinuities including joints, faults, bedding planes, etc. using a classification according to Deere (1964),
- (d) condition of the joints including aperture, continuity, roughness, wall condition and infilling and
- (e) groundwater conditions where the effects of water on rock mass strength are taken into account.

The parameters are given ratings taken from **Table 12**. **Table 13** includes a rating adjustment that takes into account joint orientations and has to be read in conjunction with **Table 15** where the descriptive terms of **Table 13** are explained. The total rating for the rock is obtained by adding the ratings for each parameter and the adjustment for joint orientations (see **Table 14**). The rock is classified as follows:

Rating	RMR Class	Description
< 20	V	Very poor rock
20 to 40	IV	Poor rock
40 to 60	III	Fair rock
60 to 80	II	Good rock
> 80	I	Very good rock

Bieniawski (1976) relates the rating to stand-up time. This may be useful for weak rocks but is of little use for hard rocks such as those common to Hong Kong. The ratings may be used as a guide to determining the support required (see Section 4.5.4). A full discussion of the RMR method of rock classification and support recommendations is given by Hoek & Brown (1980).

Bieniawski (1976) shows that the relationship between the above rating and the equivalent Q-values is adequately described by the equation:

$$\text{RMR} = 9 \log_e Q + 44$$

4.5.4 Rock Bolt Lengths and Spacings

(1) *Design Basis.* In hard rock tunnelling and cavern construction, the term 'bolt' normally refers both to tensioned bolts and fully grouted untensioned bolts as the mode of action is dominantly in tension in both cases. The latter type of bolt is termed 'dowel' in the Geotechnical Manual for Slopes (GCO, 1984).

Rock bolts installed to increase general stability in tunnels and caverns are usually not subjected to the design methods applied to bolts and dowels used to stabilise rock slopes, as sliding failure mechanisms normally cannot be defined. Bolt spacings and lengths for tunnels and caverns are determined using empirical design rules as described below.

Where a sliding failure mechanism can be properly defined, analysis can be applied and the stabilising measures can be determined mathematically. The recommendations in the Geotechnical Manual for Slopes (GCO, 1984) should be followed.

(2) *Spot Bolting.* Design of spot bolting is done as excavation proceeds and is actually part of the construction process. Spot bolting is used to secure individual blocks of rock. The size of the block can be estimated from observation of the joints that define the block. The bolt (or bolts) should be long enough to obtain adequate anchorage in stable rock beyond the block. Block sizes are estimated from joint directions and spacings relative to the excavation. The length of bolt should reflect the uncertainty of the block size estimate. The length of bolt should be 1 m to 2 m beyond the block, and preferably not less than 1.5 m, dimension d in Figure 15. The factor that governs the length d is primarily the rock quality. The tensile strength of the bolt can be fully utilised with these anchorage lengths. Section 5.6.4 recommends a minimum bolt diameter of 20 mm. Spot bolting at the excavation face is determined on the basis of experience as the time available precludes analysis. Some bolt

types are pre-tensioned to activate their anchorages and to ensure that these are effective. The pre-stressing of the rock is essentially incidental.

The load capacity of the bolts must be sufficient to support the block. It is good practice to assume that only half of the bolts will be effective, so the number of bolts is doubled, thus giving an added factor of safety over and above the factor of safety given by consideration of the ultimate load capacity of each separate bolt. With pre-tensioned bolts the number of bolts can be reduced depending on the installation procedure and quality, as the pre-tensioning gives assurance that they are carrying load. The extent to which joint shear strength can be taken into account varies with circumstances. For smaller blocks joint shear strength is commonly ignored. The cost penalty of this conservatism can become large for major blocks, particularly sliding wedges in walls, and joint shear strength is then taken into account. A balance between design effort and cost, and installation cost, must be achieved.

(3) *Systematic Bolting.* Systematic bolting is used to achieve a general increase in stability. The bolts are normally installed in a pattern but with some common-sense adjustment. The block size is the principal factor in determining bolt spacing and cavern span dominates the determination of bolt length. The bolting is frequently done in conjunction with shotcrete or fibrecrete and can be installed after its application. Various bolt diameters are in use, dependent on the steel quality and method of installation (see Section 5.6.4). Yield strengths are typically in the range 9 to 12 tonnes.

The rock bolt spacing is conveniently estimated using the Q-system or RMR classifications. Tables giving recommended support measures may be found in Barton et al (1977), Bieniawski (1976) and Hoek & Brown (1980). Bolt spacings of 1 m to 3 m are indicated for most situations depending on the calculated rock quality (see, for example, Figure 14). Bieniawski (1976) gives rock bolt spacings of 2.5 m for RMR 61 to 80, 1.5 to 2 m for RMR of 41 to 60 and 1 to 1.5 m spacing for RMR < 41. No bolting is indicated for RMR > 80. Bolt spacings of less than one metre are not normally considered practicable and alternatives such as straps, shotcrete and fibrecrete should be considered in conjunction with bolting.

There are several means of estimating bolt lengths in common use. A formula such as that given by Schach et al (1979) gives bolt lengths in metres in accordance with common practice:

$$L = 1.40 + 0.184 B$$

where L = the bolt length and
 B = the cavern span.

In any case the length should not be less than that required for spot bolting. Shorter bolts may be used towards the cavern walls, but should not be less than 2 m. Figure 16 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 the holes to take into account

the joint directions, but the designer must take into account the added complication in installation and control.

Rock bolt design for major zones of instability created by seams or persistent smooth joints should be the subject of stability analysis (see Section 4.6.2).

4.5.5 Design of Shotcrete and Fibrecrete Support

Both the Q-system and RMR method indicate the thickness of shotcrete support required for various rock qualities. With the development of the support method, fibrecrete technology has advanced significantly and has largely replaced mesh reinforced shotcrete. The support recommendations given by Barton et al (1977) include a recommendation for very large thicknesses of shotcrete for some situations. This advice should be treated with caution as little advantage can be 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 applied occasionally to small areas of particularly poor rock.

As a general rule, systematic bolting with fibrecrete should be used for permanent support of roofs of caverns that will be occupied most of the time or caverns that contain important processes or machinery.

Shotcrete and fibrecrete support has a better effect if it acts in unison with the rock bolts. The bolt heads should either be incorporated in the sprayed concrete or the bolts may be installed with bearing plates after spraying. In some situations the heads of the bolts may be protected prior to spraying concrete, and the bearing plates and nuts installed afterwards. Plates installed after placing the shotcrete or fibrecrete support may require further sprayed concrete for corrosion protection.

4.5.6 Design of Cast Concrete Support

Cast concrete support is not a realistic means of achieving general stability in cavern excavations, although it can be applied to limited areas of instability such as weakness zones. Stability of the rock is achieved primarily with rock bolts and sprayed concrete. A requirement for concrete lining is for reasons other than achieving primary rock stability and is subject to design on the basis of parameters other than those of rock mechanics. Owners may in some instances require an arched cast concrete roof as insurance against the fall of rock or loosened shotcrete.

4.6 ANALYTICAL AND NUMERICAL METHODS

4.6.1 Introduction

Analytical and numerical methods in cavern engineering can be divided into the following classes:

- (a) limit equilibrium methods for discrete blocks and wedges,
- (b) statistical-analytical methods, e.g. key block analysis,
- (c) numerical continuum methods, e.g. finite element methods and boundary element methods and
- (d) numerical discontinuum methods (distinct element methods).

For an analysis to be useful it must address the factors that are important for cavern design. For example, the design of caverns installed at modest depths in hard rock is normally governed by the weakness zones and a series of intersecting joints, and not by insitu or induced stresses and intact rock strengths. Simple stability analyses of problematic areas using limit equilibrium methods are therefore often used to supplement the support design given by the rock classification systems.

Numerical models are used to extrapolate, and occasionally to check, the empirical methods and designs. Such modelling can increase confidence in a particular design and in interpreting instrumentation results. While many major cavern schemes have been built successfully without any numerical modelling of stresses, strains and joint shearing, etc., in some situations such modelling is an important design tool.

Situations where analytical and numerical methods may be applicable are discussed below.

4.6.2 Stability Analyses

(1) *Sliding Blocks and Wedges.* There are many published methods for determining the stability of blocks and wedges in excavated rock surfaces and the support required to achieve stability. These methods are based on limit equilibrium analysis. Hoek & Brown (1980) present such methods. Similar analyses may be made to check pillar widths and their support requirements (see Section 4.3.3).

(2) *Key Block Analyses.* Key block analyses may be used to determine which blocks in a cavern roof or walls control stability. Securing these key blocks will ensure overall stability. The analyses may be used to predict the likely location and appearance of key blocks using statistical joint data or joint maps taken from excavations when specific key blocks can be identified. The first mentioned use is an aid to recognising the key blocks as such from the joint pattern observed. The ultimate purpose is to apply support where required and in the most cost-efficient way. This method may on occasion be an aid to, but is no substitute for, the judgement of the experienced engineering geologist or tunnelling engineer. The method is described by Goodman & Shi (1985).

4.6.3 Numerical Models

(1) *General.* Fundamental to all numerical modelling of rock excavations is the problem of defining input data. Rock is heterogeneous and large numbers of measurements may be required to obtain statistically valid data. All numerical models represent rock behaviour more or less imperfectly. Although this means that numerical models will not yield

exact solutions, the performance of sensitivity analyses by such methods is useful for the assessment of the relative impact of different parameters. Improved understanding of complex situations can be achieved. A programme of field instrumentation will allow model results to be compared with field measurements. Early implementation of the instrumentation programme will allow timely adjustments to final rock support.

Starfield & Cundall (1988) discuss a methodology for modelling rock mechanics problems and reference to this paper may be made for guidance and necessary cautions. Hoek et al (1991) provide similar guidance with an overview on the use of various numerical modelling methods.

Following Starfield & Cundall, rock modelling problems can be classified into four regions according to the data available and the understanding of the problem (Figure 17; Holling, 1978). In region 1 there is good data but poor understanding, and in region 3 there is both good understanding and good data. Regions 2 and 4 represent problems where data is limited or cannot easily be obtained. Most problems in rock mechanics fall into this group of data-limited problems, whereas problems in structural engineering fall largely into region 3. Attempts have been made to collect sufficient data to move rock mechanics problems into region 3. This may, however, lead to more and more complex models and increasingly expensive ground investigations without significant improvements in understanding and design.

Closed-form solutions that indicate the relations between stress and displacement around underground openings are available for simple excavation shapes in homogeneous ground conditions. These solutions have considerable value in providing a conceptual understanding of rock mass behaviour and in testing numerical models. Solutions for circular and elliptical excavations can be found in Bray (1987).

Numerical models may be classed either as the continuum type or the discontinuum type. For continuum models based on the boundary element method, the free surfaces are divided into elements and the interior of the rock is treated as an infinite continuum. The stresses and strains applied to an element have a calculable effect on the other surface elements and throughout the medium. Thus changes at one surface element will affect all other elements. The method has the advantage that only the boundaries have to be divided into elements, and the far-field stresses are not influenced by the creation of an excavation. Although joints 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 the sophisticated modelling of joint behaviour. Also, in general, the 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 finite element and finite difference methods relate the conditions at nodal points to the state within the 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. Joints can be represented explicitly by means of specific joint elements, but generally in limited numbers.

The discrete element method models blocky materials as a discontinuum and is well suited to modelling these highly non-linear problems. In this approach, the jointed rock mass is represented by 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. The method has been until now primarily a research tool, but recently some understanding has been gained of where, how and when the method may best be applied, although Hoek et al (1991) suggest that further experience in this respect is needed.

Hybrid approaches to modelling can be used advantageously, whereby the desirable elements of each approach are retained and the undesirable aspects suppressed. Thus, the far-field behaviour of material in a finite element analysis may be modelled by linking the outer elements to a boundary element system which models the effect of infinite boundaries well.

(2) *Analysis of Multiple Cavern Systems.* Numerical modelling of multiple cavern systems may assist in understanding the mode of rock mass deformation and in defining potential failure mechanisms. It is important to ensure that the correct problem has been modelled and that there is adequate intellectual control of the process.

Various finite element methods can aid in achieving this understanding, and discrete element and other techniques may be considered where joint deformation is important. Johansson et al (1988) gives a finite difference method for analyzing multi-cavern systems.

(3) *Groundwater Modelling.* Groundwater modelling may be required when the effects on the groundwater regime can be deleterious to existing constructions or where groundwater table maintenance is an essential part of the cavern design, such as with unlined oil and gas storage caverns. The rate of flow into a facility may have important consequences for the construction and operating costs. The rate of inflow, as a function of cavern location and configuration, as well as the effects of drainage systems and grouting requirements, can be studied. In both cases, modelling should be aimed at parametric studies to find the effect of different configurations. A useful reference on groundwater analysis is Rushton & Redshaw (1984).

Finite element methods are available for two and three dimensional studies. Finite difference methods may be used for two dimensional models.

(4) *Thermal Analysis.* Thermal analyses should be carried out for caverns that will operate at temperatures substantially different from ambient rock temperature. Examples of such caverns are cold stores with operating temperatures below -20°C . The analyses are needed to estimate the energy demand to cool or heat the rock during the initial life of the facility and to estimate the energy required to maintain the temperature gradients. The migration of ice fronts and their location in their ultimate development should be estimated and effects on the environment assessed.

Thermal analyses may be carried out using simple linear or two-dimensional models. Finite element and finite difference software suited to both two- and three-dimensional thermal analyses are available.

(5) *Modelling Caverns in Soft Rock and Squeezing Ground.* Most caverns that will be constructed in Hong Kong will be located in rock which is strong compared with the stresses around the caverns. Exceptionally weak zones may be encountered, or a cavern constructed so deep, that the strength of the rock is comparable to the stresses. For such situations continuum or discontinuum models can be of assistance in determining stresses, deformation and support requirements. Insitu monitoring can be used to improve the predictability of the model.

(6) *Locating Unlined Pressure Shafts.* Unlined pressure shafts could be required in connection with hydropower developments, including pumped storage schemes. Continuum methods are normally used to estimate the stress field in a mountain with the objective of locating unlined pressure shafts at depths where the water pressure in the shaft is less than the minimum principal stress so as to avoid unacceptable leakage. The results from the model are used for preliminary siting, with the final location being adjusted according to field stress measurements.

(7) *Estimation of Gravitationally-induced Anisotropic Stresses.* Gravitationally induced insitu stresses can be important for caverns sited in valley sides below high mountains where anisotropic stresses can be of engineering significance. Continuum methods are useful in estimating the stresses in these conditions. The results can be verified by field measurements (see Section 3.7.5).

(8) *Analysis of Pressurised Bulkheads.* Pressurised bulkheads are common components of oil products and gas stores, hydropower schemes and civil defence and military installations. Modelling of the stress field around and within such bulkheads, normally using continuum models, is useful in design.

(9) *Analysis of Cavern Installations Subject to Vibrations.* Some civil defence and military installations and caverns housing particularly hazardous substances or sensitive processes may have to be analyzed for vibrations caused by earthquakes or military action. Caverns have been shown to perform well under heavy vibrations from earthquakes and nuclear blasts and there is therefore no general requirement for sophisticated finite element analyses or other numerical modelling that include dynamic effects (Dowding, 1977).

5. CAVERN CONSTRUCTION

5.1 INTRODUCTION

5.1.1 Purpose and Scope

This Chapter describes the methods of cavern construction and advises on good practice for the various processes involved. The advice is based on construction practice elsewhere as there is little experience of cavern construction 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 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 5.6.1](#)).

5.1.2 Methods of Excavation

Drill and blast construction dominates the construction of underground space. In the last few decades, the basic methods have not changed, but improvements in equipment and blasting technology have resulted in increased production rates and lower costs. Although full face boring can be an attractive alternative to the drill and blast method for long tunnels and shafts of uniform cross-section, the drill and blast method is expected to remain viable for tunnelling for years to come, and will remain the only 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 and
- (c) benching with vertical drillholes.

All three methods are commonly employed in the same cavern construction.

5.1.3 Working Cycle

Excavation by drilling and blasting is done in a series of cycles, each consisting of:

- (a) drilling of charge holes,
- (b) charging with explosives,
- (c) detonation,

- (d) ventilation,
- (e) scaling,
- (f) mucking-out and
- (g) initial support works

The cycle is essentially the same for tunnels, cavern top headings and benching. Probing ahead of the rock face in top headings may also be carried out (see Section 5.8.1). In some ground conditions, scaling, mucking-out and initial support may be done in an order other than that shown above (see Section 5.3.5).

5.1.4 Initial and Final Support

Initial support is the support required to make the excavation safe for the cavern construction workforce. Normally, only this support should be installed at the excavation face. Subsequently, final support that ensures long-term security and stability should be constructed some distance behind the excavation face where it does not interfere with the drill and blast cycle. As far as possible, initial support should be designed to form part of the final support. Methods and materials for initial and final support are described in Section 5.6.

5.2 PLANNING THE EXCAVATION

5.2.1 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 is governed by several factors which commonly require the top heading to be divided into two or more sections. These factors are:

- (a) the reach of the drilling jumbos and the platforms used for support works, which limits the height to 7 to 10 m depending on the plant used,
- (b) the area of unsupported roof that can be exposed at any one time, which is primarily a function of rock quality,
- (c) the presence of weak rock which might limit the area of the unsupported excavation face because of instability,
- (d) limitations on the quantity of explosives discharged in a round given by blast vibration acceptance criteria and
- (e) practical depth of blast holes, which is 4 to 5 m depending on the cross-sectional area of the top heading.

It is normally economical to excavate as large a face as possible, but the above factors may limit the size of the top heading to some 100 to 120 m².

In poor rock conditions, the initial top heading may need to be substantially smaller than 100 m² 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 top heading depends on the span of the cavern and the maximum practical size of heading. Figure 18 shows typical excavation stages for a top heading excavation in hard rock conditions for different ranges of cavern span.

The order of driving the sections depends on the rock joint orientations and the need for support. The sections should be driven in the order that avoids 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 top heading. The principal options available are either to drive one section first to completion, followed by excavation of the subsequent stages, or to allow each section to lead subsequent sections by a small distance. The selection of option is dependent on factors that centre 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, some 8 to 10 m, as this allows one jumbo to operate on two faces. Such a small separation is feasible, without impinging on vibration limits, by using non-electric (Nonel) initiation systems (see Section 5.7.4), where the risk of the first blast destroying the blasting cables of the second is avoided subject to correct planning and execution.

5.2.2 Benching

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

Production rates of bench excavation can be high. Maximum rates of 60 000 m³ of rock excavated per week have been reported from a six-cavern crude oil scheme in Mongstad, Norway.

Bench excavation may be done with vertical drillholes as in a quarrying operation, or with horizontal drillholes. Normally vertical drillholes are preferred as the drilling of holes is then independent of other sections of the excavation cycle except during blasting and ventilation. The holes are usually made with drilling jumbos or with crawler-mounted quarrying rigs. However, horizontal drilling may be required if a clean floor is necessary. 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.

The cavern excavation should be divided into benches of a suitable height. The height of the benches may be determined from consideration of the following:

- (a) *Access*: The 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 6° .
- (c) *Reach of jumbo*: Bench heights with horizontal holes are limited to the reach of the drilling jumbo, which is some 7 to 10 m; this is not a limitation for vertical holes.
- (d) *Stability of walls*: In some conditions, walls may be unstable if too high, such that successive bench excavations and support will be required.
- (e) *Cavern use*: Some uses, such as unlined fluid stores, do not require optimisation with respect to shape; only volume is important, and they are optimised primarily with respect to efficient construction.
- (f) *Cost-effective construction*: Optimisation of the excavation cycle.

Figure 19 shows typical bench excavation stages for a high cavern.

5.2.3 Access Tunnels

Access to the top heading level and to the cavern bottom level will be required for cavern excavation to proceed. Other construction accesses 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 those needed for construction.

Construction access to shallow caverns may be conveniently achieved by ramping the approach of the access tunnel to the levels of top heading and cavern bottom.

The location of the access to the cavern excavation depends on requirements for efficient excavation, permanent access requirements and geological constraints. For short caverns, construction accesses to one end of the cavern that allow excavation from single faces may be satisfactory. With long caverns, this arrangement may lead to unacceptably long construction periods. An 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 might accrue from working multiple faces.

Large cavern schemes with long and high caverns will require special access arrangements. A common method is to excavate an inclined transportation tunnel between the main caverns down to the bottom level of the caverns. Access to the different bench

levels may then be achieved through short side tunnels from the transportation tunnel. The tunnel system arrangement will vary 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 rates. **Figure 20** shows a typical 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. Some contractors may prefer a gentler gradient and it may be prudent to allow for this in the design if the penalties of so doing are modest. Paving of the access tunnels can be cost-effective because it reduces tyre wear and increases driving speeds.

5.2.4 Ventilation

Fresh air of adequate quantity has to be introduced into the tunnels and caverns to give an acceptable working environment. Vehicle exhaust fumes and blast gases must be removed. Dust should be damped by watering the tunnel spoil prior to removal and other places as required, to prevent dust being introduced into the ventilation systems in significant quantities. Ventilation may be achieved economically by providing flexible ducting with external fans blowing fresh air into the various headings and other places of work. The exhaust air should then return to the surface through the access tunnels. In caverns and tunnel systems, the ventilation system should ensure compliance with the Hong Kong air quality standard.

Detonated explosives release a large volume of toxic and asphyxiating gases. The time required to vent blast gases can be considerable, with significant implications on 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 the flexible ducting, providing rigid ducting with external suction fans and constructing tunnels and shafts to aid ventilation. In the interest of overall project economy, such tunnels and shafts should preferably also have a permanent use.

5.3 TOP HEADING EXCAVATION

5.3.1 Drilling Blast Holes

Blast holes are drilled using jumbos with hydraulic percussion drills. **Figure 21** shows a modern drilling jumbo. Rates of drilling are of the order of one metre per minute in granites and volcanic rocks and can be higher in weaker rocks. Drillholes are normally 45 to 51 mm in diameter, with the latter being the most common. The peripheral drillholes are splayed out by 5 to 8 degrees relative to the excavation axis. A maximum splay of 6 degrees is recommended for holes drilled for permanent rock faces.

Drilling pattern design is discussed in **Section 5.7.4**. **Figure 22** shows a typical drilling pattern for a tunnel or top heading.

The length of drillholes is commonly 4 to 5 m and is dictated by overall optimisation of the operation.

5.3.2 Explosives and Charging

All handling and use of explosives must be in accordance with the Dangerous Goods (General) Regulations (Government of Hong Kong, 1983). The contractor is required to apply to the Mines Division for a Category I Dangerous Goods Licence and a Permit to Use Category I Dangerous Goods under Regulations 9 and 46 respectively of the Regulations. Explosives will be delivered to the licensed site daily by the Mines Division, and the contractor will be required to pay the prescribed removal permit/delivery fees in respect of each delivery. At least one shot firer with a valid Mine Blasting Certificate issued by the Superintendent of Mines must be employed by a licensee to handle explosives on site. Safety conditions, in terms of prevention of fly-rock, allowable vibration limits (or maximum charge-weight per delay period), suppression of excessive blast noise and dust, etc. may be imposed as permit conditions.

Charging of the holes should be done by hand from a hydraulic platform on the jumbo. Charging explosives can be done safely after drilling of the face is completed. However, subject to the approval of the Commissioner of Mines, consideration may be given to charging holes whilst drilling. The holes should be charged with a high explosive bottom charge and a tube charge of a blasting agent. The ignition caps are introduced at the time of loading and the caps interconnected to form a controlled blasting system (see Section 5.7.4). Detonation must be done remotely after clearance of men and equipment. The contour holes should have tube charges only and the centre cut holes, which are of larger diameter, should have no charge (see Section 5.7.4). Nitroglycerine-based tube charges are not approved in Hong Kong.

5.3.3 Scaling

After blasting and ventilation, the roof should be scaled. This may be done either by hand or with mechanized equipment. Mechanical scalers 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. In some rock conditions, particularly where the rock is easily loosened in the scaling operation, scaling may be limited or dispensed with entirely and shotcrete applied to lock any loose material in place. A more detailed description of scaling and scaling equipment is given by Tamrock (1986). 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 final rock support is installed.

5.3.4 Mucking-out

Rubber tyred loading and transportation equipment is the normal mucking-out equipment for large tunnel and cavern top heading excavations. Front-end loaders or load-haul-dump (LHD) loaders in combination with 20- to 40-tonne off-road trucks provide large mucking-out capacities. The type and number of plant items need to be optimised with respect to cost and construction programme. In urban areas, smaller road vehicles may be required which may have consequences for the planning of the excavation and choice of plant because of the increased time needed to muck-out.

5.3.5 Rock Support

Initial rock support is installed after initial scaling and normally consists of rock bolts and/or shotcrete or fibrecrete. The support is required to secure the working space and should be installed as soon as possible. The amount and type of initial support is governed by the condition of the rock exposed by the blast. In conditions of poor stability it should be installed prior to spoil removal or after partial removal of the spoil. In very poor rock, shotcrete may be applied as initial support with scaling reduced to a minimum or not done at all. Occasionally, the contractor may elect to carry out the final support immediately (see Sections 5.6.3 and 5.6.6). Section 5.6 describes rock support methods in detail.

Final rock support should normally be installed when the top heading excavation is complete or when the contractor can carry it out without adversely affecting overall progress (see Section 5.6.2).

5.4 BENCH EXCAVATION

5.4.1 Drilling Blast Holes

Horizontal drillholes for bench excavation are made in the same way as in tunnelling, and the same type of jumbo is used. Crawler drill units or jumbos are used for benching with vertical drillholes. Drillhole lengths up to 18 m can be achieved. However, drill deviations normally limit the bench height to 15 or 16 m. Vertical holes are normally drilled beyond the desired floor level to ensure that good fragmentation is achieved throughout. Drillhole diameters are commonly in the range of 51 to 76 mm.

5.4.2 Control of Over-break

To control over-break, and where smooth cavern walls are desired, the smooth-blasting method should be used prior to benching with horizontal or vertical drilling. This is discussed in more detail in Section 5.7.4.

5.4.3 Explosives and Charging

The charging of the holes follows patterns and procedures common to quarry blasting in benches and is discussed in more detail in Section 5.7.4.

5.4.4 Scaling

Scaling of exposed side walls is required to remove loose material. This may be done by hand from hydraulic platforms or with hydraulic rock breakers. The timing of the scaling operation is less critical than in top heading excavation because wall stability is generally better than roof stability and the workforce is not exposed to the same extent. However, falls of rock from any source or height can be deadly and all walls must be made safe prior to work being done beneath them. The scaling can be aimed at bringing down large loose

blocks and the risks from smaller blocks falling covered by nets fixed to the walls. Reference may be made to Tamrock (1986) and other handbooks on the subject for further guidance.

5.4.5 Mucking-out

Spoil removal for bench excavation is similar to that described in Section 5.3.4 for top headings, except that even larger loading plant and trucks are occasionally used.

5.4.6 Rock Support

Rock support, if required, may be installed as excavation proceeds or after completion of the excavation, depending on the stability of the walls. Rock support to walls of high caverns should be installed as excavation proceeds, while the walls are within reach of the plant. Where wall stability is critical, the rate of support installation may govern the rate of excavation.

5.5 SHAFT EXCAVATION

5.5.1 Need for Shafts

Shafts may be required for a variety of purposes including ventilation, access and process requirements. However, the unit cost for excavating shafts is substantially higher than that for tunnel construction, and the rates of advance for large diameter shafts are much lower.

5.5.2 Choice of Method of Shaft Construction

Shafts may be entirely in hard rock or may exit through soft ground at the surface. Construction methods for soft deposits and residual soil are not covered by this Geoguide; only hard rock construction is covered. The choice of shaft excavation method depends primarily on the diameter and length of the shaft and accessibility to its top and bottom. The geology can influence the choice, but is usually of secondary importance. The following may be used as a guide:

(a) Bottom and top access available:

- (i) *Long-hole method* for shafts up to 50 m long of moderate to large diameter.
- (ii) *Raise boring* for long shafts of small diameter.
- (iii) *Alimak method* for long shafts of moderate diameter.
- (iv) *Pilot shaft method* for long shafts of large diameter.

(b) Bottom access only:

- (i) *Raise building* for short shafts.
- (ii) *Alimak method* for long shafts of moderate diameter.

(c) Top access only:

- (i) *Shaft sinking* for short and long shafts of intermediate to large diameter.
- (ii) *Drilling* for long shafts of small to intermediate diameter (blind shaft boring).

Table 16 gives typical ranges of diameters and lengths for the various methods of shaft construction. These methods are briefly discussed below.

The requirement for access to the lower end of a shaft, where spoil from the shaft excavation can be removed, is common to all cost-effective methods of shaft excavation. Bottom access should be provided wherever possible.

Where the use of the shaft space allows, consideration may be given to constructing several small diameter shafts instead of one large shaft to suit the most economic method of shaft construction.

5.5.3 Long-hole Drilling

Shafts shorter than 50 m, and with cross-sections of some 30 m², may be drilled with long drillholes. The limitation on the shaft height is drillhole deviation. All the holes should be drilled before blasting is carried out in several rounds from the bottom upwards. The spoil falls to the bottom of the shaft where it is loaded and transported out. Normally, scaling and rock support is required after blasting. The method is shown in Figure 23. Compared with other shaft excavation methods, this method can be the most cost-effective.

5.5.4 Shaft Excavation by Raise Boring

The raise boring method, or reaming method, is commonly used for small diameter shafts. Both vertical and inclined shafts can be formed with this method, which is illustrated in Figure 24. A pilot hole with a diameter of about 250 to 350 mm is made and the reaming head 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 0.5 m and 6 m and lengths up to 1 000 m have been formed by this method. The resulting hole is smooth and it seldom requires scaling or support.

5.5.5 Alimak Method

The Alimak method is commonly used for shaft lengths more than 40 m. With this method, the drilling and blasting are carried out upwards. The method is illustrated in Figure 25. The equipment consists of a rail-bound platform supplied with all services and tools from which all drilling, charging, scaling and support works are carried out. Shafts with diameters of between 1.8 m and 3.2 m are commonly excavated using a standard rig.

5.5.6 Raise Building

Raise building is a method of constructing shafts from the bottom upwards but without the mechanisation of the Alimak method. It involves drilling from a platform set close to the shaft roof. The platform has to be removed before each blast. The method is slow and hazardous and is not much used.

5.5.7 Pilot Shaft Method

For large vertical shafts, the normal procedure is to establish a pilot shaft of approximately 5 m². The pilot shaft can be advanced by the Alimak method or by raise boring. After the pilot shaft is established, the final shaft is excavated from above by the drill and blast method, and the spoil is mucked down the pilot hole. Scaling and rock support should be carried out successively from above. Shafts up to 80 m² have been sunk with this method. Figure 26 shows the method.

5.5.8 Drilling

Shafts can be drilled from the surface using large drill rigs. Diameters up to 6 m are possible. The method is slow and expensive. Suitable equipment is not widely available.

5.6 ROCK SUPPORT METHODS

5.6.1 Design Philosophy

Two fundamental principles are commonly used as the basis for designing economic rock support for hard rock excavations:

- (a) to achieve a self supporting rock mass with minimum support and
- (b) 'design as you go'.

Section 4.1.4 indicates that initial rock support assessments should be made on the basis of a rock classification system such as the Q-system or RMR method. With the 'design as you go' concept, rock support is only installed where observation of the rock surface shows that it is necessary, often with the aid of the Q-system or RMR method. Thus the amount of initial support and the support method are decided when the rock conditions are evaluated at

the site of installation and as the excavation proceeds. Depending on site conditions, final support can be determined either at the same time as initial support or later. This places considerable responsibility on the engineering geologists and tunnelling engineers, who must take the decisions on support at the work place. A full time resident engineer and supporting staff will be required. The importance of employing adequately qualified and experienced staff full-time on site for this task cannot be over-emphasized, nor can the importance of effective communication between the people on site.

5.6.2 Short-term and Long-term Safety Requirements

Rock support has to serve two purposes:

- (a) *Initial support* is required to secure, as soon as possible, safe working conditions for the construction crews. Normally, the contractor is responsible for determining this support, based on the various supporting methods described in the contract. For cost-effectiveness, initial support should, if possible, form part of the final support.
- (b) *Final support* is required to maintain stable rock conditions in the excavation during the economic life of the cavern. Final support may incorporate some or all of the initial support. Normally, the owner is responsible for the decision on the type and amount of support to achieve these safe conditions. As indicated above, the rock support methods and amount should be determined when the rock conditions can be studied and mapped on the actual site of installation during or after the excavation.

5.6.3 Selection of Support Methods

The suitability of the various types of rock support and design methods are described in Section 4.5. Although rock classification systems such as the Q-system and RMR method indicate the suitability and extent of various types of support, the final selection of support method should be based on observation of the stability of the rock. Spot bolting is usually specified on the basis of observation and experience. Systematic bolting and shotcreting is often specified on the same basis, with the Q-system or RMR method used to check the adequacy of the proposals. Judgement is sometimes augmented by numerical analysis or modelling.

The most common method of roof support for large caverns in fair to good rock is systematic bolting combined with shotcreting as final support. The amount of initial support depends on the roof stability and the contractor's method of construction and varies from virtually none to systematic bolting and shotcreting. In addition to rock conditions, the extent of final roof support should be related to the future use, occupancy and psychological factors. Walls in large caverns are supported less heavily than roofs, and spot bolting, systematic bolting and/or shotcreting should be applied in accordance with stability requirements.

The selection of support can best be done by using a classification system during excavation of the cavern. This classification should be done as soon as possible after blasting

and spoil removal. In that way the initial and the final support can be combined cost-effectively. If shotcrete is used as initial support, the classification must be done before it is applied. A bolt type approved for final support should, if possible, be used for initial support.

5.6.4 Rock Bolts

(1) *Introduction.* Rock bolting is the most common method for rock support and it is convenient and flexible to use. Rock bolts may be used both for initial support at the working face and for the final support. The rate of rock bolt installation can be 50 to 100 bolts per 7.5 hour shift, although lower rates than this are more common in Hong Kong. Bolt lengths are commonly in the range of 2 to 5 m.

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 15 and 16).

Bolts may also be employed to fix a variety of items to the 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 may be anchored at the far end and tensioned or they may be fully grouted bolts without tensioning. Some proprietary bolt systems attain friction against the 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 prestress the rock.

There are numerous types of bolts available, both for initial and long-term support, and some bolt types are suited for both uses. Schach et al (1979) give a useful description of the most prominent types, the popularity and details of which have not changed much in the last ten years.

(2) *Bolt Elements.* Most manufactured rock bolts consist of a steel rod of 16 or 18 mm diameter although larger diameters are used occasionally. High yield strength steels, including stainless steel, and rolled threads are used to achieve high working loads. Carbon steel bolts may be plain, hot-dip galvanised or epoxy coated (and sometimes both), depending on the corrosion protection required.

Apart from manufactured bolts, deformed high yield reinforcing bars are in common use, particularly for final support. The most common diameter is 20 mm but larger diameters are also employed. Reinforcing bars may receive a cut thread. Exceptionally, stainless steel deformed reinforcing bars may be required.

The yield strengths of 16 mm and 18 mm plain steel round bars are typically 9 tonnes and 12 tonnes respectively. Stainless steel bars of the same diameters commonly have higher yield strengths. Re-bar bolts are commonly of 20 mm diameter with a yield strength of some 12 to 15 tonnes, but larger diameters are also used.

Surface bearing plates may be used in conjunction with bolts in a variety of circumstances. They may be round, square or triangular and may be flat or saucer-shaped. The choice of shape and size depends on the prevailing rock conditions. For most hard rock applications with moderate stresses, round or square plates of 125 mm or 150 mm size are suitable. Larger plates, often of triangular shape, may be employed in spalling rock conditions. Such triangular plates can be as large as 350 mm x 350 mm. Bearing plates should be galvanized to give some corrosion protection.

Bearing plates for tensioned bolts should be designed to show visible signs of distress at loads below the yield load of the bolt. For most purposes a round saucer-shaped plate has proved to be the most satisfactory. When correctly designed the plate should be flattened against the rock at a load of some 10 tonnes. Loads in excess of this would cause the edges of the plate to bite into the rock, causing it to spall, or the edges would lift as the centre of the plate is pulled into the rock.

The bolt should be connected to the plate with a nut and a hemispherical seating (Figure 27). Anchor plates are available where the hemispherical seating can be dispensed with either by providing a hemispherical nut and a corresponding seating on the plate, or by shaping the plate such that the nut will seat against it even if the bolt is inclined.

In highly fractured rock steel straps and wire mesh may be used to retain rock between bolts. The steel straps are commonly 100 mm wide and 5 mm thick with slotted holes to fit the bolt steel, or parallel steel rods connected with cross-ties in ladder fashion. The wire mesh can be 50 mm mesh woven fencing. The removal of wire mesh can be hazardous and this should be taken into account in the support design.

(3) *Bolts for Initial (Immediate) Support.* Bolts for initial support, where immediate support is needed, are normally anchored at the far end and are tensioned against a bearing plate with a nut (Figure 27). The anchorages may be of an expansion 'shell' or 'bail' type. With both types a cone at the far end of the bolt expands two or more wedges against the side of the hole when tension is applied to the bolt. The bail type is commonly preferred as the bail reduces the slippage between the wedges and the drillhole wall, making it easier to tension the bolt. The hole diameter should be suited to the size of the anchorage. Other proprietary anchorages exist, including plastic expanding anchorages. The efficacy of all anchorages should be proven before general use in a construction contract.

Polyester-grouted anchorages may be used. The polyester is introduced as a cartridge containing the polyester and hardener which is broken and mixed by rotation of the rock bolt as it is introduced into the hole (Figure 28). The setting time for the polyester is commonly in the range of 2 to 10 minutes. The hole diameter should be 8 to 12 mm larger than the bolt steel diameter for optimum result.

Split-and-wedge type anchors, where the end of the bolt is split and forced against the sides of the hole by a wedge, have been used but are not recommended due to uncertain and variable performance.

Tensioned bolts with mechanical anchorage should be checked and the tensioning nut re-tightened periodically during construction if slippage of the anchorage is suspected.

(4) *Bolts for Final (Permanent) Support.* Bolts for final support are mostly installed some time after the completion of the initial support works. The bolts may be installed to improve general stability as spot bolting or systematic bolting as described in Section 4.5.4, or may be installed to provide support according to an analytical design based on clearly defined failure mechanisms.

The most common type of bolt is the fully grouted re-bar bolt (Figure 28). The various proprietary names for bolts refer mainly to the method of grout placement. Bolt diameters should be 20 mm or greater. For bolts installed to improve general stability as described in Section 4.5.4, bolts of less than 20 mm diameter may have adequate tensile strength, but would have insufficient stiffness to withstand the rigours of installation. The number of bolts installed is in excess of that required for stability (at time of installation the factor of safety against collapse is greater than one).

Bolts are suited for final support if they have adequate corrosion protection. For re-bar bolts this protection is provided by a combination of sacrificial metal and by cement grout. Bolts installed to improve general stability, as described in Section 4.5.4, satisfy these requirements because they are grouted and oversized by virtue of their over-capacity. Corrosion is thus not a controlling factor in their design, except in particularly corrosive environments such as those encountered in submarine construction.

Rock bolt support that is subject to analytical design based on clearly defined failure mechanisms may be carried out using the methods described by Hoek & Brown (1980). Such bolts should comply with the Geotechnical Manual for Slopes (GCO, 1984) with regard to stress level and corrosion protection.

In the most common method of installation, grout is injected through a tube into the drillhole starting at its far end. Sufficient grout should be injected to ensure that the hole will be full when the re-bar is inserted. The grout mix, which should be pre-mixed and require only the addition of a fixed quantity of water, must have a consistency such that the grout does not seep out of a vertical hole and does not prevent the easy insertion of the re-bar. Vertical bars should be wedged in place after insertion to prevent movement prior to the grout setting. Any seepage of grout from the annulus around the bolt may be staunched with rock wool or other fibrous material.

The grout should be a mortar containing well-graded sand with a maximum size not exceeding 2 mm. The grout may contain expansion agents and accelerators. Sand-cement ratios of 1:1 and 3:2 are suitable. Bentonite may be used at a rate of some 2% of the cement weight to improve workability and help grout expansion. The hole diameter should be at least 12 mm larger than the diameter of the bolt.

The bond between bolt and grout and grout and rock can be damaged by blast vibrations. For final support, the minimum distance between tunnel blasting and bolt installation should be specified such that damage does not occur, although this may be relaxed in unmanned caverns not containing plant. A common minimum distance is 40 m, but other criteria may be used. Much shorter distances may be used between bench blasting and grouted bolts such that bench blasting under a bolted roof is possible. The difference in criteria is caused mainly by the differences in charge density which, in the cut of a tunnel

blast, can be 25 kg/m³ or more compared to perhaps 0.5 kg/m³ or less for a typical bench blast.

Both tensioned and untensioned grouted bolts may be galvanized for added corrosion protection, but the compatibility of the galvanization and the 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). In a corrosive environment, such as with the presence of sea water, high quality stainless steel bolts may be required.

Rock bolts are an essential element for the safety of occupied caverns. They must be properly specified and installed by trained operators under close supervision. Quality assurance and control procedures should be established for the installation of final rock bolt support.

(5) *Bolts for Both Initial and Final Support.* Bolts that can be used for both initial and final support should be mechanically fixed or bonded to the rock at their far ends, as are bolts for initial support, and in addition have full corrosion protection. Economic advantages can be achieved with such bolts. Several types are available.

High quality stainless steel expansion shell bolts may be acceptable as final support without the need for further protection against corrosion. They tend to be expensive but may be warranted in the corrosive environment of submarine tunnels and caverns.

Plain steel bolts may be hot-dip galvanized. In some groundwater environments they can be prone to corrosion if the zinc layer is damaged. This can be overcome by coating the galvanized bar with epoxy. Galvanized bolts should be grouted for added corrosion protection.

Hollow tube expansion shell bolts may be used. The bolt is tensioned, as for any other expansion shell bolts, for immediate support, and is subsequently injected with cement grout for corrosion protection. The steel may be hot-dip galvanized to provide corrosion protection prior to grouting.

Grouted re-bar bolts may be used for immediate support if the end of the bolt is polyester grouted for immediate anchorage, or an accelerator is added to the cement grout. The polyester cartridge should be introduced into the end of the hole prior to grout placement and bolt insertion.

Polyester grout may be used for fully grouted bolts. It is expensive, but is nevertheless specified from time to time, particularly for smaller works.

The choice of bolt and bolt protection will depend on price, availability, rock conditions and the contractor's experience. All the above bolts can be satisfactory for both initial and final support.

(6) *Bolts for Shutters and Fixtures.* Bolts for permanent and temporary fixings should be suited to the loads to be carried and should have a suitable design life. Shuttering ties are commonly needed. The ties may be equipped with bail-type expansion shells that operate

when the tie is tensioned. Removal of the tie is achieved by unscrewing it from the anchorage shell. There are various proprietary systems available.

Reinforced concrete may be tied to the rock with rock bolts, with ends protruding to give an adequate bond length with the concrete. The bars may be bent to suit a particular design. Rock bolts for this application should be designed in accordance with the provisions of the Geotechnical Manual for Slopes with particular reference to stress level and corrosion protection.

(7) *Pre-tensioning of Bolts.* Tensioned bolts should not be tensioned to more than 50% of their ultimate tensile strength for final support. The bolts may be tensioned manually with spanners and tensions of three to four tonnes can be achieved. Manual and compressed air operated torque wrenches can give more consistent tensioning, but the correlation between torque and tension is not good due to variable friction between the nut, threads and seating. Bolts may be tensioned more reliably with a hydraulic tensioner. The tensioner applies a direct pull to the bolt without torque. The nut is tightened by hand using an integral pipe wrench. The tension of previously installed bolts can be checked by re-tensioning the bolt until the nut loosens. Hydraulic tensioners should be used for all major rock bolting.

5.6.5 Long Bolts Ahead of the Face: Spiling

Spiling is a means of reinforcing the rock mass ahead of the excavation face with long bolts (spiles). This support system may be applied to particularly unstable rock to restrict deformation of the surrounding rock and to prevent severe rock falls following blasting.

The spiling holes should be drilled from the perimeter of the excavation face and slanted outwards into the rock surrounding the tunnel 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 the minimum pile length being the length of 1.5 rounds. The spacing of the spiles should be based on a detailed evaluation of the current ground conditions.

5.6.6 Shotcrete

(1) *Introduction.* Shotcrete is concrete placed by spraying. 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. Shotcrete may be used plain or reinforced with mesh, and may be used in conjunction with rock bolts.

Shotcrete has been in use since the 1920's for concrete repair work and for rock support. In the early 1950's, reinforced shotcrete was used for the first time as a substitute for traditional cast insitu tunnel linings. Development of spraying robots in the late 1950's allowed shotcrete to be used for early support of poor rock without prior scaling, as the operator could stand remote from the face under previously supported rock. In recent years

it has become common to add steel fibres to the shotcrete, then known as fibrecrete, which can be used in lieu of mesh reinforced shotcrete.

The combined use of fibrecrete and rock bolts has become a common method of support. The fibrecrete has considerable flexural strength and has the ability to span between bolts, giving continuous rock support. The combination has a low overall cost, is flexible in its application and is quick to place.

Compared to cast-insitu concrete lining, shotcrete and fibrecrete have the advantages of:

- (a) short mobilisation time for equipment,
- (b) no requirement for formwork,
- (c) independence of the shape of the excavation,
- (d) high rates of application,
- (e) ease of combination with other support methods and
- (f) no need to fill over-break volume.

The principal disadvantage is 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. The above advantages have led to shotcrete replacing cast-insitu lining in many cases.

Fibrecrete has properties that are as good as or better than mesh reinforced shotcrete. The time and cost of placing reinforcing mesh in tunnels and caverns can be considerable and are higher than the time and cost of providing fibre. Fibrecrete should therefore be used in preference to mesh reinforced shotcrete in most situations.

The use of shotcrete in underground works requires an understanding of both rock mechanics and concrete technology.

(2) *Shotcrete and Fibrecrete as Rock Support.* Shotcrete may be used for final and initial support, with or without rock bolts, where rock bolts alone give insufficient support or where additional safety against the fall of rock fragments is required. Initial assessments of shotcrete and fibrecrete thicknesses and their suitability for final support can be obtained from the Q-system and RMR method.

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.

Care should be exercised in using shotcrete, placed as initial support at the excavation face, as part of the final support. The shotcrete may be damaged by blast vibrations and may conceal stability problems which will only become apparent later. The shotcrete cannot readily be removed and it will become incorporated in the final support. This implies that

the total support strategy must be determined before the section is sprayed. Normally de-bonded shotcrete is removed, systematic bolting placed and a further layer of shotcrete applied.

Shotcrete does not adhere to clayey material. Where clayey seams are wider than the thickness of the shotcrete layer, precautionary measures should be taken prior to shotcrete application. 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 29). Fibrecrete can span clayey zones of up to 500 mm width provided the clay does not swell. Shotcrete and fibrecrete may not be suitable for stabilising wide zones of swelling clay. Support of such zones must be subject to special designs and insitu concrete should also be considered.

Water pressure may build up behind shotcrete and fibrecrete and may cause cracking of the material and destroy the bond to rock. Drillholes have proven to be ineffective as drains in many situations and surface drainage is normally preferred. Figure 30 shows a typical detail of surface drainage. The pipe shown in this figure may be omitted if the seepage is minor. This type of surface drainage works well if correctly installed, but it requires good workmanship.

(3) *Methods of Application.* Shotcrete is placed by pumping concrete to a nozzle from which it is ejected at high velocity. The composition and properties of the wet concrete are a compromise between requirements for pumpability, ease of application, loss reduction and properties of the final product.

The two principal processes for shotcreting are the dry-mix process and the wet-mix process. In the dry-mix process, the cement and sand are pre-mixed and contain only natural humidity. The mix is disintegrated and transported through hoses by a compressed air stream to the nozzle where water is added. In the wet-mix process all the constituents of the concrete, including water, are mixed and pumped to the nozzle. Compressed air is injected at the nozzle to disintegrate the mix and to provide the required spray velocity.

The wet-mix process has become the more common method in major works because of several advantages over the dry-mix process. The following points summarise the relative benefits of the two processes:

- (a) *Consistency:* In the dry-mix process, the water is added at the 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 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 250 mm thick without risk of slumping or separation from the rock. A normal layer thickness is usually in the range 50 to 100 mm.
- (c) *Rate of application:* Dry-mix is commonly placed at 0.5 to 1 m³/hour, with a maximum of 3 m³/hour for expert contractors, against a rate of typically 3 to 4 m³/hour and up to 5 m³/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 are 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*: The 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.

The lower strength achieved with the wet-mix compared to the dry-mix is no disadvantage, as the strength is more than adequate at 20 to 40 MPa (see below). However, contractors may occasionally offer dry-mix shotcrete at competitive prices, particularly for smaller jobs, and there is no over-riding reason not to employ it.

(4) *Properties*. The strength achieved is similar to that of poured concrete. The dry-mix process can give compressive strengths of 40 to 50 MPa, whilst the wet-mix process typically yields values of 20 to 30 MPa, which can be increased to 40 MPa with the addition of plasticisers, and to 60 or 75 MPa with specially designed mixes. Of considerable importance in rock support work is the adhesion to rock achieved. The adhesion is typically 0.5 to 1.5 MPa but poor workmanship can reduce this to zero.

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

Shotcrete shrinks by 0.06 % to 0.10 %, depending on the water/cement ratio. In tunnel 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 quality of shotcrete and fibrecrete should be checked using standard test procedures for spraying trial panels, coring and laboratory testing (Concrete Society, 1980; Government of Hong Kong, 1990).

(5) *Sand and aggregate*. The grading requirements for sand and aggregate for shotcrete are the same as for any other quality concrete. Rounded gravel should be preferred when obtainable, as it improves pumpability, but other material can also be used successfully. For many applications, the maximum aggregate size is set at 8 mm, but this should be increased up to the capacity of the shotcreting plant if large single layer thicknesses (150 mm or more) are to be placed.

(6) *Additives*. Additives may be used in wet-mix shotcrete to enhance various properties. Plasticisers and super-plasticisers should 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, added diluted in water at the nozzle, should be used in most mixes to achieve improved adhesion to wet surfaces, increase layer thicknesses and to allow a shorter time interval between the application of successive layers. Silica dust may be added by up to 8 % by weight of cement to reduce losses and fall of concrete and to increase strength and improve resistance to sulphate attack.

(7) *Fibrecrete*. Fixing reinforcing mesh to the rock surfaces is time-consuming and costly. The simple and effective solution offered by fibrecrete has substantially reduced the use of mesh reinforced shotcrete where enhanced tensile strength is required. Tests have shown that the addition of 50 to 75 kg of steel fibres per cubic metre of shotcrete allows mesh reinforcement to be omitted (Opsahl et al, 1982; Skåtun, 1986). Polypropylene fibre may be added at a rate of 1 kg/m³ to improve bond to rock, to give slightly less rebound loss and to improve the distribution of shrinkage cracks. Polypropylene is not a substitute for steel reinforcement. Expertise and care are required to prevent balling of fibres in the mixing process.

With the use of fibres, a high density, high strength concrete is required to reduce permeability so as to prevent corrosion of the fibres. This is particularly important where aggressive groundwater is encountered. Experience shows that fibrecrete performs satisfactorily also in salt water environments (submarine tunnels), with corrosion of the fibre being limited to the exposed ends only.

(8) *Workmanship and Quality Control*. The quality of shotcrete is dependent not only on design, but also on the site process and the shotcreting crew. The type and quality of the shotcreting equipment are important and should be matched with the shotcrete mix. The crew should be familiar with the equipment and the whole process of shotcrete application.

The shotcreting process creates a potentially poor working environment. If mitigating measures are not introduced, not only will the health of the crew suffer, but the quality of the shotcrete will be poor. The crew should be equipped with protective clothing and full-face shields and dust masks. Lighting should be good, easily movable and placed such that reflections from fog are at a minimum. There must be means of keeping face shields and lights clean. The fog created in the wet-mix process can be chemically aggressive due to the shotcrete additives, and ventilation should therefore be good. When shotcrete is used for immediate support of poor rock, the risks of using hand-held equipment should be realised and consideration be given to employing a shotcreting robot.

The shotcrete should be applied in a stream normal to the rock face. If mesh reinforcement is used, the angle of the stream should be varied to avoid voids forming in the shadow of the reinforcement. This requires a working platform of good mobility, usually a hydraulic platform.

Where rock bond is important and the rock is obviously dirty, consideration should be given to cleaning the rock surface with compressed air and water spray. Ideally, the rock surface should be allowed to dry before shotcreting. Oil films resulting from diesel exhaust and rock drilling can be difficult to see and difficult to remove. Oil films can be seen best in ultraviolet light. Removal of the film may be achieved with a de-greasing agent and a water and compressed air spray.

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.

The rebound loss material accumulates and sets hard. Where such permanent accumulations are undesirable, plastic or other cover should be provided to ease subsequent removal.

The air supply used in the process must be free of oil.

Quality assurance and control procedures should be established and implemented for all shotcrete and fibrecrrete work. Experienced crews and specialist contractors with plant suited to the work should be employed if good quality is to be achieved.

(9) *Equipment.* For tunnels and caverns having a cross-sectional area of more than 25 m², an integrated system should be used for applying shotcrete. Concrete pump, accelerator pump, accelerator tank with heating element, electro-hydraulic power pack, hose and cable drums, flood lights and other equipment should be mounted on a heavy carrier. The carrier may be a wheel-loader or truck chassis. The rig should be made ready for operation within 10 minutes of arrival at the area to be treated.

5.6.7 Rock Anchors

The use of rock anchors may be required in special cases, such as for permanent stabilisation of high walls of caverns. 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.

The anchors may have grouted fixed lengths or may be fixed at their far ends within anchorage tunnels driven for that purpose.

Typical lengths of anchors vary between 10 m and 30 m. They should have adequate corrosion protection in accordance with Geospec 1 (GCO, 1989). Long-term monitoring of permanent anchors will be required in accordance with Geospec 1.

5.6.8 Concrete Lining

Concrete lining, shown in **Figure 31**, may be applied as final support where large areas of poor rock occur, since the arch structure can take large loads. It is, however, the most costly and time-consuming support method.

Where a concrete lining immediately behind the working face is required, the concreting normally follows the excavation cycle. Plain concrete is commonly used in tunnels, but reinforcement may be used under special circumstances and in large caverns. Casting concrete is often preceded by installation of initial support which may be rock bolts and shotcrete.

Concrete linings may be required to treat areas of swelling clay where shotcrete or fibrecrete is inadequate to withstand the swelling pressures.

A simple steel shield is normally used as formwork and the ends of the shield have to be adapted on site to suit the actual cross-section. Shortly after the form is filled with concrete, the next round can be drilled and blasted. Blast vibrations do not significantly affect the properties of the concrete (see Section 5.7.2).

5.7 BLAST VIBRATIONS AND BLAST DESIGN

5.7.1 Introduction

All blasting causes vibrations which are transmitted to the environs. If sufficiently strong, these vibrations may cause damage to structures and equipment and may cause discomfort and disquiet to individuals. The 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 the excavation can be damaged if design or execution is unsuitable.

Blast design is aimed at breaking up and loosening rock as fast and economically as possible within the constraints set by limits on blast vibrations. 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. Knowledge is required of the energy propagation properties of the rock and soil between the blasting site and the receivers such that the attenuation of vibrations with distance can be estimated. Lastly, the vibrations in the immediate vicinity of a blast must be estimated from the configuration and size of the blast.

As a general guide, 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.

Vibrations are measured using vibrographs which record the particle velocities or particle acceleration and dominant frequencies. The sensors are mounted in three orthogonal directions and register separately. The sensors are rigidly mounted on building basements or other points on structures to measure the blast vibrations.

5.7.2 Blast Vibration Acceptance Criteria

Blast vibration acceptance criteria depend on the type of structure, technical installations and occupancy as well as the dominant frequency of the vibration. Blast vibrations normally have a frequency of 20 to 200 Hz which exceeds the natural frequency of most buildings. Only massive concrete structures may have a comparable natural frequency. 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. Therefore, in general, amplification of vibrations due to resonance will not occur.

Blast vibration limits depend on the frequency range of the dominant vibrations. Depending on the frequency range and type of structure or installation, particle acceleration, particle velocity or particle displacement may be limiting. Peak particle velocity and maximum displacement are normally limiting for structures. Specification of these two values covers the full frequency range. Some rotating machinery and computer installations may be subject to limits on allowable peak particle acceleration which is also a factor important in assessing the dynamic stability of slopes. Figure 32 shows typical safe vibration limits on a tripartite logarithmic graph.

Upper limits on blast vibrations can be set using standard values which have been shown through experience to be acceptable. The safe vibration limits used in various countries vary but all follow the same pattern. A peak particle velocity of 50 mm/s or 70 mm/s and a peak displacement of 0.07 mm are often the basic values (Table 17). In Hong Kong a value for peak velocity of 25 mm/s has been used for many years. The limits are maximum values normally acceptable for a building. Blast vibration is normally measured on a basement wall and the vibration criteria are related to this location. Where resonance may occur, the vibration levels must be monitored at various levels in the structure or the acceptance criteria must be lowered to take into account amplification through resonance. In the latter case the limiting particle velocity may be halved for some structures founded on soft ground. Limiting accelerations for computer installations depend on many factors, but are commonly 0.25 g (0.25 x gravitational acceleration) for momentary ground motions with a frequency exceeding 7 Hz. Limits on vibrations for slopes and computer installations and other sensitive equipment must be established separately for each case.

Blast vibration limits for cavern schemes in Hong Kong should be set following a survey of existing structures, equipment, services and building use. Note that the extra costs of adapting blast design to strict limits can be high. Figure 33 shows an indicative cost curve as a function of distance and vibration constraints. The effect shown may be reduced somewhat by using non-electric (NoneI) ignition systems.

Blast vibrations limited by engineering criteria 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.

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 (1988) 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, at least 150 mm/s. These results support Scandinavian practice, but some investigators have recommended lower limits (Olofsson, 1988).

5.7.3 Energy Propagation

The vibrations that will result from a blast may be calculated using a formula of the form:

$$A = K Q^d R^{-b}$$

where	A	= predicted particle velocity in mm/s or predicted maximum amplitude in micron (0.001 mm)
	K	= a 'rock constant'
	Q	= maximum charge weight per delay interval in kilograms
	R	= distance in metres between the blast and the measuring point
	d	= charge exponent
	b	= attenuation exponent.

Various exponents are in use. In Scandinavia, the values $d = 0.5$ and $b = 0.75$ are in common use for distances up to 100 m (Dyno, 1980; Tamrock, 1989). These formulae give reasonable agreement with observation for distances up to 100 m. The principal uncertainty relates to the rock constant, K. K-values measured at the same locations for a series of blasts show a considerable spread. Large numbers of readings for a range of distances and charge weights are necessary to yield a statistically significant result. Such tests can be done, but normally no improvement on predictability is achieved beyond that given by standard values. The value of K is largely related to the rock type, structure and the confinement of the blast. Thus the values of K to be used for tunnel blasting, including cavern top headings, will be larger than for bench blasting. Values of K for the granitic and volcanic rocks in Hong Kong are in the range of 1 000 to 1 200 for tunnel blasting (Dyno, 1980) and may be 400 for bench blasting (Tamrock, 1989). For surface blasting of granitic rocks in Hong Kong, Clover (1986) and Smith & Morton (1986) reported values of d, b and k to be mainly within the range of 0.6 to 0.8, 1.2 to 1.6 and 200 to 1 000 respectively. The values of K in soft rock, including decomposed rock, may be taken as half the above values and in soil can be one quarter of the hard rock values. The Blasters' Handbook (ETI, 1980), which follows North American practice, gives K values for bench blasting as 600 and up to 5 times this value, i.e. 3 000, for heavily confined blasts (both values quoted have been recalculated for metric units) but a slightly higher attenuation factor, b, of 1.6 and a higher charge exponent, d, of 0.8 are used.

When 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 and thus optimise the charge weights.

5.7.4 Blast Design

(1) *Objectives.* The objective of blast design is to break the rock, 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 criteria. A pattern of drillholes and a charging pattern has to be determined. Langefors & Kihlstrom (1979) gives procedures for blast design and reference may also be made to Persson et al (1980).

(2) *Drilling patterns.* Drillholes for tunnel blasts can be classified into three groups according to their purpose:

- (a) *cut holes*, which are blasted first, are heavily charged and create the free surface for the rest of the blast,

- (b) *blast holes*, which break the bulk of the rock, and
- (c) *contour holes* on the edge of the blast, which define the excavation perimeter and which limit the overbreak and damage to the surrounding rock.

A typical drilling plan is shown in Figure 22. The most usual cut-type today 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.

The length of the holes is normally in the range 4 to 5 m long and the pull, the length of tunnel blasted, is typically 0.5 m less than this. Exceptionally, the pull may be increased to 5 m if rapid advance is desired.

(3) *Control of Over-break.* The method most commonly used to control damage and over-break in final walls and roofs in underground excavations is smooth blasting. The method is characterised by correct drillhole spacings 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 to reduce the drillhole deviation are important.

Tube-charges are normally used with the smooth blasting technique. Tube-charges are prefabricated explosive elements, normally with diameter between 17 mm and 22 mm. Tube-charges have the benefit of giving correct charge distribution within each charge hole. With the smooth blasting technique, the contour holes are drilled, charged and blasted in the same excavation cycle as the cut and the main blast holes. However, the contour hole charges are detonated after the main blast. The charge in the contour holes, as well as the blast holes nearest to them, should be carefully controlled so as to prevent damage to the rock beyond the contour holes. With the pre-splitting technique the charges in the contour holes are detonated before the main blast holes. Current practice is to move away from pre-splitting because of higher cost and vibration levels, and because of masking of joints of unstable blocks.

(4) *Maximum Charge Weight Per Delay Interval.* In urban areas where rock blasting may cause damage to surrounding structures due to excessive vibrations, a reduction of the unit charge (charge weight per delay interval) has to be made. In order to achieve this, it is necessary to either reduce the round length or divide the cross-section into two or more sections. Alternatively, consideration may be given to increasing the number of delay intervals by mixing different types of electric detonators or by using non-electric (Nonel) initiation systems.

The formula in Section 5.7.3 gives the maximum charge weight per delay. The value given relates to the charge weight detonated instantaneously. Detonators of the same nominal delay have delays that are dispersed around the nominal value following an approximately normal distribution. The charge weight given by the formula may be increased to take into account this spread in detonation times. In tunnel blasting the actual maximum charge weight per delay may be increased by 50% of the calculated value. Tamrock (1989) gives details on how to estimate permissible charge weight per delay for bench blasts.

The drill pattern should be designed to give as good a charge distribution as possible. For hole diameters greater than 38 mm, which is usual for excavation faces larger than 12 m², the specific charge (explosive weight per cubic metre of rock) can be between some 0.9 and 2.4 kg/m³ of rock. The specific charge for the cut considered alone can be substantially higher than this. For smaller cross-sections the specific charge may be even larger. Figure 34 shows an example of the change in specific charge with varying tunnel cross-sectional area. Cavern top headings are excavated in a similar way to large tunnels. For cavern benching, both specific drilling (length of blast hole per cubic metre of rock) and specific charging are 40% to 60% of that required for the cavern top heading.

(5) *Explosives and Initiation Systems.* Cartridged explosives should be used for all underground blasting works where there are constraints on blast vibrations or where groundwater is present. Bagged explosives or explosives delivered in bulk, such as bulk ANFO (ammonium nitrate/fuel oil), may be used where there are no such constraints, particularly for benching. The holes should be bottom primed using cartridged explosives or cast boosters.

The blast may be initiated using electrical or non-electrical (Nonel) initiation systems. Nonel systems have the advantage of not being sensitive to stray ground current and they permit greater control of the blast because a larger number of delays are possible.

5.8 EXPLORATORY DRILLING AND GROUTING

5.8.1 Need for Exploratory Drilling and Grouting

Exploratory drilling may be carried out to check the rock ahead of the top heading or tunnel face. Such drilling should be carried out in weakness zones and water bearing zones to determine if grouting may be required for stabilisation or seepage reduction.

Where there are strict criteria with respect to water inflow, such as in fuel stores and excavations in areas where the allowable draw-down of the water table is limited, exploratory drilling should be carried out systematically and be followed by grouting where necessary. Systematic exploratory drilling is required for all submarine tunnels and caverns. This is also necessary for unlined caverns used for storing high pressure liquids or gases to limit the rate of water inflow and attendant pumping costs. Probe holes should extend forward and slightly above the top heading.

The criteria for grouting should be determined from the geological conditions, the cavern use and any restrictions on groundwater lowering.

5.8.2 Drilling Exploratory Holes and Grout Holes

Exploratory and grout holes are normally drilled using a drilling jumbo. Hole lengths of 20 m can be achieved easily and longer holes may be made. Very long holes may require a different type of rig such as a long-hole machine. Only exceptionally should diamond coring be considered due to the time required to make the holes. The number of holes, their lengths and directions should be determined on site according to the situation encountered.

Exploratory holes should extend beyond the line of the next stage of excavation and should be sufficient in number to explore the rock adequately. The direction and number of grout holes should be such that the hole spacing will ensure good treatment of the rock around the excavation at the remote end. 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.

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.

The decision to grout can be taken according to pre-set criteria on the basis of the water made from probe holes. This approach gives better results than if the extent of grouting were specified on the basis of data from investigation holes drilled from the ground surface. Water testing may be required to obtain better quality data, but the advantage of such testing should be weighed against the additional time required.

5.8.3 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 joints in the rock mass. Neat cement grouts are used to treat rock with Lugeon values down to 0.5, but especially fine cements may be required for the lower range. Bentonite and other agents are used in cement grouts to improve grout properties. Laboratory tests should be done to check the grain size and suitability of the cement.

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 watertightness is needed, and is commonly applied after first stage sealing with cement grouts.

Grouting and sealing works should preferably be performed ahead of the excavation face ('pre-grouting'). Grouting and sealing carried out after the opening is established ('post-grouting') is more costly and time-consuming and can be less effective.

Injection of many holes simultaneously is necessary to achieve adequate production rates. The grouting equipment should at least allow pressure readings for each individual hole and should preferably allow the grout take of each hole to be recorded. The grouting limits should be set by combined grouting pressure and grout take criteria.

The quality of the grouting should be checked by drilling new exploratory holes prior to resuming the excavation cycle.

5.9 DISPOSAL OF SPOIL

5.9.1 Rates of Spoil Production

Cavern excavation implies high excavation rates. Rates of 60 000 m³ of rock per week can be achieved in large multi-cavern systems as described in Section 5.11.1.

5.9.2 Options for Spoil Disposal

The options for spoil disposal vary from site to site and include:

- (a) tipping in adjacent reclamation,
- (b) crushing for concrete aggregate, drainage and road stone and
- (c) barge or road transport to remote reclamation or other disposal site.

Road transport in urban areas may be severely restricted and may not be feasible for a large cavern with high spoil production rates.

5.10 WATER PROOFING AND OTHER MITIGATION MEASURES

Grouting is the primary means of water control. The method may not exclude water sufficiently for all cavern uses and other methods of water control may have to be introduced. The principal methods to be considered are:

- (a) drainage at the rock face behind shotcrete,
- (b) concrete lining,
- (c) drip screens and
- (d) impermeable membranes.

The choice of method depends on the amount of seepage and requirements given by the cavern use and by the cost of the various options. Water and fuel stores, in general, do not have any requirements with respect to water-proofing beyond the control afforded by grouting, but this may not apply to treated water reservoirs. For most cavern uses dripping into the cavern should be controlled to avoid damage to water-sensitive goods and equipment and to avoid nuisance. Drainage installed behind shotcrete should be considered. Figure 30 shows typical details of such drainage. The method is effective but is dependent on good workmanship. Drip screens have considerable advantages in that they are cheap, can be quickly erected and are less dependent on good workmanship being achieved. Drip screens can form a pleasing architectural finish and they also prevent any minor loose particles from falling into the cavern. PVC fabric and coated polyester fabric membranes may be considered where no moisture from the rock is allowed to enter the cavern. Concrete lining is very expensive and should not be introduced for reasons of water exclusion alone.

5.11 CONSTRUCTION TIME

5.11.1 Rates of Excavation

High production rates of tunnelling by drill and blast are common. The following rates can be achieved:

Cross-sectional area, m ²	Advance, metres/week	
	Achievable	Hong Kong experience
< 12	80 to 150	50
12 to 35	70 to 120	40
35 to 75	> 60	35
75 to 120	> 50	30

These rates are for a working week of 10 shifts (about 100 hours) per week with good rock conditions where only a small amount of rock support is required.

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.

Rock support and groundwater control requirements on occasion 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 m³ to 10 000 m³ per week increasing to 60 000 m³ per week or more for multi-cavern excavations.

5.11.2 Rates of Support Installation

The preceding sections on rock support indicate the production rates that can be achieved. These rates are summarised as follows:

- (a) Rock bolting: 5 to 12 bolts per hour
- (b) Wet-mix shotcrete: 3 to 5 m³ per hour
- (c) Dry-mix shotcrete: 0.5 to 3 m³ per hour

The rates for rock bolting apply equally to tensioned bolts and untensioned grouted bolts. The production rates that can be 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.

5.12 CONSTRUCTION RECORDS

As-built drawings should be prepared for the works, which should include the line, level and location of all excavations. The cavern design is based, among other things, on rock quality predictions and support estimates made from ground investigation results. The as-built drawings should include engineering geological maps of the rock exposed by the excavation, with records of the support and groundwater mitigation measures installed, in sufficient detail to confirm the execution of the design.

6. QUALITY CONTROL, MONITORING AND MAINTENANCE

6.1 INTRODUCTION

Underground works have to be monitored and maintained in the same way as any other construction.

Monitoring does not, in itself, ensure good performance. Performance is affected fundamentally by the design and construction. Quality control and quality assurance is therefore needed for each stage of 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 periodic inspection of rock surfaces and shotcrete is mandatory for most occupied caverns.

Where long term monitoring of any type is required, or where there are maintenance aspects requiring particular attention, a monitoring and maintenance manual should be prepared. This should include relevant construction details, e.g. support works and drainage.

6.2 QUALITY CONTROL

Quality control and assurance routines should be instituted for all stages of project development from feasibility study through to construction and operation. The routines should be appropriate to the stage of project development. Independent detailed checks of engineering drawings, analyses and reports for the feasibility study should be carried out. Formalised routines may be required for the detailed design and tender document production. The construction phase will require specification of acceptance tests for components of the construction, e.g. rock bolts and shotcrete, and inspection and acceptance of the work as set out in conventional specifications. In addition, quality assurance routines should be instituted to ensure that the specified quality is achieved.

All quality control and assurance routines should be suited to the purpose of achieving good quality within a given time and with minimum waste. They should be such that the people involved perceive the routines as helping them to achieve the necessary quality and not as a hindrance to efficient execution. Thus the routines should be sufficient for their purpose and no more complex or time-consuming than necessary.

Of particular importance in underground projects are the works needed to ensure adequate stability of the opening. Rock bolts and shotcrete must therefore be installed in a manner that will ensure efficacy and durability. Suitable acceptance criteria for both should be specified. These should cover the method of placement as well as the end product, as acceptance of production rock bolts and shotcrete should largely be based on the method of placement. For example, testing production rock bolts in significant numbers is not usually practicable or necessary.

6.3 MONITORING

6.3.1 Need for Monitoring

All cavern installations require monitoring by direct observation or by instrumentation. The monitoring should be designed to reveal developing adverse conditions, thus allowing timely corrective action.

The majority of caverns in Hong Kong will be constructed in hard rock at shallow depth. The most likely cause of problems developing during construction and operation will be a progressive loosening of isolated blocks of rock from an essentially stable rock mass. Instrumentation is not suited to detecting this type of failure, but visual examination of the bare rock or the shotcreted or fibrecrusted surface can give warning of developing problems. This inspection must be done by staff qualified in rock engineering who have adequate practical experience of cavern construction.

Most movement resulting from stress redistribution takes place during and shortly after excavation. Similarly, loads on support works reach long-term values quickly. Thus few caverns in hard rock environments at shallow depth require instrumentation to monitor their performance, unless of unusually large span or complexity for the particular locality.

Monitoring is required for rock masses within a cavern with clearly defined failure mechanisms, such as major unstable wedges, where the stabilising measures have been the subject of analytical or numerical design.

Most instrumental monitoring of caverns in hard rock is related to groundwater, particularly for caverns where groundwater maintenance is an essential part of the design, and to caverns constructed close to existing facilities where monitoring may provide an early warning of undesirable movement. The monitoring of various parts of cavern works is detailed below.

Caverns with spans greater than 25 m, heights greater than 40 m or cover less than half the span or height, will usually require monitoring with instruments during construction. This applies particularly to complex installations which have been designed with the aid of numerical models and where field data (deformation, stresses and water pressure data) collected early in the construction period are used to finalise the design. In such cases the monitoring is properly part of the design process and should be planned and executed accordingly. Once this iterative design process is completed, further monitoring with instruments, except groundwater monitoring, is not normally required.

Monitoring techniques are described extensively in the literature. Franklin (1977) gives an overview of the monitoring of structures in rock.

6.3.2 Exposed Rock Surfaces

Where rock falls would be a cause for concern, all bare rock surfaces should be inspected and, if necessary, manually scaled one year after commissioning and thereafter at five-yearly intervals.

6.3.3 Shotcrete and Fibrecrete

All roofs covered with shotcrete and fibrecrete have to be checked from time to time to discover sections which may have become de-bonded from the rock and which might fall with time. The monitoring should be done by visual inspection of the whole surface for signs of cracking and other distress. In particular, note should be taken of any cracks that might suggest a loosening of rock. Remedial measures should be instituted where necessary. Inspections should be made one year after commissioning and thereafter at five-yearly intervals.

This check is not required if drip-screens have been installed. Instead, the tops of the screens and guttering should be checked for any fallen material. In the event that any such material is discovered, an inspection of the roof above the screens should be called for and remedial measures instituted as necessary.

6.3.4 Displacement Measurements

Monitoring of displacements in the surrounding rock is normally done by convergence or extensometer measurements. Tell-tales can be used to good effect to detect local relative movement between adjacent blocks of rock. Levels of the land surface above caverns may require monitoring, particularly if the cavern is at shallow depth.

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 wire and the accuracy of this type of measurement is up to ± 0.1 mm plus 10^{-6} of the measured distance. Convergence measurements will 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.

6.3.5 Rock Bolts

Although not common practice, the effectiveness of fully grouted bolts can be checked using a hollow steel bolt with the same load-deformation properties as the non-instrumented bolts. The hollow bolt is provided with internal rods of different lengths fixed at their ends to the wall of the hollow bolt. The strain of the individual rod sections can thus be measured. Exceptionally, partially grouted or shell anchored bolts can be controlled by disk load cells placed between the bearing plate and the nut of the rock bolt.

6.3.6 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).

6.3.7 Groundwater

Groundwater levels should be monitored wherever groundwater maintenance is required for operational or environmental reasons. This should be done with piezometer installations. Monitoring frequency and the number and location of monitoring points should be established for each case separately. The measurement of seepage flows into caverns may be a desirable addition to groundwater monitoring.

6.3.8 Rock Temperature

Rock temperatures may have to be monitored for environmental reasons where a cavern is being operated at temperatures substantially different from ambient, such as cold stores. This monitoring would have little influence on design or operation, but may be required to reassure third parties and may provide useful data for future schemes.

6.3.9 Stress Measurement

Hydraulic stress cells can be placed around the circumference of the 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 tangential stresses in the concrete.

6.3.10 Radon

Radon occurs naturally in many geological environments, including those in Hong Kong. Radon can be a radiation hazard if concentrations of the gas and its decay products exceed safety limits. In caverns, the concentration of radon can be kept low by ventilation systems, both during construction and operation. Nevertheless, radiation levels should be monitored, particularly after blasting. Until local standards for radon concentrations are established, the concentration should not be allowed to exceed 200 Bq/m³ as long as there are persons present.

6.4 MAINTENANCE

Maintenance of caverns is aimed at repairing defects discovered during routine inspection. This usually involves hand scaling of exposed rock surfaces and the removal or repair of shotcrete de-bonded from the rock surface. Shotcrete may be repaired by carefully cleaning the affected rock surface and respraying with shotcrete. In many cases shotcreting

is not possible due to the dirtiness of the process. Cast concrete, rock bolts, nets, straps and mortar may be used as substitutes, separately or in combination.

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TABLES

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LIST OF TABLES

Table no.		Page no.
1	Examples of Cavern Usage: Oil, Gas and Other Storage	105
2	Examples of Cavern Usage - Commercial, Community and Municipal use	106
3	Large Span Tunnels and Cavern Constructed in Hong Kong	107
4	Typical Uniaxial Compressive Strengths of Hong Kong Rocks	108
5	Ratings for Q-system Parameters: RQD	109
6	Ratings for Q-system Parameters: Joint Set Number J_n	109
7	Ratings for Q-system Parameters: Joint Roughness Number J_r	110
8	Ratings for Q-system Parameters: Joint Alteration Number J_a	111
9	Ratings for Q-system Parameters: Joint Water Reduction Factor J_w	112
10	Ratings for Q-system Parameters: Stress Reduction Factor SRF	113
11	Excavation Support Ratios ESR	114
12	RMR Geomechanics Classification of Jointed Rock Masses Classification Parameters and Their Ratings	115
13	RMR Geomechanics Classification of Jointed Rock Masses Rating Adjustment for Joint Orientations	115
14	RMR Geomechanics Classification of Jointed Rock Masses Rock Mass Classes Determined from Total Ratings	116
15	RMR Geomechanics Classification of Jointed Rock Masses The Effect of Joint Strike and Dip Orientations in Tunnelling	116
16	Ranges of Sizes for Different Methods of Shaft Construction	116
17	Vibration Damage Limits for Buildings for Selected Countries	117

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Table 1 - Examples of Cavern Usage: Oil, Gas and Other Storage

Use	Location	Reasons for Use	Span x Height	Pillar Width	Rock Type	Other Details and Reference
Crude storage	Mongstad, Norway	Cost, safety, environment	22 x 30 15 x 15 18 x 33	50	Meta-anorthosite, gneiss	5 caverns (Froise, 1988)
Crude storage	Sture, Norway	Cost, safety, environment	19 x 33			4 caverns
Crude storage	Brofjorden, Sweden	Cost, safety, environment	20 x 30	35	Granite	6 caverns (Stille, 1986)
Products	Oslo, Norway	Cost, safety, environment	12 x 10 15 x 10		Granite	Regional fuel store
LPG store	Mongstad, Norway	Cost, safety, environment	22 x 26		Meta-anorthosite, gneiss	
Gasoline, naphtha	Heroya, Norway	Cost, safety, environment	10 x 15			
Propane	Rafsnes, Norway	Cost, safety, environment	19 x 22		Granite	(Pettersen, 1976)
Ammonia	Heroya, Norway	Cost, safety, environment	10 x 10		Limestone	
Propane/butane	Yeosu, S. Korea	Cost, safety, environment	18 x 22			170,000 m ³ propane 120,000 m ³ butane
General	Kansas, USA	Cheap space in limestone mine			Limestone	Room and pillar (Woodard, 1980).
Cold store	Bergen, Norway	Low running cost	20 x 11		Granitic gneiss	(Barbo & Bollingmo, 1982)
Cold store	Sweden	Low running cost	16 x 8			3 caverns
Wine and liquor	Aristodal, Sweden	Low running cost	20 x ?			2 caverns, completed 1957
Solar heated water	Lyckebo, Sweden	Cost, thermal insulation	18 x 30	35		Annular cavern (Pilebro et al, 1987)
Industrial waste	Odda, Norway	Environment			Gneiss	Aluminium smelter waste (Aarvoll et al, 1987)
Nuclear waste	Forsmark Stripa, Sweden	Containment	69 x 30		Gneiss	Cylindrical

Table 2 - Examples of Cavern Usage: Commercial, Community and Municipal Use

Use	Location	Reasons for Use	Span x Height	Pillar Width	Rock Type	Other Details and Reference
Sports centre, community	Holmen, Norway	Combined with civil defence shelter	25 x 13 15 x 9.5	12	Nodular limestone and shale	(Rygh, 1982)
Sports centre, community	Holmlia, Norway	Combined with civil defence shelter	25 x 13 15 x 9.5	12	Granite	(Rygh, 1982)
Sports centre	Tampere, Finland	Combined with civil defence shelter	32 x ?		Porphyritic granite	(Paavola, 1986)
Tourism	North Cape Norway	Environment, commercial				Theatre, restaurants, viewing galleries
Water treatment	Oslo, Norway	Cost, land take	13 x 16		Syenite	(Saetersmoen, 1982)
Water storage	Oslo, Norway	Cost, land take			Syenite	50,000 m ³ treated water (Saetersmoen, 1982)
Refuse transfer	Stockholm, Sweden	Shorter travelling time, land take				
Sewage treatment	Käppala Sweden	Environment, land take	12 x ?			450,000 p.e. (population equivalent)
Sewage treatment	Oslo, Norway	Environment, land take	16 x 10	12	Shale and limestone	600,000 p.e. (Medbo & Loset, 1982)
Sewage treatment	Henriksdal, Sweden	Environment, land take				518,000 p.e.
Sewage treatment	Stavanger, Norway	Environment, land take			Phyllite	240,000 persons (Asting, 1990)

Table 3 - Large Span Tunnels and Caverns Constructed in Hong Kong

Name and Purpose	Max. Span	Shape	Maximum Overburden	Rock		Design Method	Construction Problems	References
				Type	Condition			
Taikoo Station cavern. MTR station.	24 m	Rectangular base, arch roof	80 m, now reduced to 11 m	Granite, medium to coarse	Good	Q-system and boundary elements		Wong & Croix-Marie, 1989
Eastern Harbour Crossing, Quarry Bay Station. Traffic.	15 m	Rectangular base, elliptical roof.	170 m	Granite	Excellent	Q-system		Matson, 1989
Tate's Cairn Tunnel. Traffic.	12 m	Rectangular base, circular roof	390 m	Granite	Fair	NATM, Q-system, RMR		
Tseng Kwan O Tunnel. Traffic.	10.8 m	Horseshoe	70 m	Volcanics	Fair	Q-system		
Bowen to Wah Fu cable tunnel.	7.2 m	Rectangular base, circular roof	285 m	Volcanics	Good	IMS		
Aberdeen Road Tunnel.	10.1 m	Rectangular, circular roof	260 m	Granite	-	NATM, computer modelling		Twist & Tonge, 1979
Lion Rock Tunnel. Traffic.	9.8 m	Horseshoe	320 m	Granite	Good to fair	Empirical (?)	Water inflows, alteration zones	Davis, 1963 Payne, 1963
New Beacon Hill Tunnel. Traffic.	10.6 m	Horseshoe	400 m	Granite	Good to fair	Empirical (?)		
Western Aqueducts, Phase II, Portal G.	11 m	Horseshoe	100 m	Volcanics	Fair	Empirical	Weathered seams	
Mass Transit Railway shunting tunnels from Fortress Hill to Sai Wan Ho.	7.6 m	Horseshoe	30 m	Granite	Good to very good	Empirical	Water inflows	

Table 4 - Typical Uniaxial Compressive Strengths of Hong Kong Rocks

Material Decomposition Grade	Uniaxial Compressive Strength, MPa		
	Granites	Volcanics	Granodiorite
Fresh	101 - 179 ² 150 ± 25 ⁴	150 - 340 ¹	170 - 290 ¹ 150 - 200 ³
Slightly Decomposed	101 - 179 ² 82 ± 22 ⁴	110 - 190 ¹	120 - 215 ¹ 75 - 175 ³
Moderately Decomposed	85 ² 33 ± 9 ⁴	10 - 120 ¹	10 - 145 ¹ 15 - 100 ³
<p>Notes:</p> <p>1 Table based on: ¹ Irfan (1985) ² Irfan et al (1991). ³ Irfan & Powell (1985) ⁴ Roberts (1991).</p> <p>2 Figures following ± are standard deviations.</p>			

Table 5 - Ratings for Q-system Parameters: RQD

1 ROCK QUALITY DESIGNATION	RQD
A Very poor	0 - 25
B Poor	25 - 50
C Fair	50 - 75
D Good	75 - 90
E Excellent	90 - 100
Notes: (1) Where RQD is reported or measured as ≤ 10 , (including 0), a nominal value of 10 is used to evaluate Q in equation (1), page 46.	
(2) RQD intervals of 5, i.e. 100, 95, 90 etc., are sufficiently accurate.	

Table 6 - Ratings for Q-system Parameters: Joint Set Number J_n

2 JOINT SET NUMBER	J_n
A. Massive, no or few joints	0.5 - 1.0
B. One joint set	2
C. One joint set plus random joints	3
D. Two joint sets	4
E. Two joint sets plus random joints	6
F. Three joint sets	9
G. Three joint sets plus random joints	12
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15
J. Crushed rock, earth-like	20
Notes: (1) For intersections use $(3 \times J_n)$	
(2) For portals use $(2 \times J_n)$	

Table 7 - Ratings for Q-system Parameters: Joint Roughness Number J_r

3. JOINT ROUGHNESS NUMBER	J_r
<p>(a) <i>Rock wall contact and</i> (b) <i>Rock wall contact before 100 mm shear</i></p> <p>A. Discontinuous joints B. Rough or irregular, undulating C. Smooth, undulating D. Slickensided undulating E. Rough or irregular, planar F. Smooth, planar G. Slickensided, planar</p> <p>(c) <i>No rock wall contact when sheared</i></p> <p>H. Zones containing clay mineral thick enough to prevent rock wall contact J. Sandy, gravelly or crushed zones thick enough to prevent rock wall contact</p>	<p>4 3 2 1.5 1.5 1 0.5</p> <p>1 1</p>
<p>Notes: (1) Descriptions refer to small-scale features and intermediate scale features, in that order. (2) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 metres. (3) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.</p>	

Table 8 - Ratings for Q-system parameters: Joint Alteration Number J_a

4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
<i>(a) Rock wall contact</i>		
A. Tightly healed, hard, non-softening, impermeable filling, e.g quartz or epidote	0.75	-
B. Unaltered joint walls, surface staining only	1.0	25 - 35°
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	25 - 30°
D. Silty or sandy clay coatings, small clay fraction (non-softening)	3.0	20 - 25°
E. Softening or low friction clay mineral coatings, i.e. kaolin or mica. Also chlorite, talc, gypsum, graphite etc., and small quantities of swelling clays	4.0	8 - 16°
<i>(b) Rock wall contact before 100 mm shear</i>		
F. Sandy particles, clay-free disintegrating rock, etc.	4.0	25 - 30°
G. Strongly over-consolidated non-softening clay mineral fillings (continuous, but < 5 mm thickness)	6.0	16 - 24°
H. Medium or low over-consolidation, softening, clay mineral fillings (continuous but < 5 mm thickness)	8.0	12 - 16°
J. Swelling clay fillings, i.e. montmorillonite (continuous, but < 5 mm thickness). Value of J_a depends on percentage of swelling clay-size particles, and access to water	8 - 12	6 - 12°
<i>(c) No rock wall contact when sheared</i>		
K,L, Zones or bands of disintegrated or crushed rock	6, 8 or	
M. and clay (see G, H, J for description of clay condition)	8 - 12	6 - 24°
N. Zones or bands of silty or sandy clay, small clay fraction (non-softening)	5.0	-
O,P, Thick continuous zones of clay (see G,H,J for	10, 13 or	
R. description of clay condition)	13 - 20	6 - 24°

Table 9 - Ratings for Q-system Parameters: Joint Water Reduction Factor J_w

5. JOINT WATER REDUCTION FACTOR	J_w	Approx. water pressure kPa
A. Dry excavations or minor inflow, i.e. < 5 l/min. locally	1.0	< 100
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	100 - 250
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	250 - 1 000
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	250 - 1 000
E. Exceptionally high inflow or water pressure at blasting, pressure decaying with time	0.2 - 0.1	> 1 000
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1 - 0.05	> 1 000
<p>Notes: (1) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.</p> <p>(2) Special problems caused by ice formation are not considered.</p>		

Table 10 - Rating for Q-system Parameters: Stress Reduction Factor SRF

6. STRESS REDUCTION FACTOR			SRF
<i>(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated</i>			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)		10
B.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)		5.0
C.	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)		2.5
D.	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)		7.5
E.	Single shear zones in competent rock, clay-free (depth of excavation ≤ 50 m)		5.0
F.	Single shear zones in competent rock, clay-free (depth of excavation > 50 m)		2.5
G.	Loose or open joints, heavily jointed or 'sugar cube' etc. (any depth)		5.0
<i>(b) Competent rock, rock stress problems</i>			
		σ_c/σ_1	σ_t/σ_1
H.	Low stress, near surface	> 200	> 13
J.	Medium stress	200 - 10	13 - 0.66
K.	High stress, very tight structure (usually favourably to stability, may be unfavourable for wall stability)	10 - 5	0.66 - 0.33
L.	Mild rock burst (massive rock)	5 - 2.5	0.33 - 0.16
M.	Heavy rock burst (massive rock)	< 2.5	< 0.16
			0.5 - 2
			5 - 10
			10 - 20
<i>(c) Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure</i>			
N.	Mild squeezing rock pressure		5 - 10
O.	Heavy squeezing rock pressure		10 - 20
Note: This table continues on the next page.			

Table 10 (cont.) - Rating for Q-system Parameters: Stress Reduction Factor SRF

6. STRESS REDUCTION FACTOR (cont.)	SRF
<p>(d) <i>Swelling rock: chemical swelling activity depending on presence of water</i></p> <p>P. Mild swelling rock pressure</p> <p>R. Heavy swelling rock pressure</p>	<p>5 - 10</p> <p>10 - 15</p>
<p>Notes: (1) Reduce these values of SRF by 25 to 50% if the relevant shear zones only influence but do not intersect the excavation.</p> <p>(2) For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\sigma_c$ and $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compression strength, and σ_t = tensile strength (point load), and σ_1 and σ_3 are the major and minor principal stresses.</p> <p>(3) Few case records available where depth of crown below surface is less than the span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).</p>	

Table 11 - Excavation Support Ratios ESR

Type of Excavation	ESR
A Temporary mine openings etc.	ca. 3 - 5 (?)
B Permanent mine openings, water tunnels for hydropower (excluding high pressure penstocks), pilot tunnels, drifts and headings for large openings	1.6
C Storage caverns, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels etc.	1.3
D Power stations, major road and railway tunnels, civil defence chambers, portals, intersections	1.0
E Underground nuclear power stations, railway stations, sports and public facilities, factories	ca. 0.8 (?)

Table 12 - RMR Geomechanics Classification of Jointed Rock Masses
Classification Parameters and Their Ratings

PARAMETERS			RANGE OF VALUES				
1	Strength of rock	Point load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range uniaxial compress. strength is preferred
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa 1-5 MPa <1 MPa
	Rating	15	12	7	4	2 1 0	
2	Drill core quality		90 - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of joints		> 2 m	0.6 - 2 m	0.2 - 0.6 m	60 - 200 mm	< 60 mm
	Rating		20	15	10	8	5
4	Condition of joints		Very rough surfaces. Not continuous. No separation. Hard joint wall rock.	Slightly rough surface. Separation < 1 mm. Hard joint wall rock.	Slightly rough surface. Separation < 1 mm. Soft joint wall rock.	Slickensided surface or Gouge < 5 mm thick or Joints open 1 - 5 mm Continuous joints.	Soft gouge > 5 mm thick or Joint open > 5 mm. Continuous joints.
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length	None	< 10 l/min.	10 - 25 l/min.	25 - 125 l/min.	> 125 l/min.
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5
		General condition	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

Table 13 - RMR Geomechanics Classification of Jointed Rock Masses
Rating Adjustment for Joint Orientations

Strike and dip orientation of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 14 - RMR Geomechanics Classification of Jointed Rock Masses
Rock Mass Classes Determined from Total Ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 21
Class No.	I	II	III	IV	V
Description	Very Good Rock	Good Rock	Fair Rock	Poor Rock	Very Poor Rock

Table 15 - RMR Geomechanics Classification of Jointed Rock Masses
The Effect of Joint Strike and Dip Orientations in Tunnelling

Strike Perpendicular to Tunnel Axis				Strike Parallel to Tunnel Axis		Dip 0° - 20° Irrespective of Strike
Drive With Dip		Drive Against Dip		Dip 45° - 90°	Dip 20° - 45°	
Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°			
Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable	Fair	Unfavourable

Table 16 - Ranges of Sizes for Different Methods of Shaft Construction

Method	Typical Length	Typical Diameters		Inclination
		minimum	maximum	
Longhole	< 50 m	2 m	> 10 m	min. 45°
Raise boring	< 1 000 m	0.5 m	6 m	any
Alimak	< 400 m	1.8 m	3.2 m	min. 45°
Raise building	< 200 m		> 10 m	vertical
Pilot shaft	> 400 m		> 10 m	min. 45°
Shaft sinking	> 1 000 m		> 10 m	vertical
Drilling	> 500 m	< 1 m	6 m	steeply inclined

Table 17 - Vibration Damage Limits for Selected Countries

Peak Particle Velocity, mm/s	Country			
	Sweden	Norway	Canada	USA (Bureau of Mines)
< 50			No damage	No damage
< 70	No cracking	No cracking		
< 110	Threshold value	Threshold value		Threshold value
< 115			Threshold value	
> 115			Damage	
< 150	Fine cracks	Fine cracks		Small damage
> 150				Severe damage
< 225	Cracking	Cracking		
> 225	Severe damage	Severe damage		

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FIGURES

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LIST OF FIGURES

Figure no.		Page no.
1	Geological Sketch Map of Hong Kong	123
2	Polar Stereographic Projection and Joint Rosette	124
3	Principles of Seismic Refraction Surveying	125
4	Typical Results of a Seismic Refraction Survey	126
5	Typical Results of a Cross-hole Tomographic Survey	127
6	Insitu Stress Measurement by Over-coring	128
7	Insitu Stress Measurement by Hydraulic Fracture	129
8	Drilling Rate Index (DRI) versus Bit Wear Index (BWI)	130
9	Rock Support Related to Orientation of Weakness Zones	131
10	Location of Unstable Intruding Edges and Corners	132
11	Typical Cost Curves for Different Cavern Spans and Heights	132
12	Standard Roof Arch Shape	133
13	Stability of Pillars with Potential Sliding Planes	133
14	Q - diagram with Support Recommendations	134
15	Spot-bolting of Isolated Blocks	135
16	Typical Roof Support with Pattern Bolting	135
17	Classification of Modelling Problems	136
18	Excavation Stages for Top Headings in Large Caverns	136
19	Typical Excavation Stages for a Large Cavern	137
20	Access Tunnel for High, Long Caverns	137
21	Typical Modern Drilling Jumbo	138
22	Typical Drilling Pattern for a Top Heading Blast	139
23	Shaft Excavation by the Long-hole Method	140
24	Shaft Excavation by Raise Boring	141
25	Alimak Method of Shaft Excavation	142

Figure no.		Page no.
26	Pilot Shaft Method of Shaft Excavation	143
27	Main Rock Bolt Elements	144
28	Principal Types of Rock Bolt	144
29	Pre-treatment of Clayey Seams for Shotcreting	145
30	Drainage Installed Prior to Shotcreting	145
31	Concrete Roof Lining	146
32	Typical Safe Vibration Limits	147
33	Cost Related to Vibration Restrictions	148
34	Specific Explosive Charge versus Cross-sectional Area	148

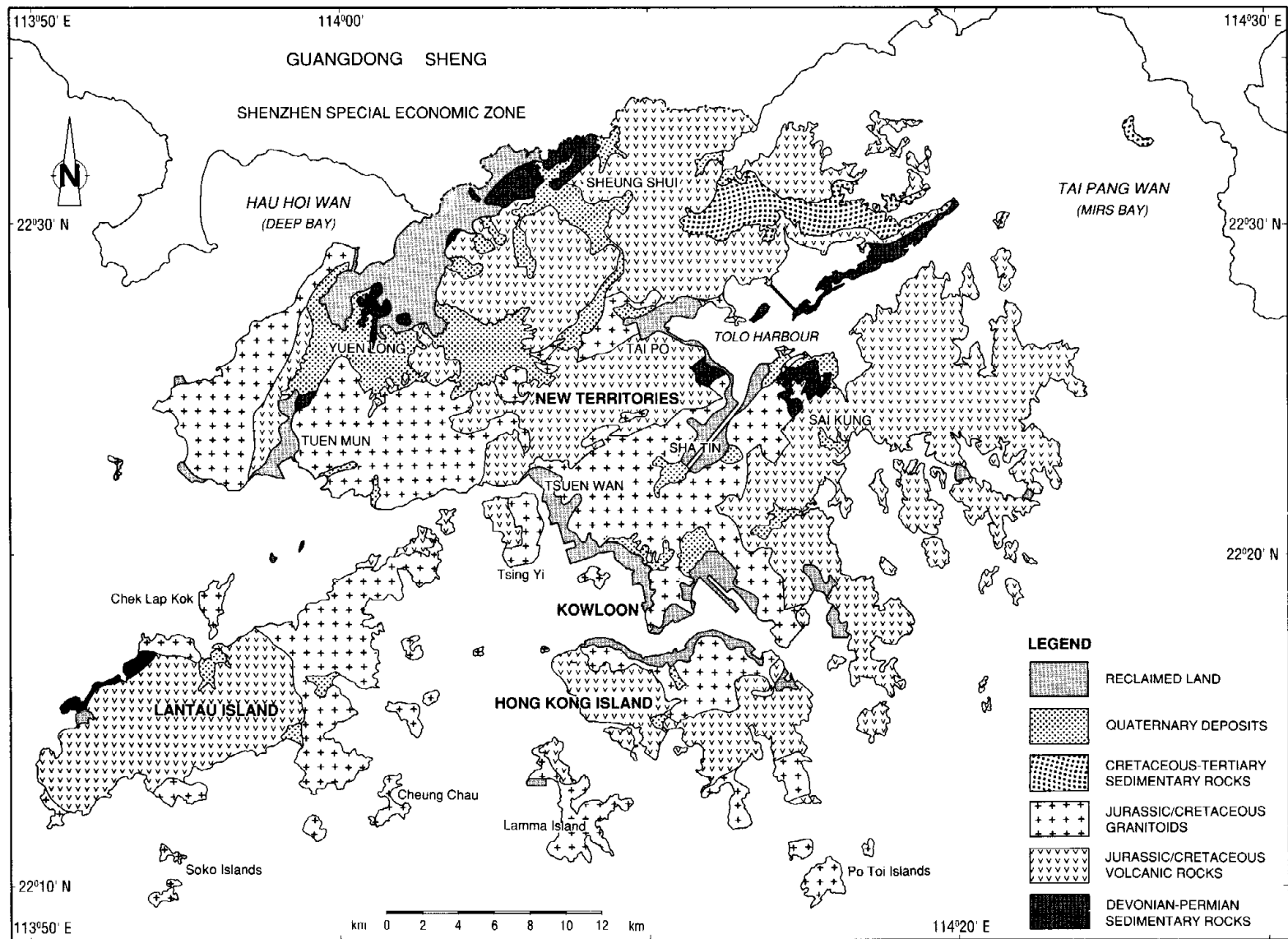


Figure 1 - Geological Sketch Map of Hong Kong

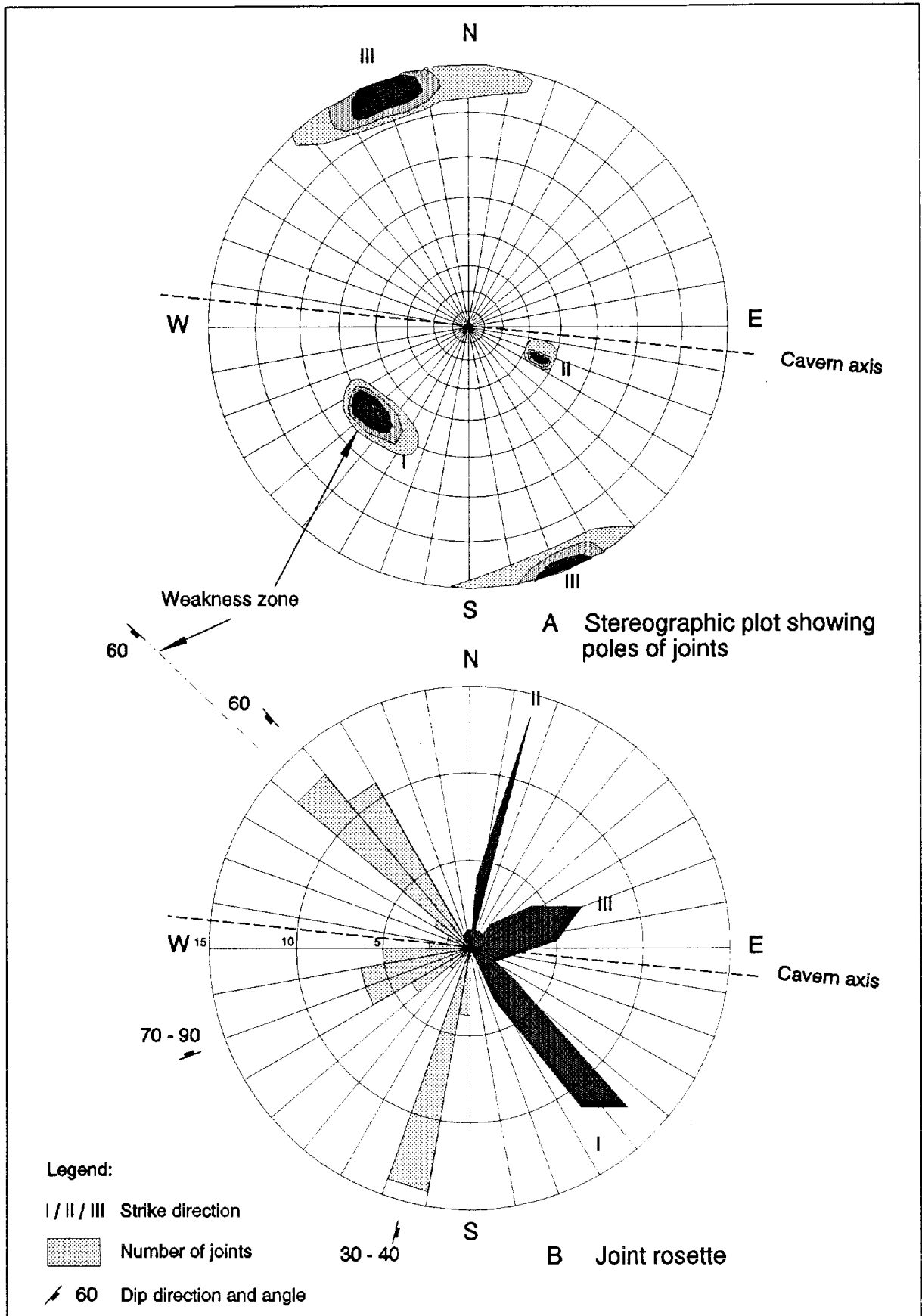


Figure 2 - Polar Stereographic Projection and Joint Rosette

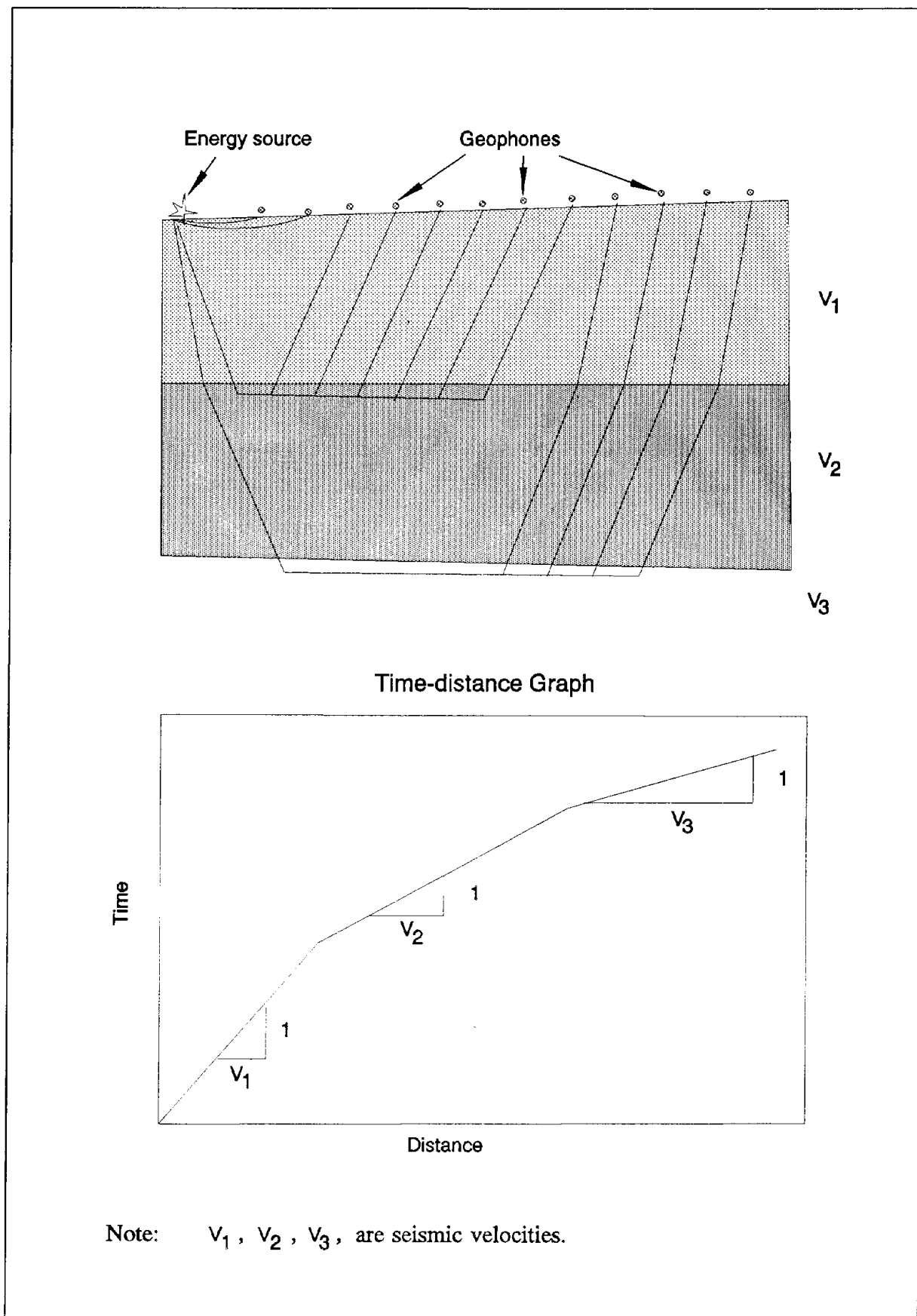


Figure 3 - Principles of Seismic Refraction Surveying

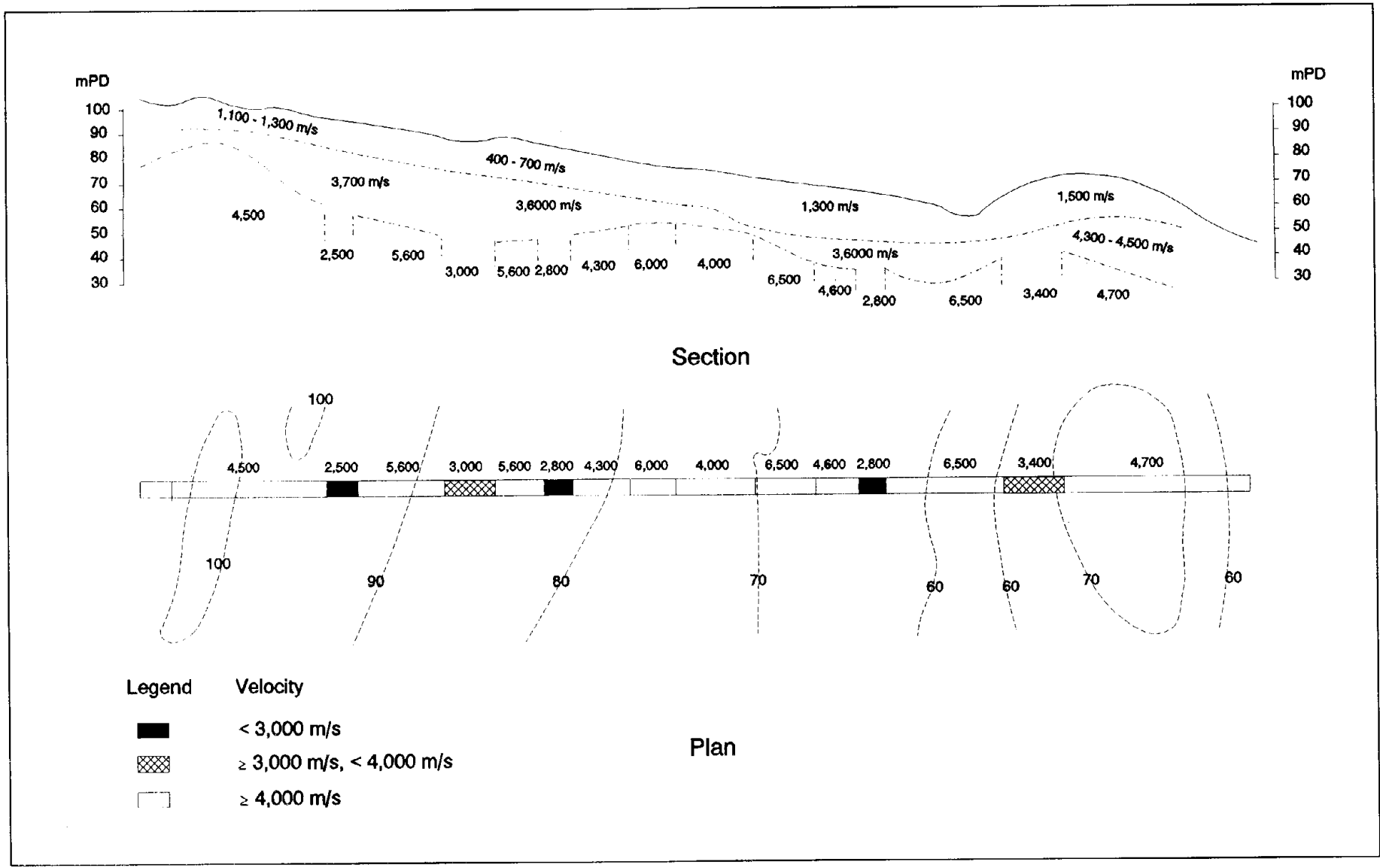


Figure 4 - Typical Results of a Refraction Seismic Survey

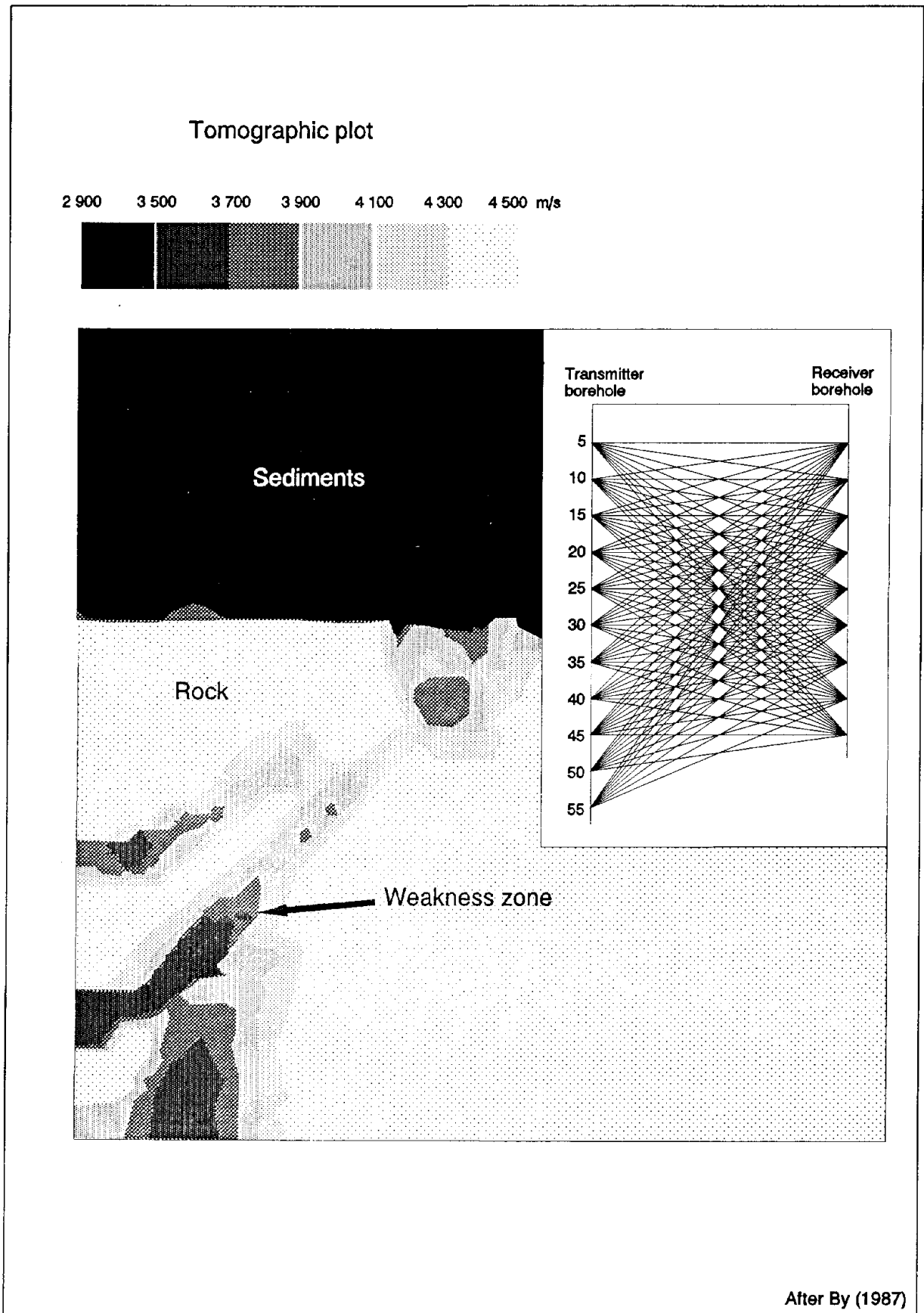


Figure 5 - Typical Results of a Cross-hole Tomographic Survey

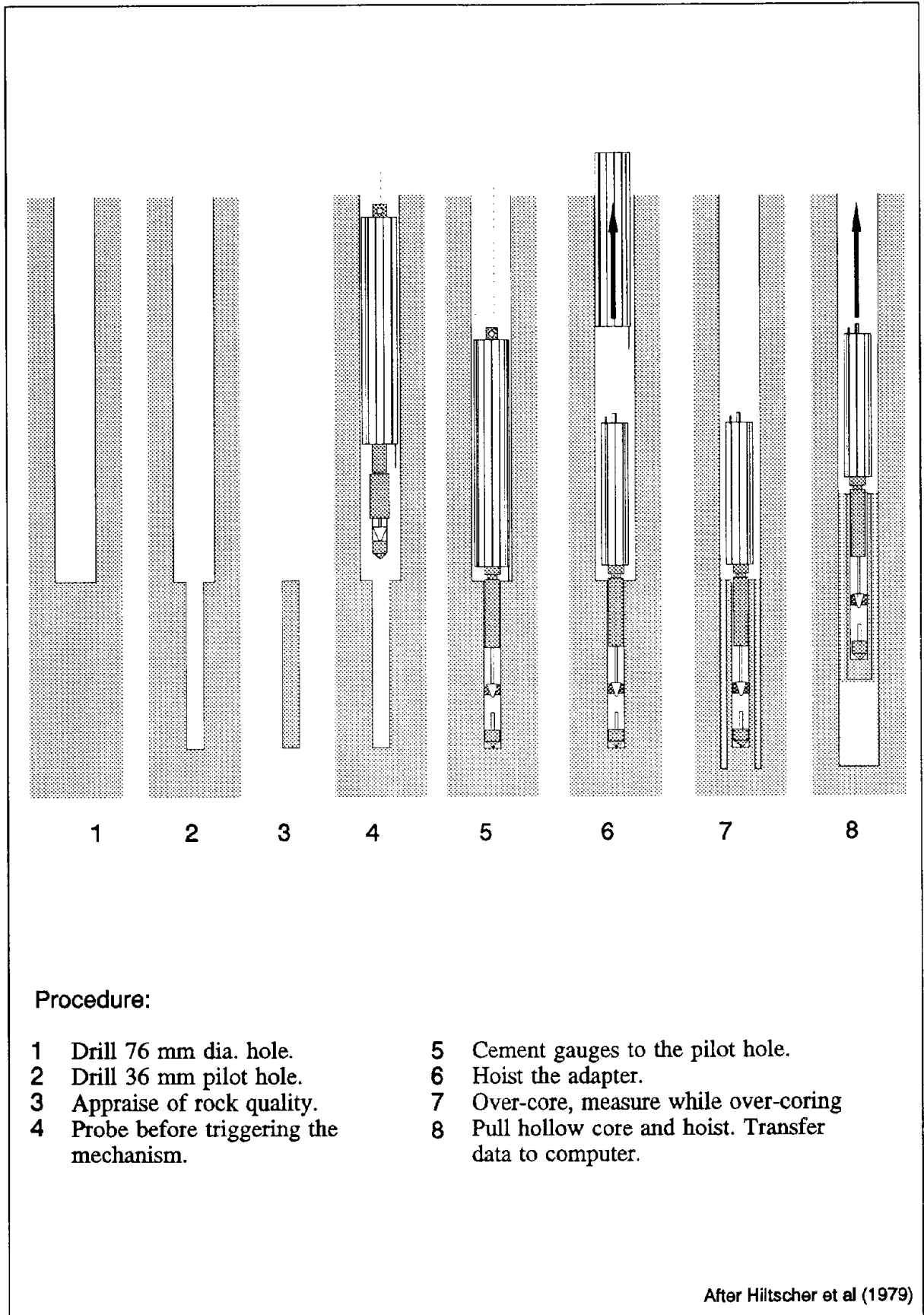


Figure 6 - Insitu Stress Measurement by Over-coring

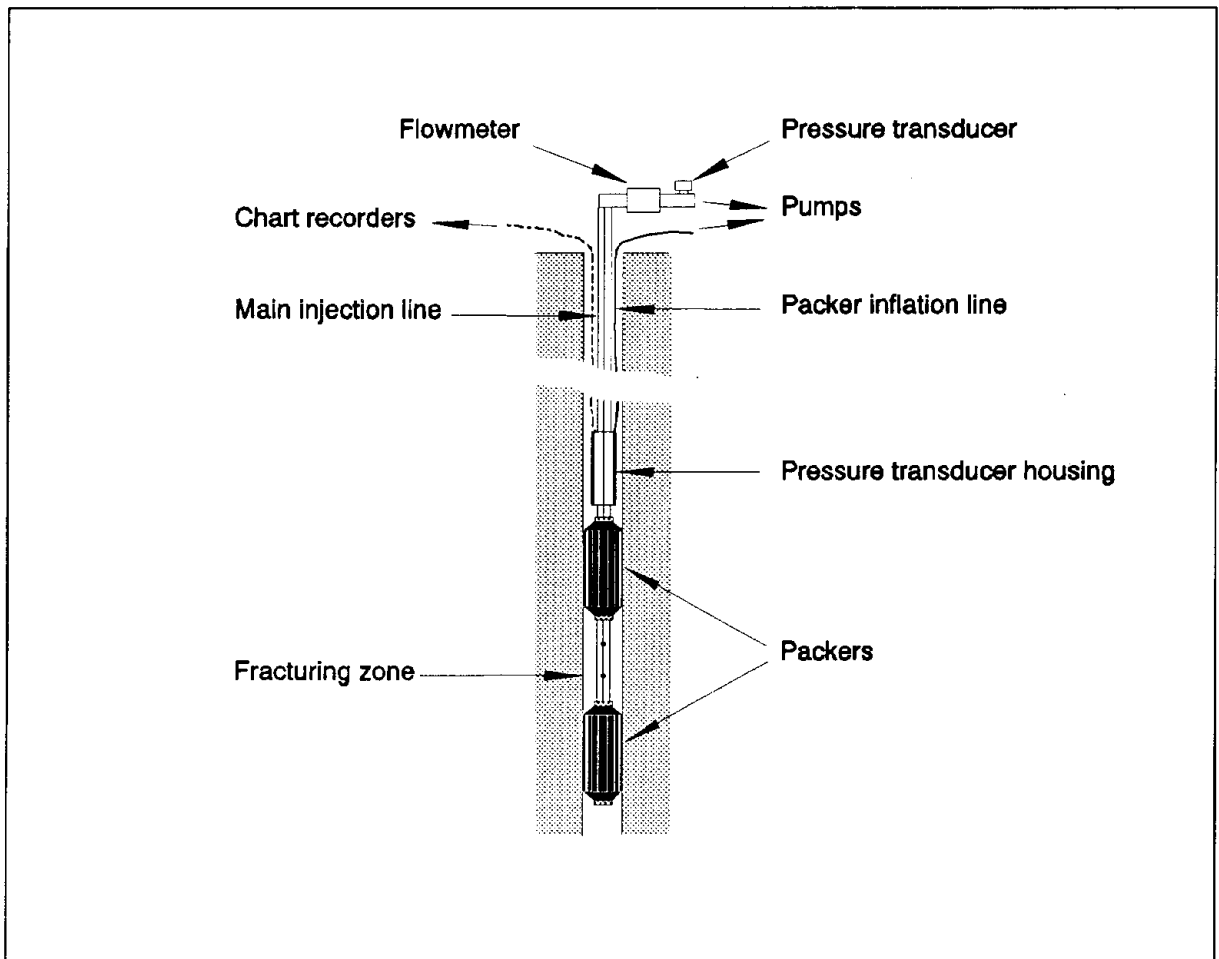


Figure 7 - In-situ Stress Measurement by Hydraulic Fracture

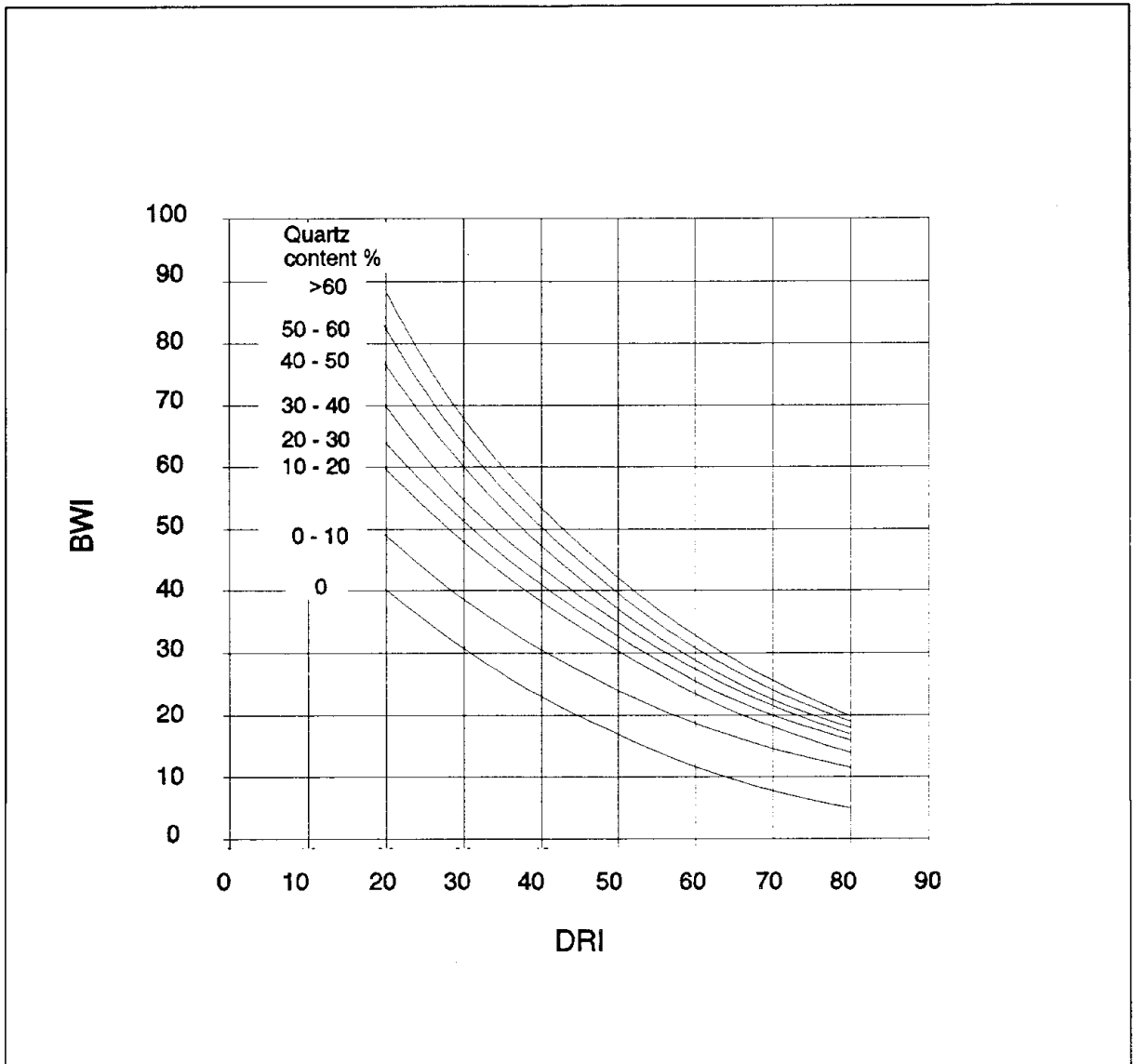
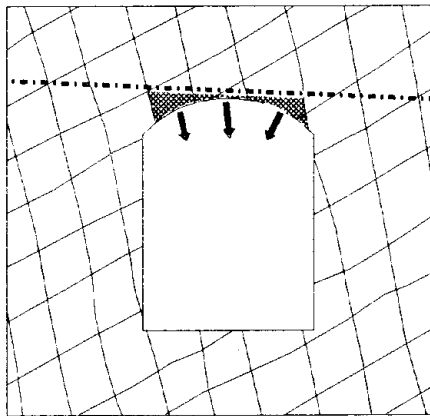
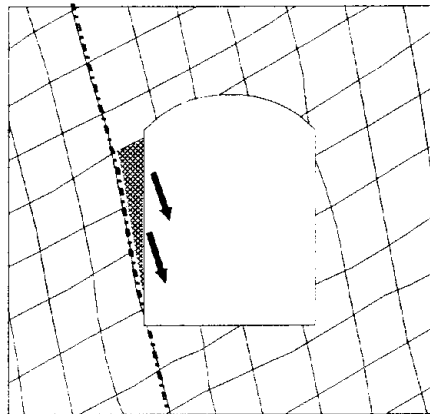


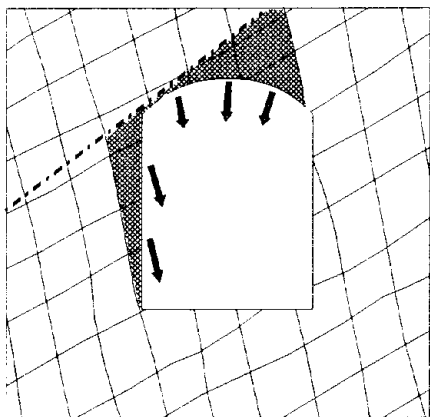
Figure 8 - Drilling Rate Index (DRI) versus Bit Wear Index (BWI)



Heavy roof support required



Heavy wall support required



Heavy wall and roof support required

Legend:

..... weakness zone

Figure 9 - Rock Support Related to Orientation of Weakness Zones

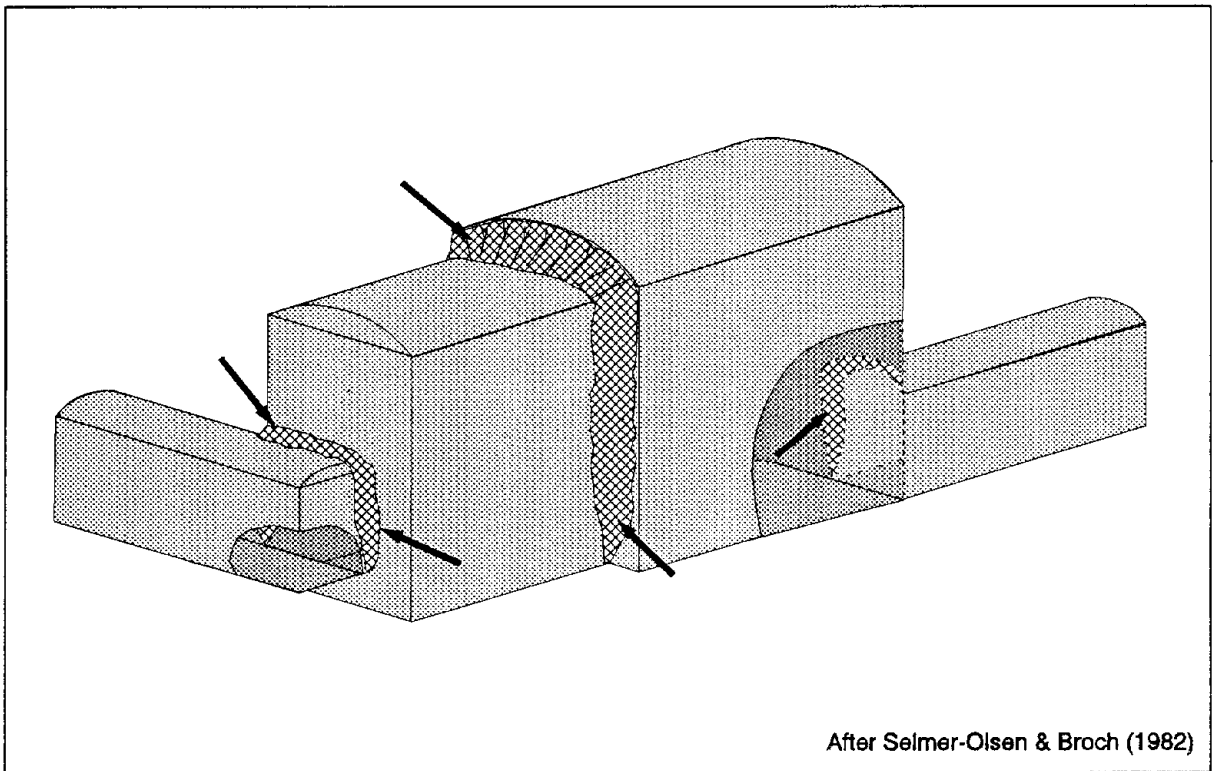


Figure 10 - Location of Unstable Intruding Edges and Corners

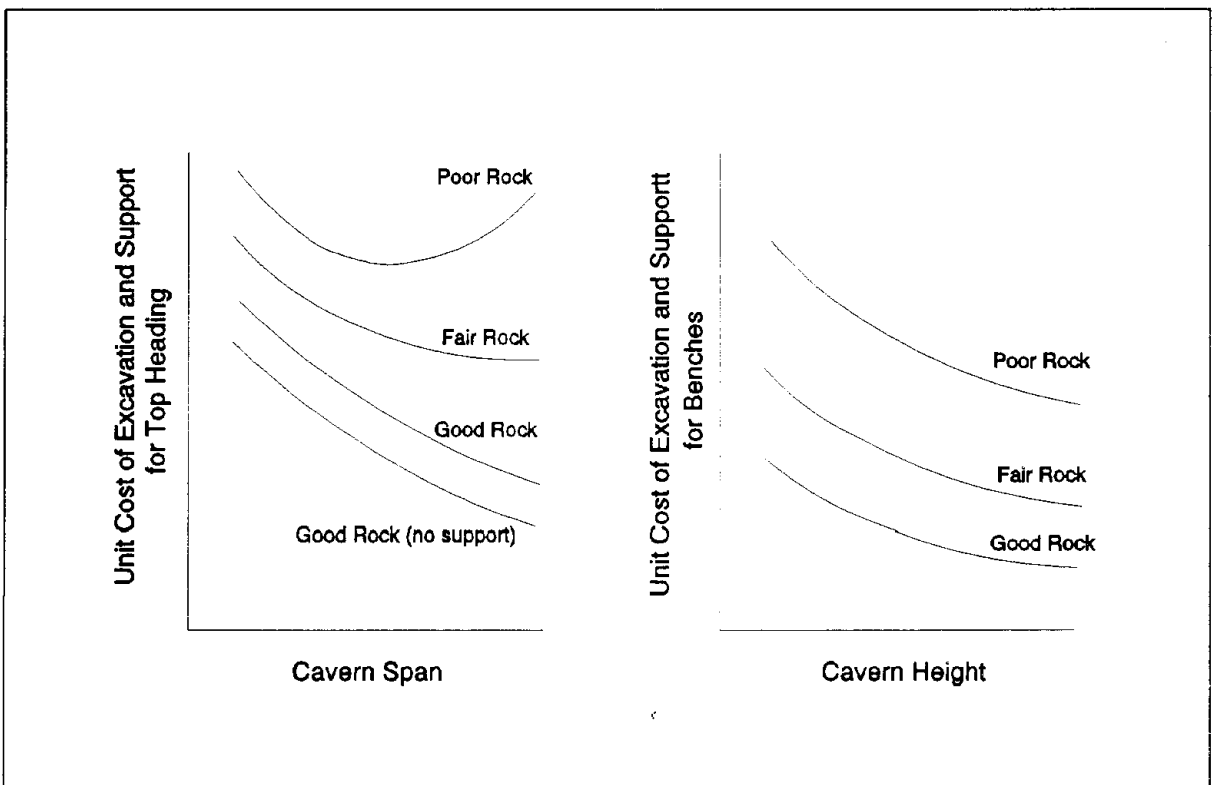


Figure 11 - Typical Cost Curves for Different Cavern Spans and Heights

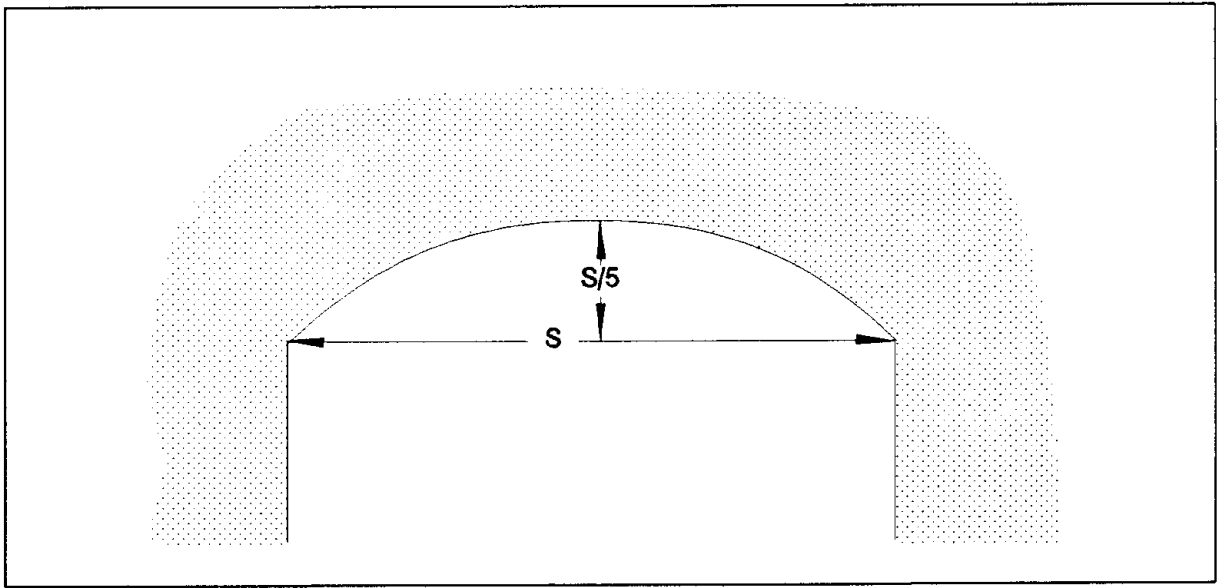


Figure 12 - Standard Roof Arch Shape

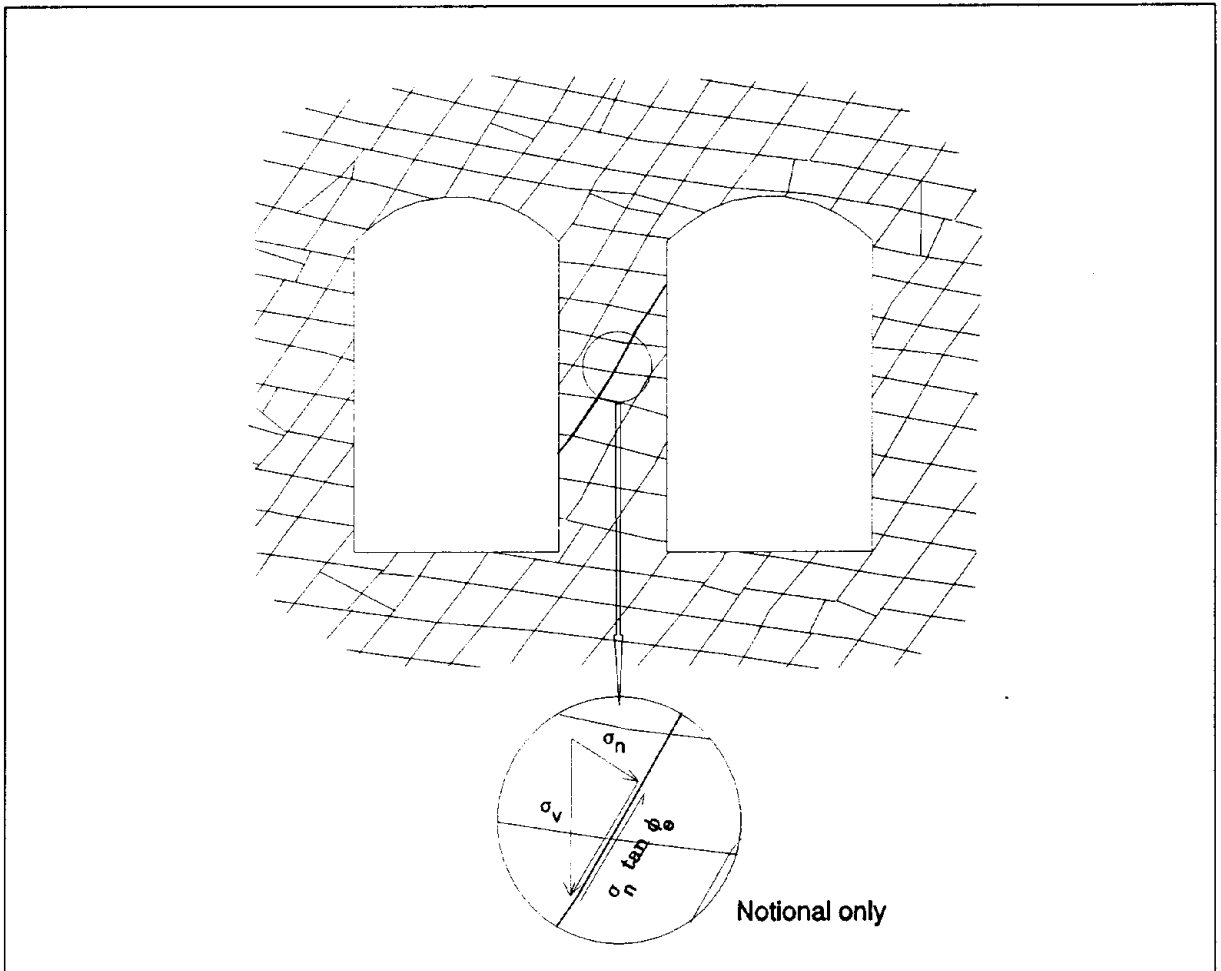


Figure 13 - Stability of Pillars with Potential Sliding Planes

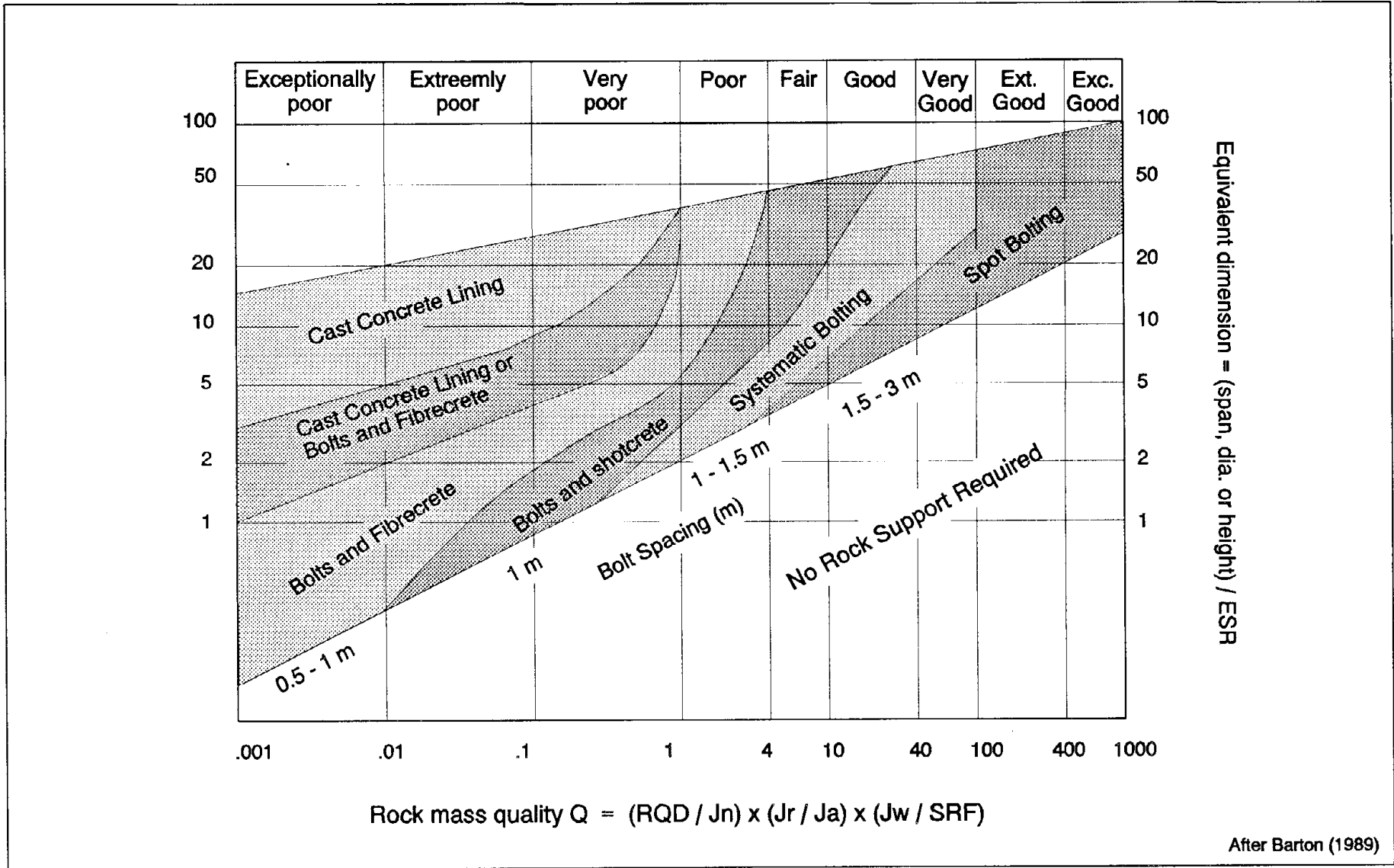


Figure 14 - Q - diagram with Support Recommendations

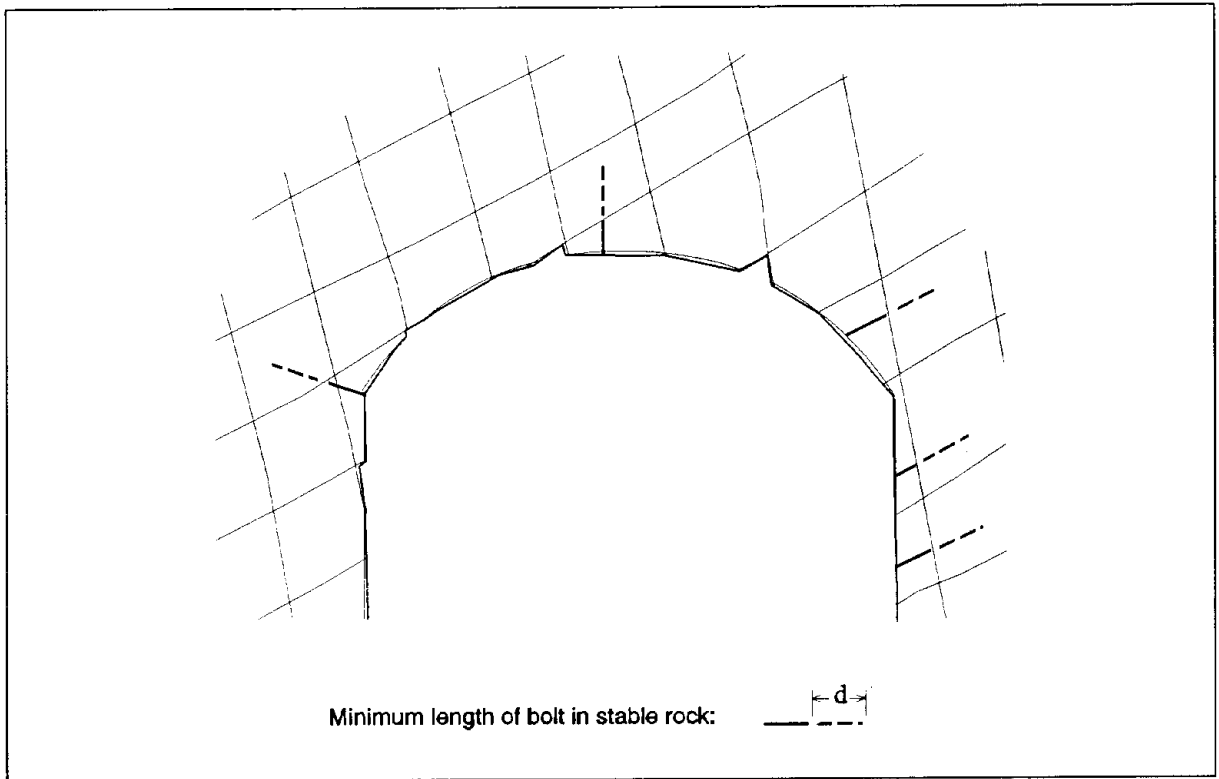


Figure 15 - Spot-bolting of Isolated Blocks

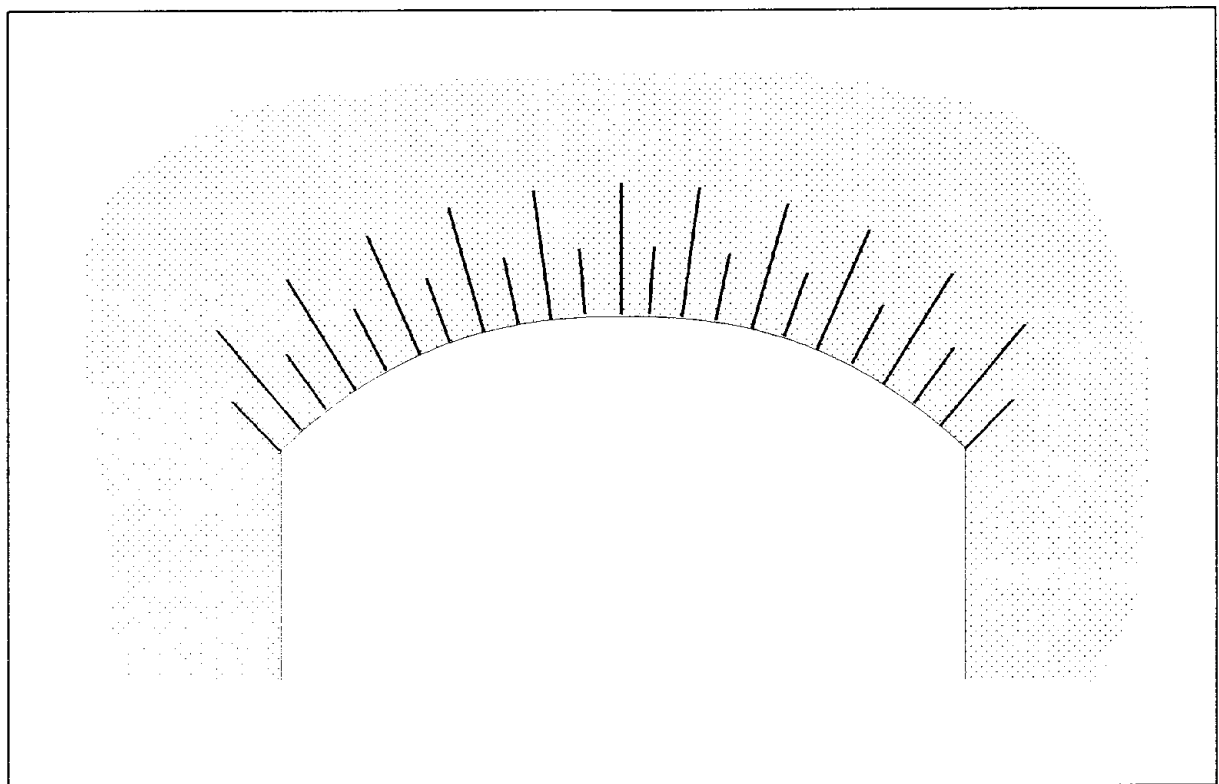


Figure 16 - Typical Roof Support with Pattern Bolting

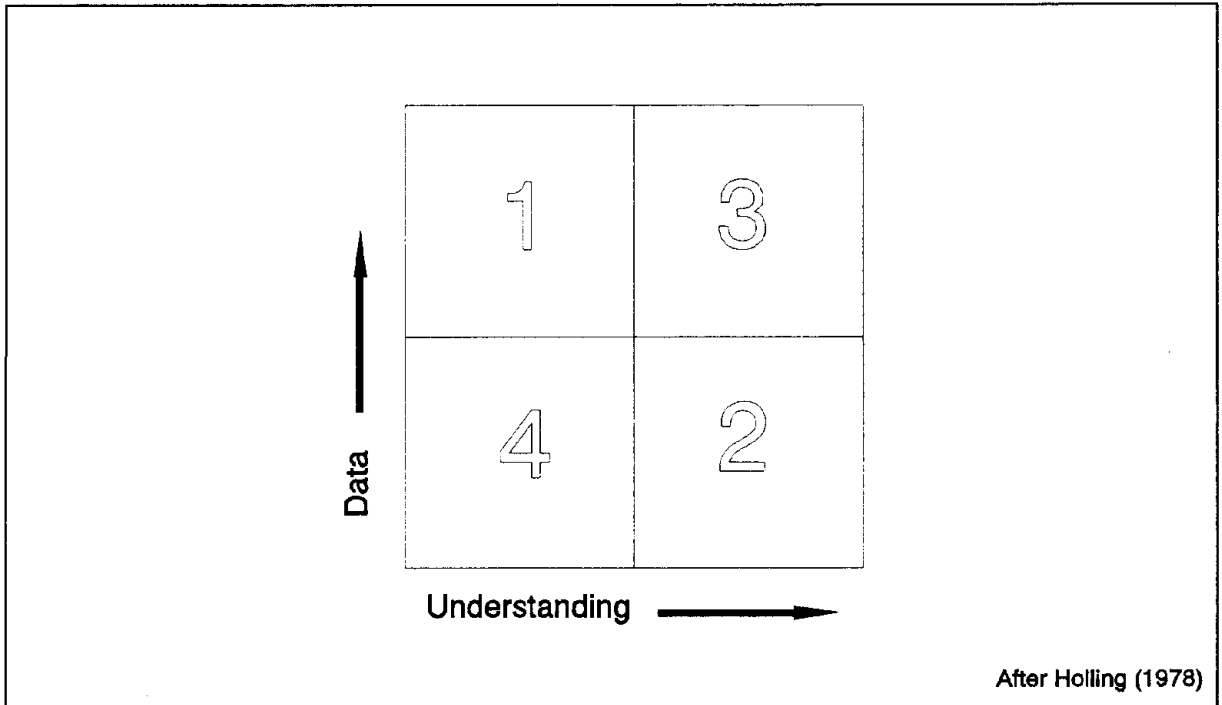


Figure 17 - Classification of Modelling Problems

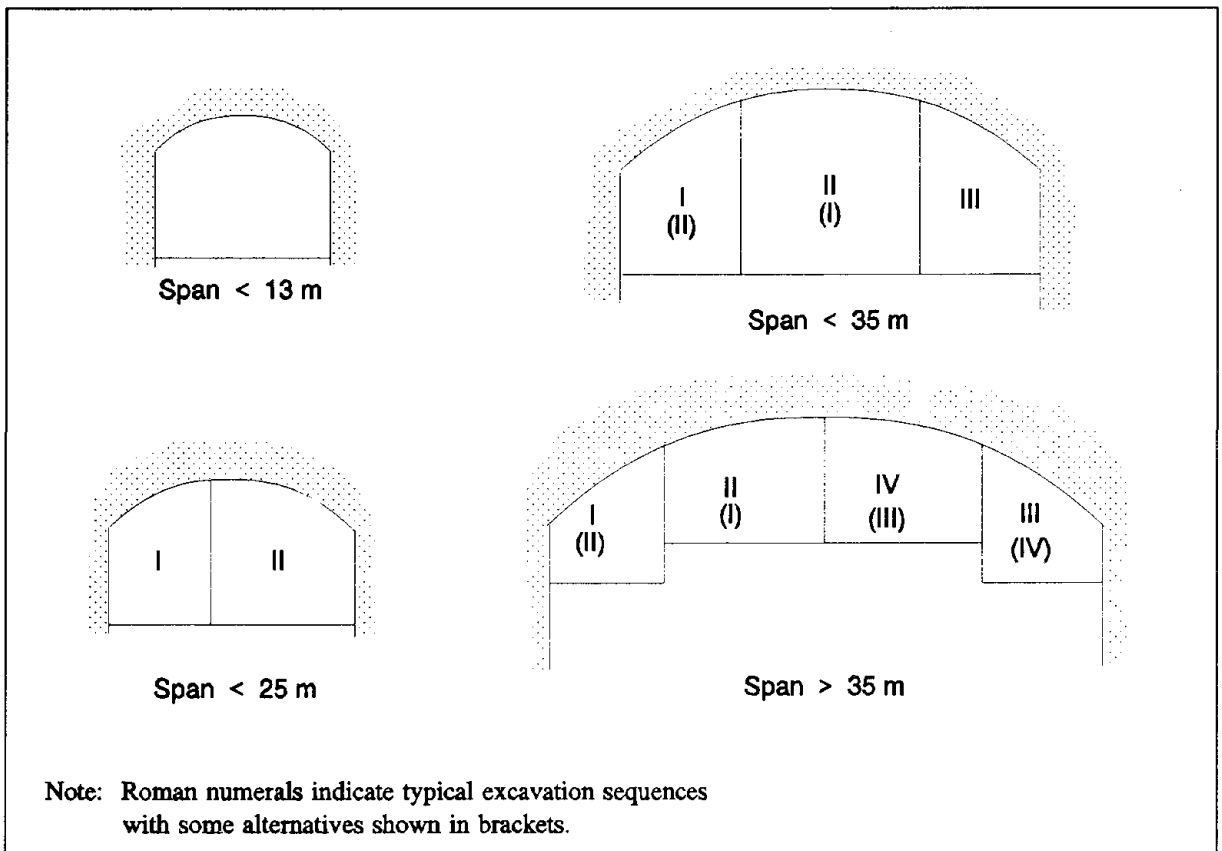


Figure 18 - Excavation Stages for Top Headings in Large Caverns

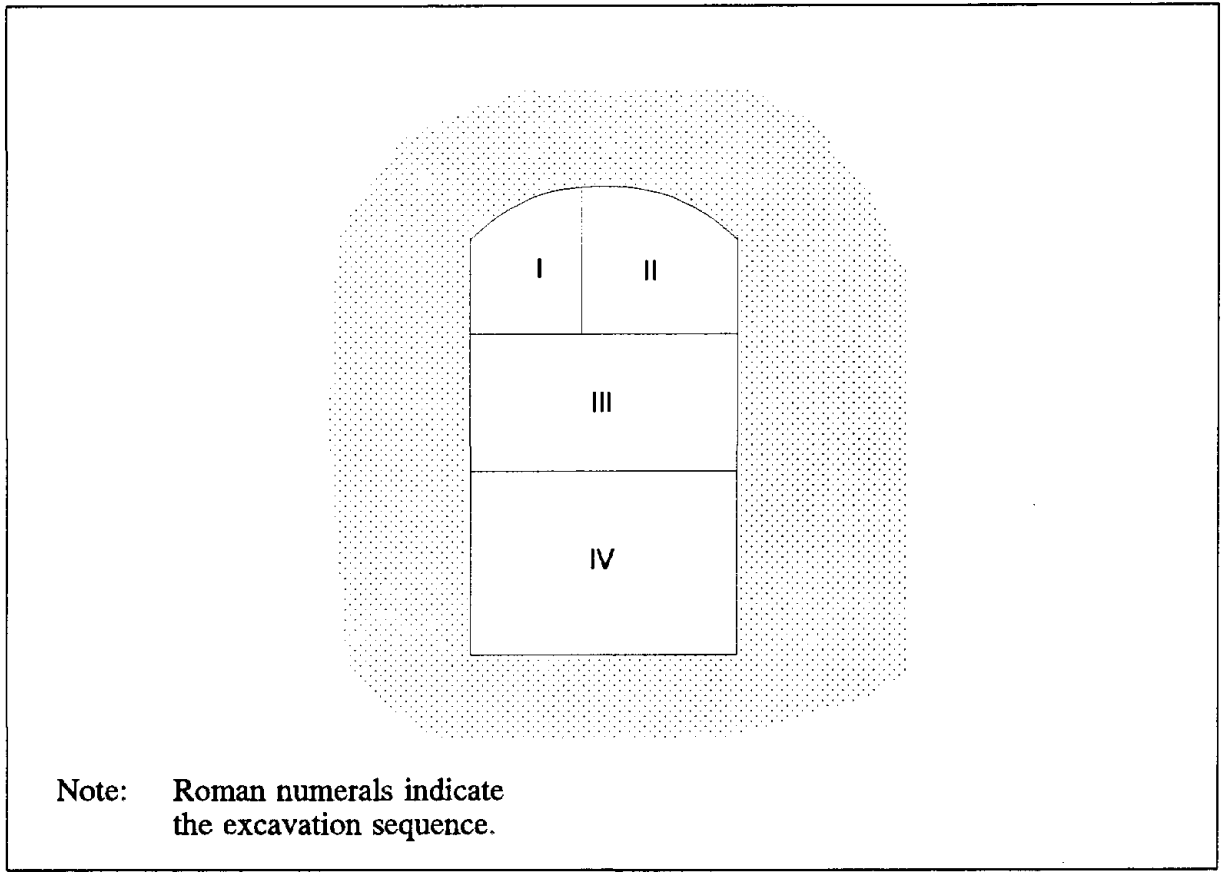


Figure 19 - Typical Excavation Stages for a Large Cavern

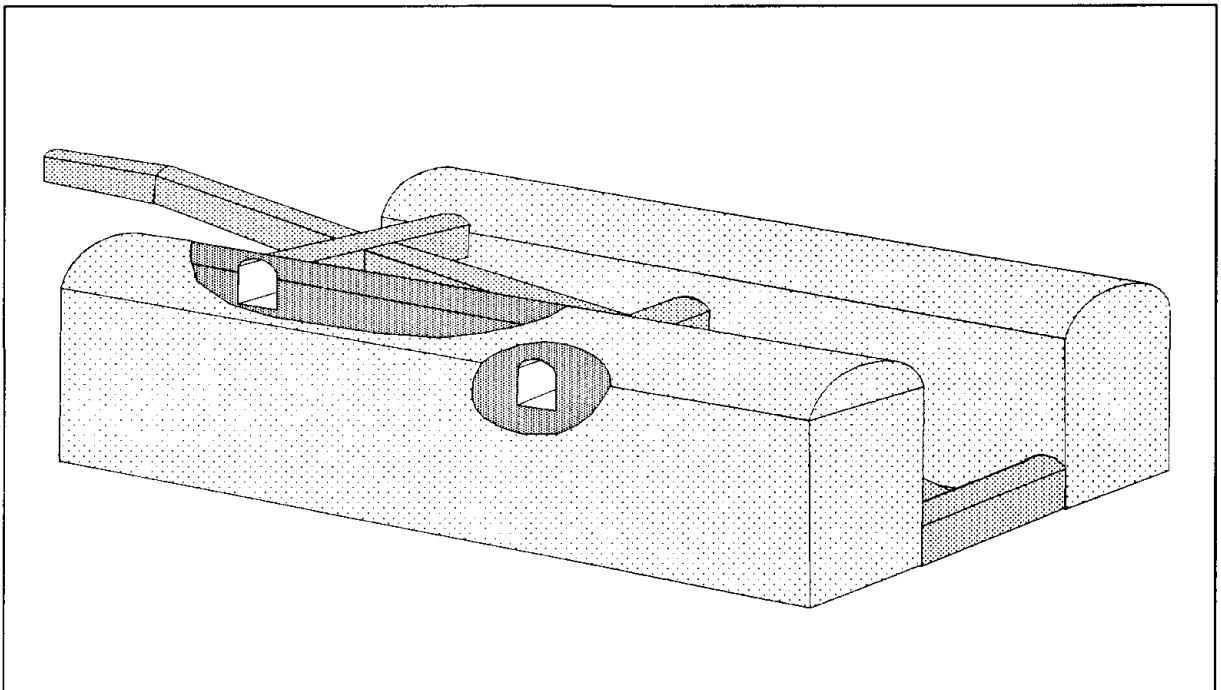


Figure 20 - Access Tunnel for High, Long Caverns

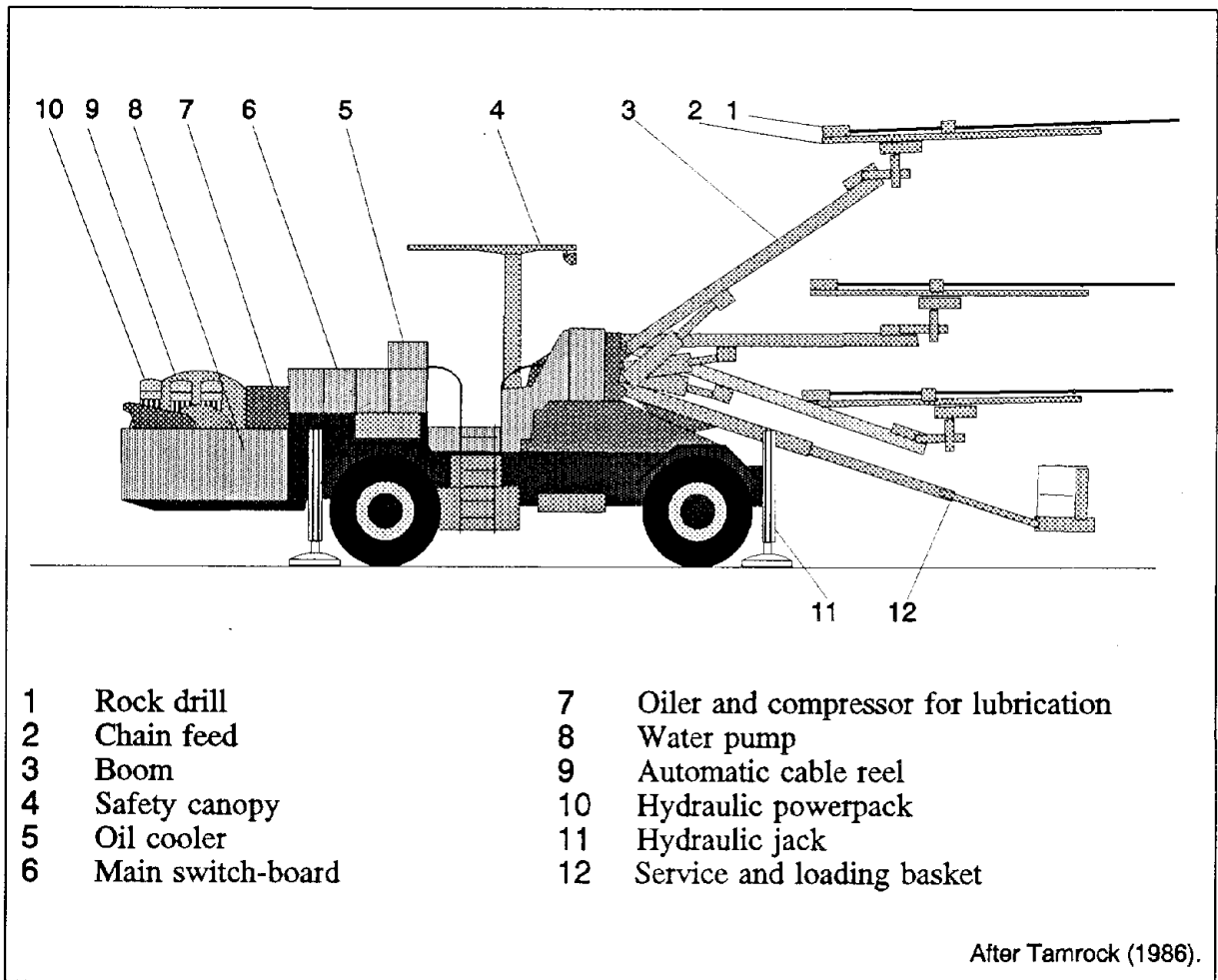


Figure 21 - Typical Modern Drilling Jumbo

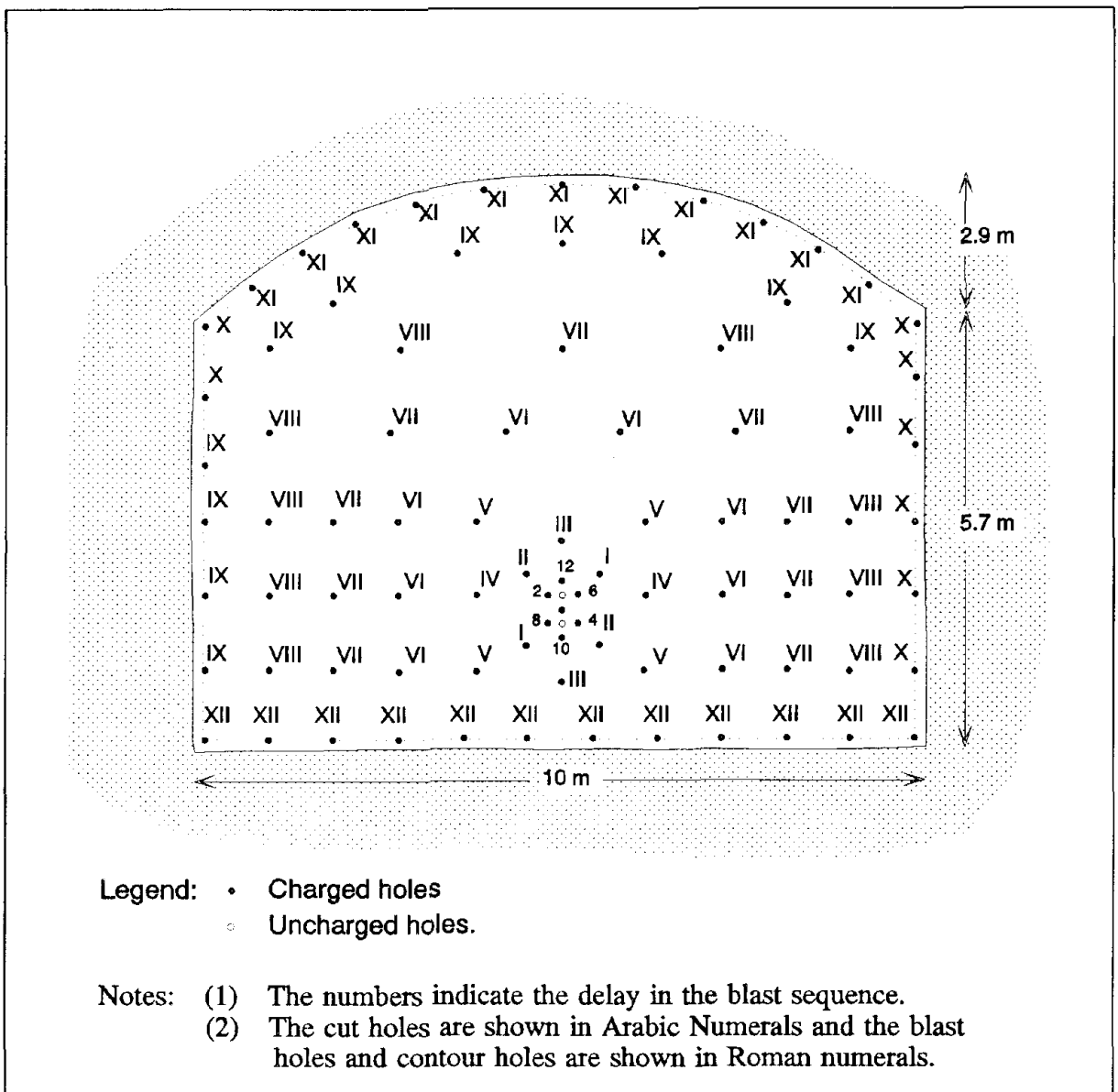


Figure 22 - Typical Drilling Pattern for a Top Heading Blast

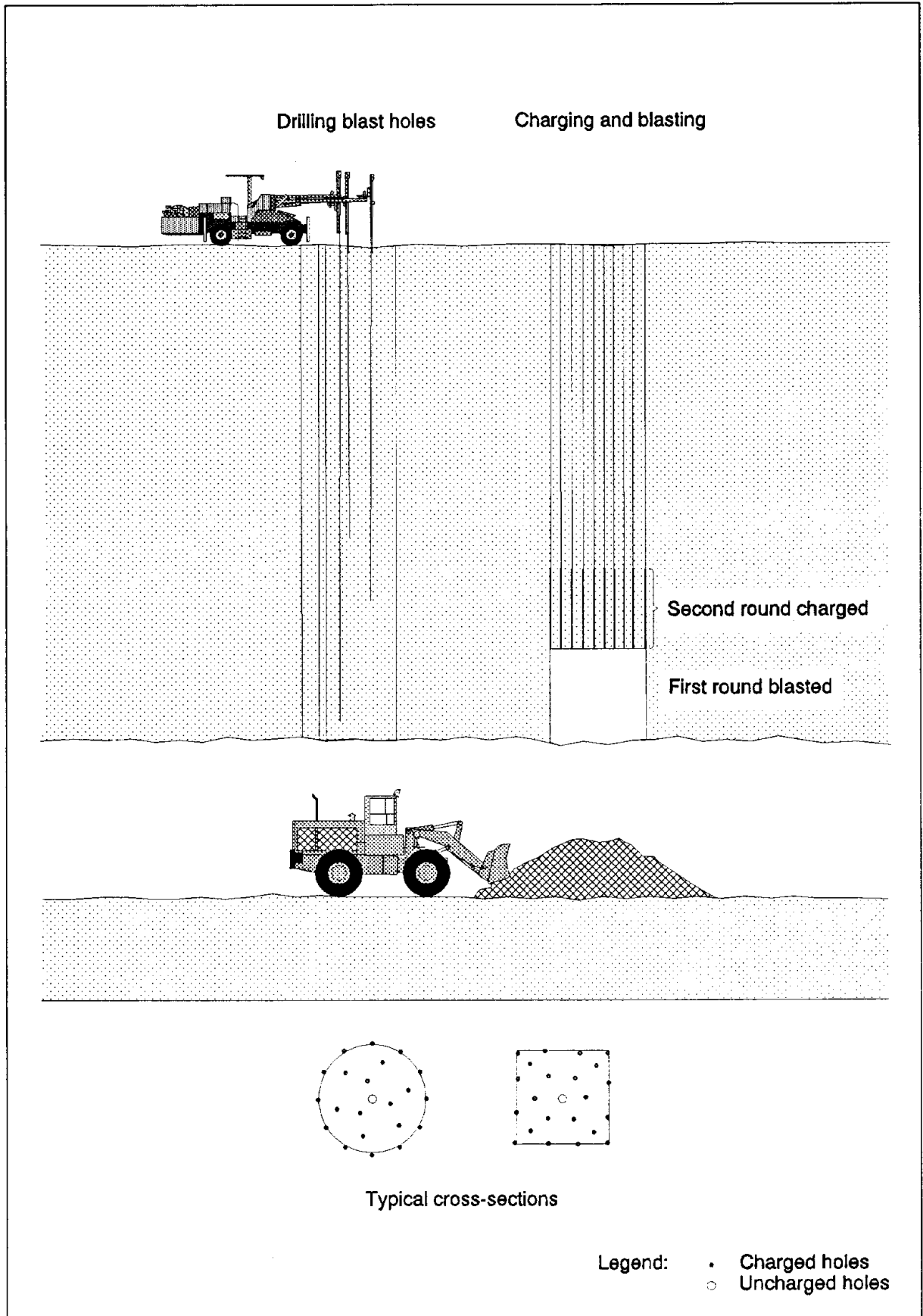


Figure 23 - Shaft Excavation by the Long-hole Method

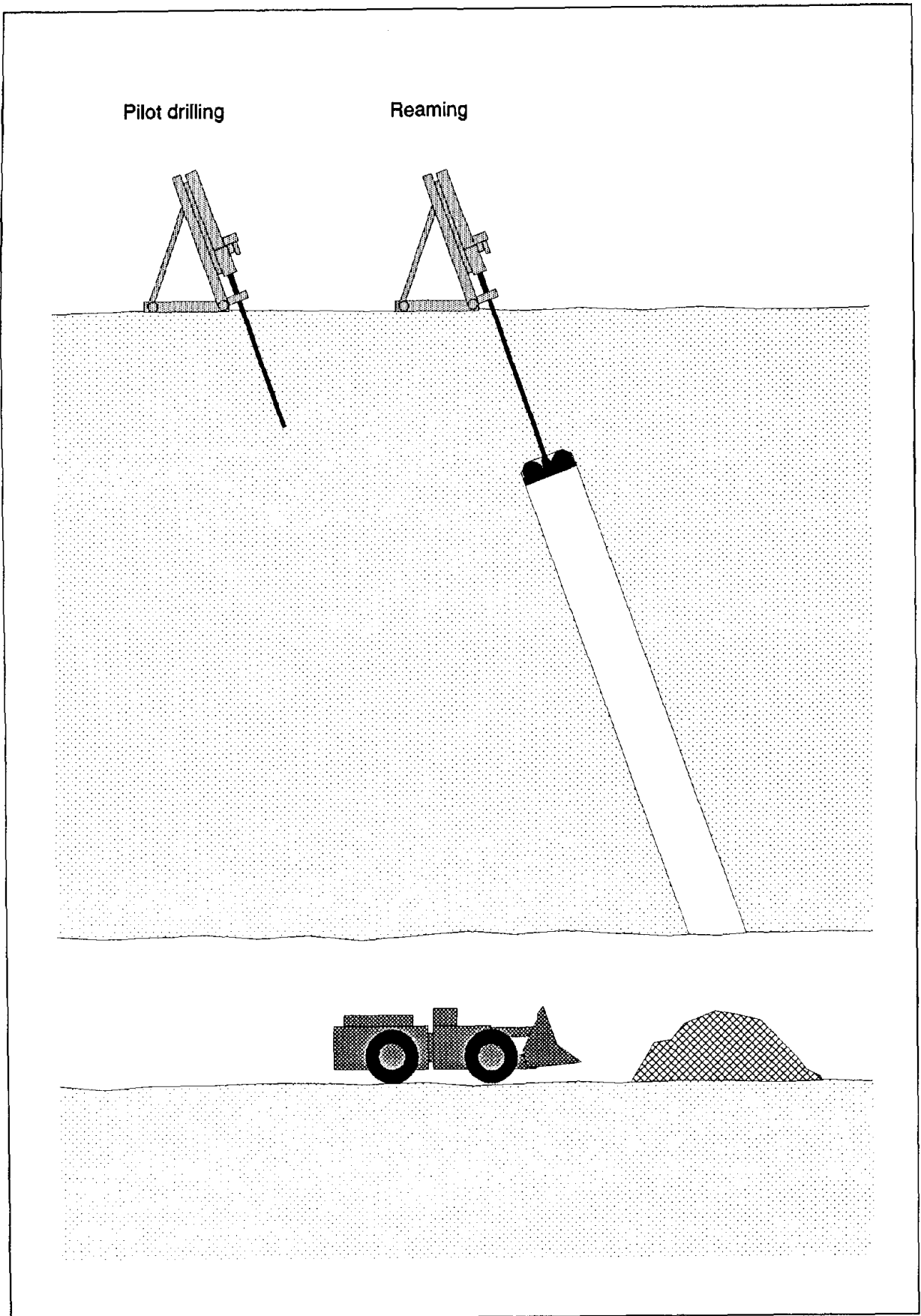


Figure 24 - Shaft Excavation by Raise Boring

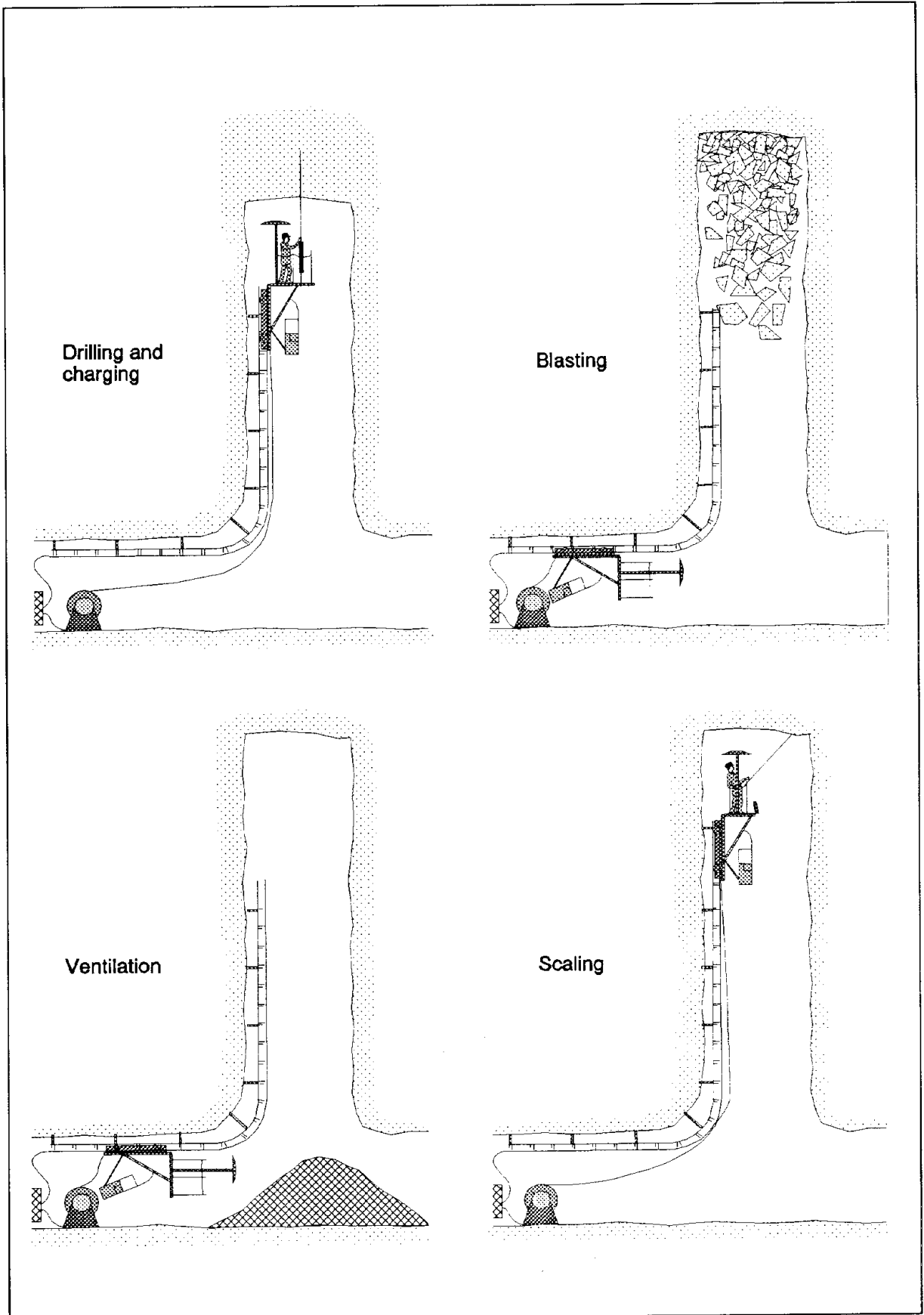


Figure 25 - Alimak Method of Shaft Excavation

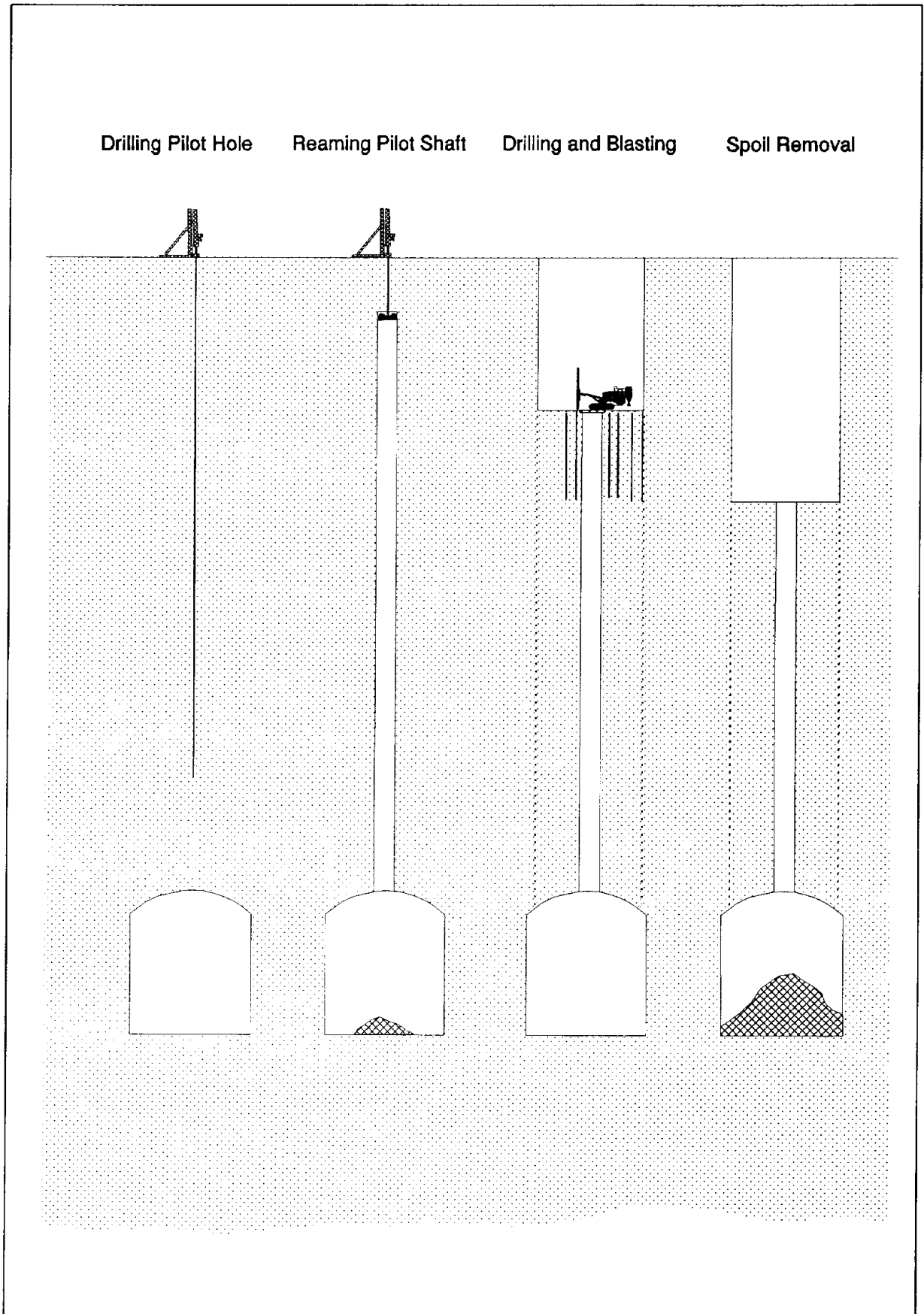


Figure 26 - Pilot Shaft Method of Shaft Excavation

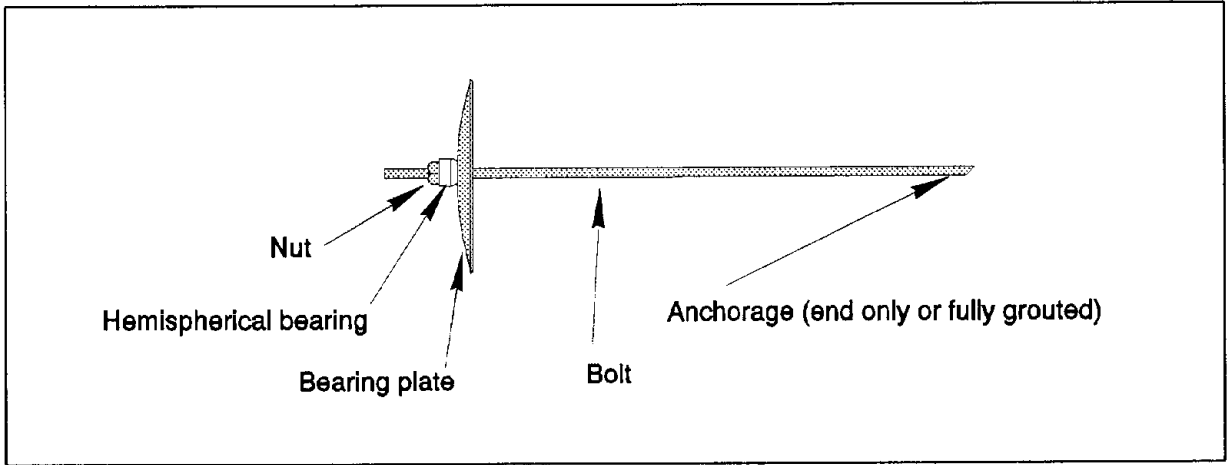


Figure 27 - Main Rock Bolt Elements

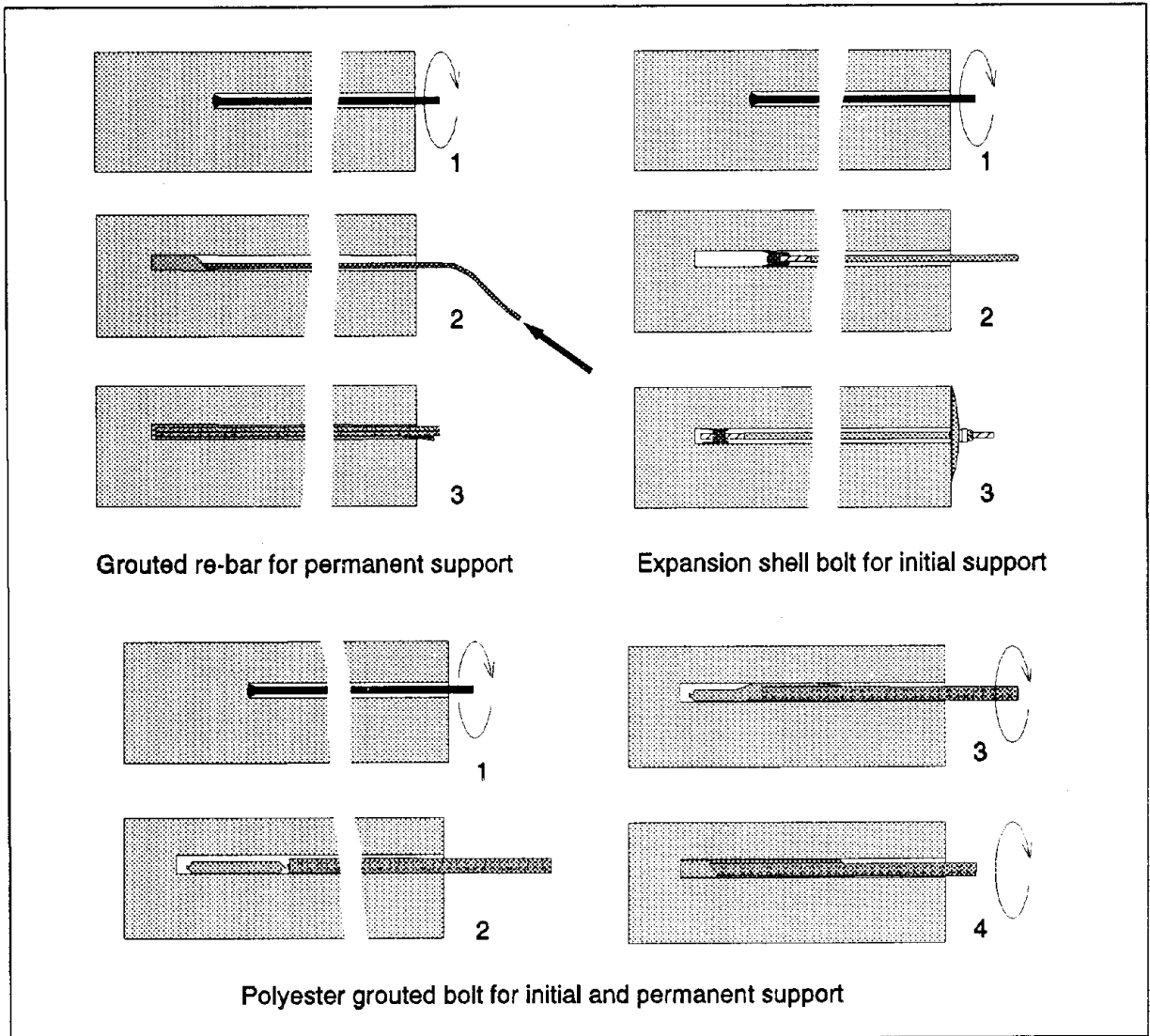


Figure 28 - Principal Types of Rock Bolt

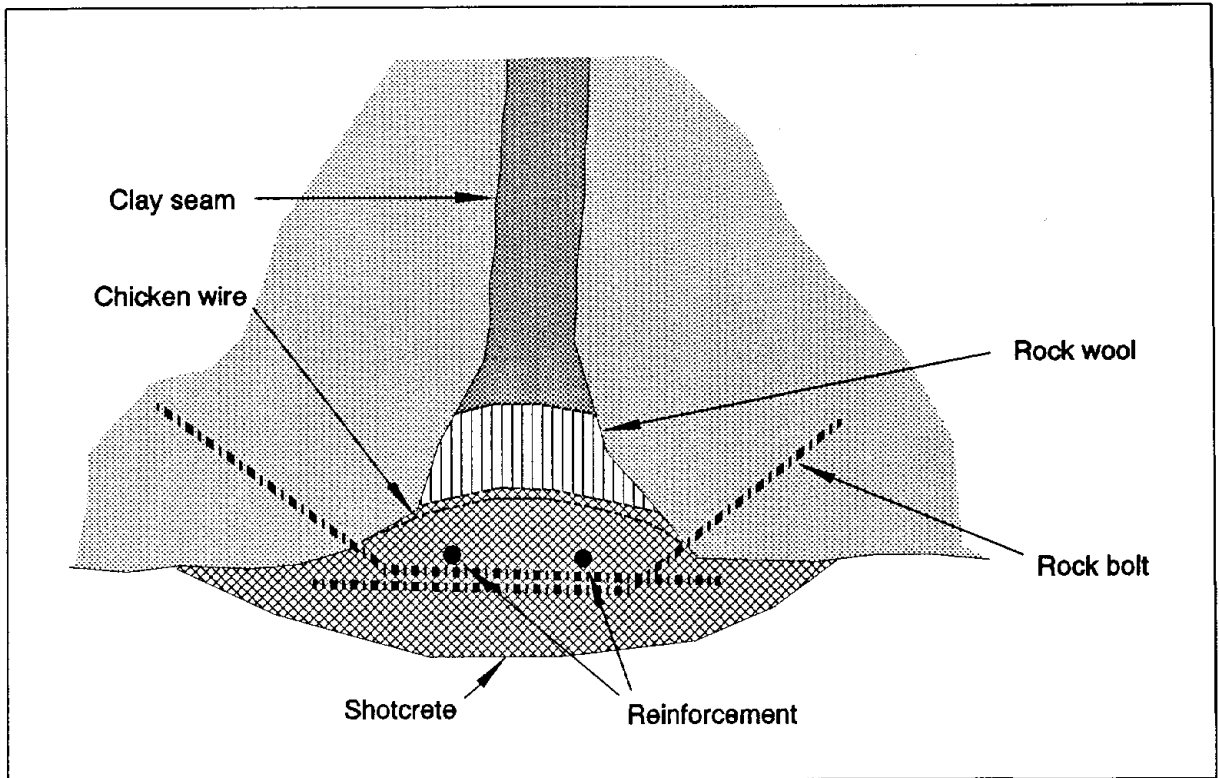


Figure 29 - Pre-treatment of Clayey Seams for Shotcreting

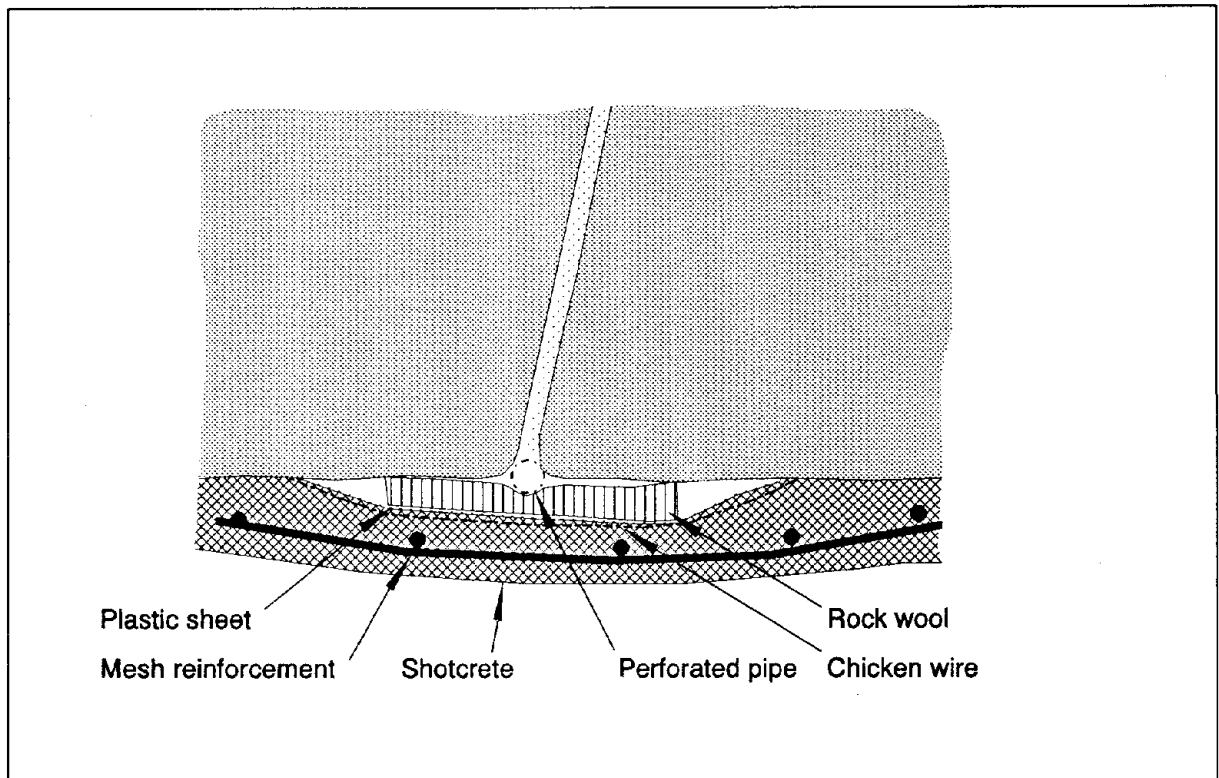


Figure 30 - Drainage Installed Prior to Shotcreting

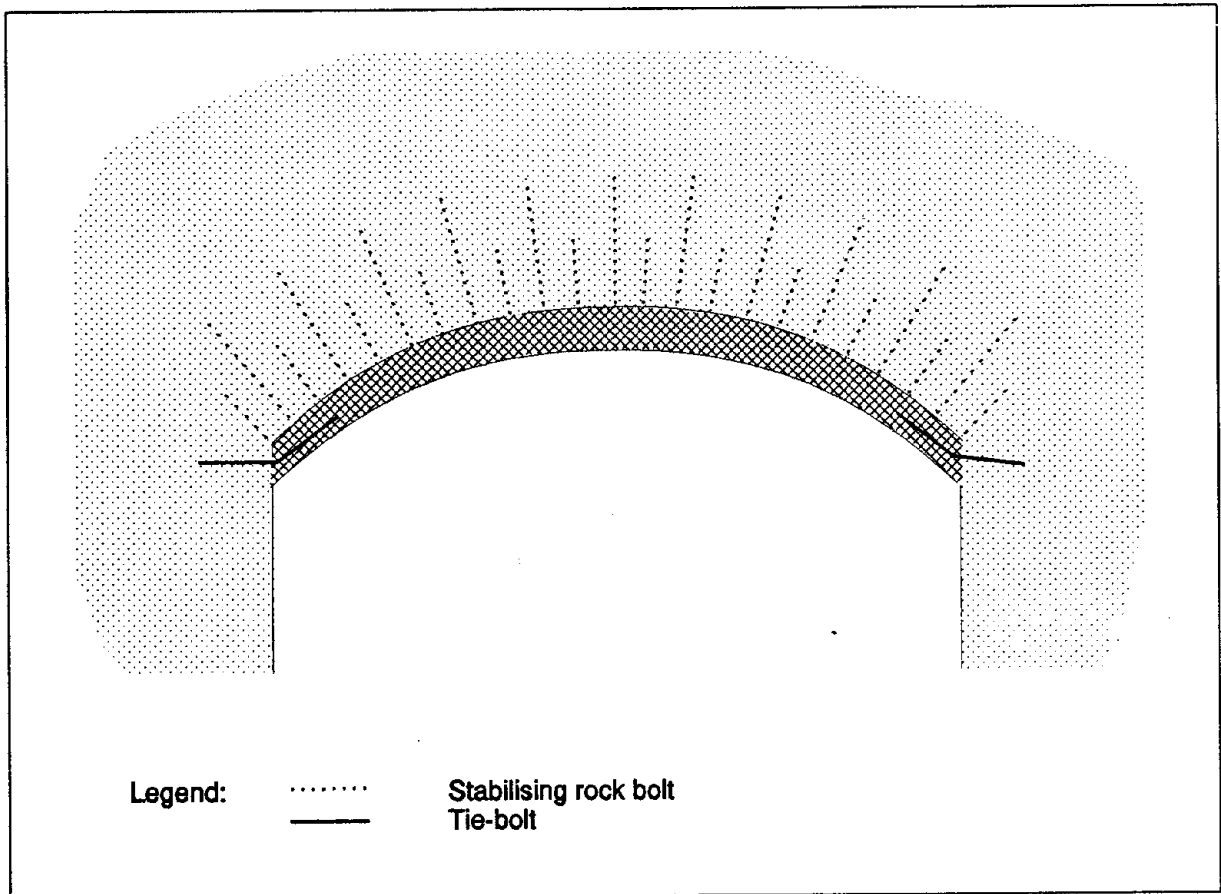


Figure 31 - Concrete Roof Lining

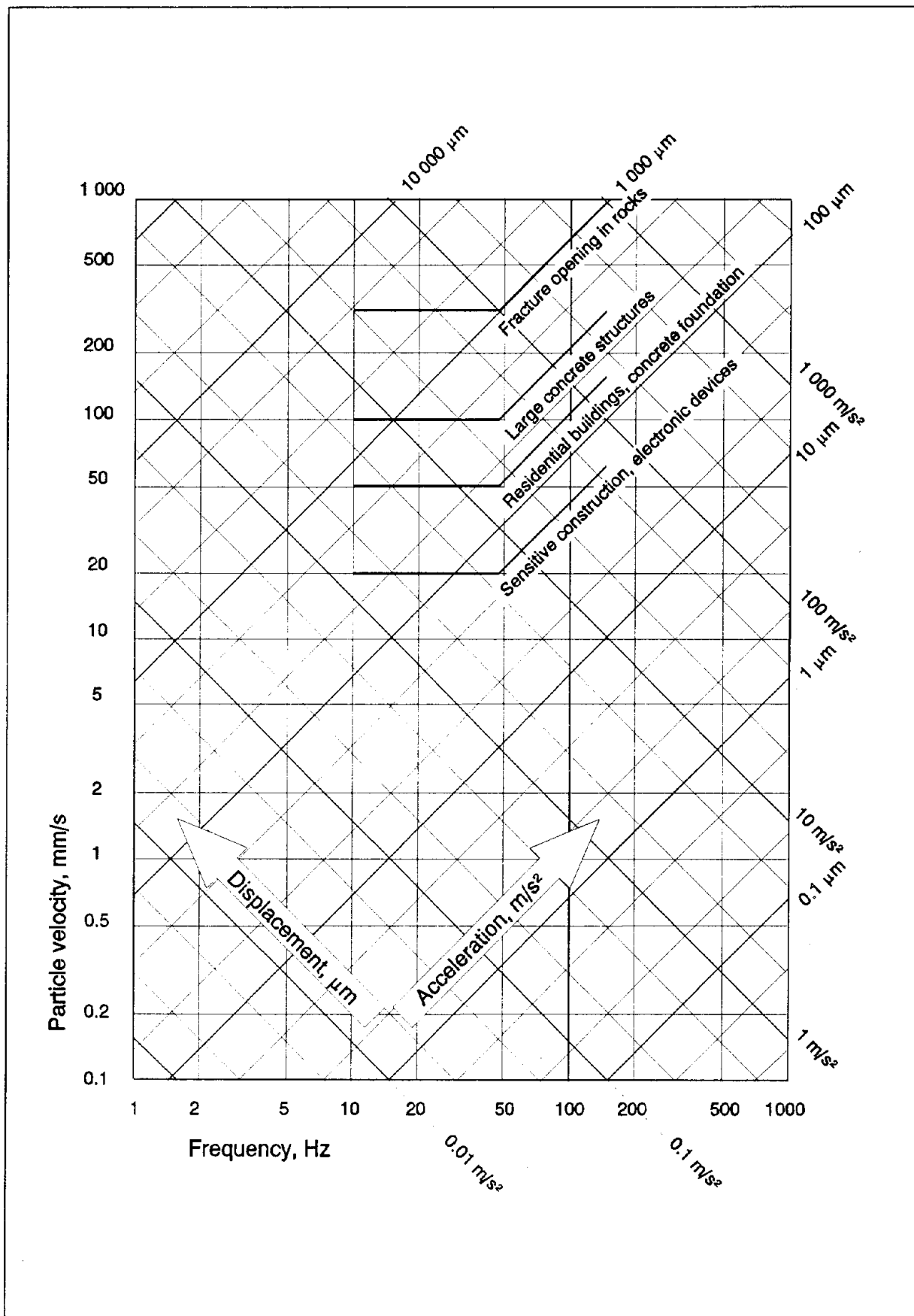


Figure 32 - Typical Safe Vibration Limits

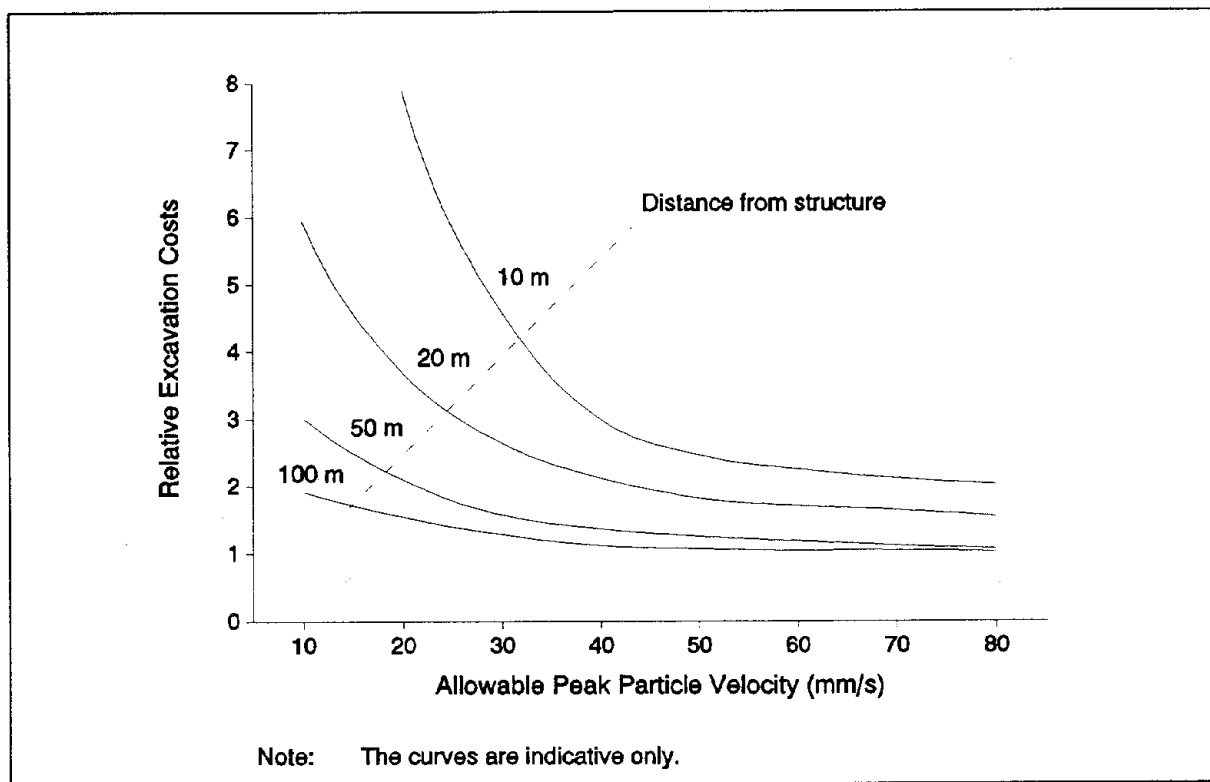


Figure 33 - Cost Related to Vibration Restrictions

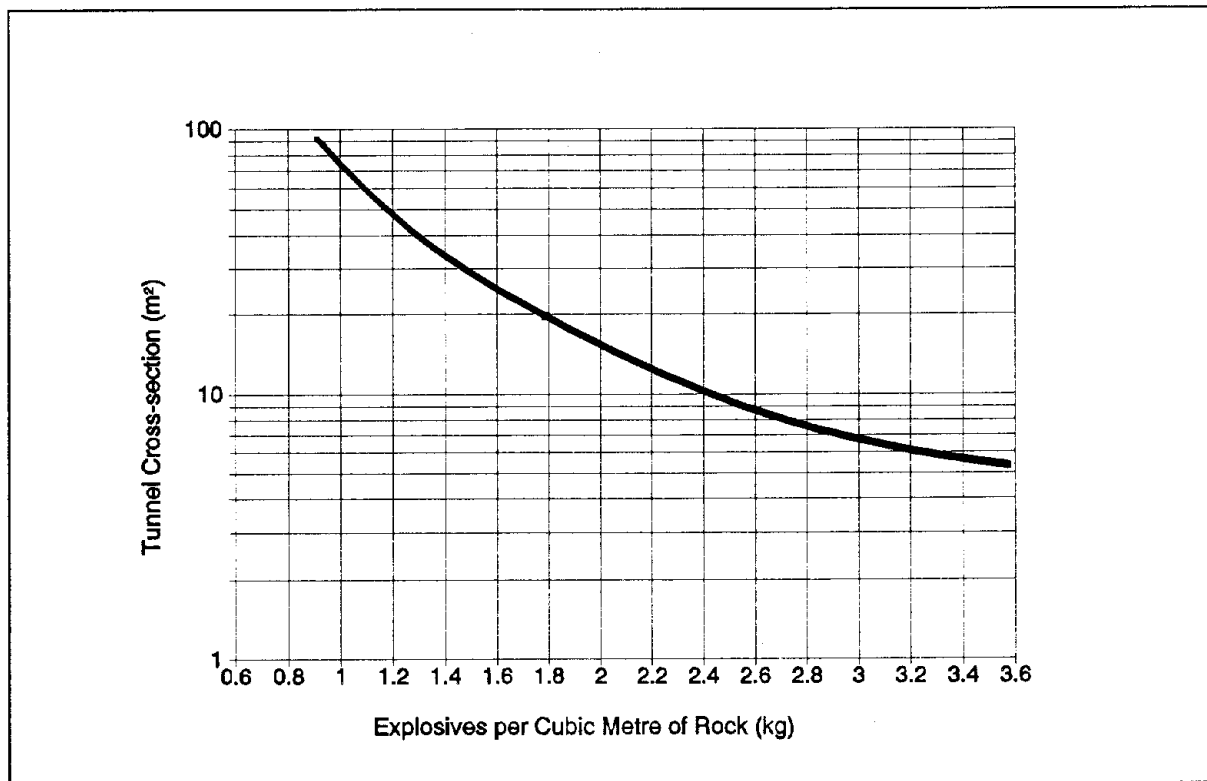


Figure 34 - Specific Explosive Charge versus Cross-sectional Area