Probabilistic Assessment of Engineering Rock Properties in Singapore for Cavern Feasibility

by

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ABSTRACT

Singapore conducted various cavern studies since the 1990s, and has since constructed two caverns. The study done in this thesis concentrates on an area of interest within South-Western Singapore, using four logged boreholes that are each 200 meters in depth. Through the use of the empirical methods, the RQD and *Q*-system, rock support can be estimated for different ground classes for an assumed cavern size. The cost per cubic meter of cavern construction, which includes excavation support using bolts and shotcrete, and grouting, is then estimated. To account for the variability of the ground in the area of interest, probabilistic analyses and assessments of the rock mass parameters derived from the boreholes were carried out. Discrete probabilities were obtained from observed frequencies, and and depth and spatial variability are assessed. Depth Selector Maps (DSM) are created to give planners an indication of the ideal location of a cavern, both in depth and spatially, by providing them with an indication of the variability of the ground so that planners can take the associated uncertainty into consideration when making decisions.

Thesis Supervisor: Herbert H. Einstein
Title: Professor of Civil and Environmental Engineering

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CHAPTER ONE

UNDERGROUND SPACE IN SINGAPORE

1.1 SINGAPORE'S STRATEGY FOR SPACE

The growing population in many cities has resulted in an increased competition for land use. Singapore is a small island state with only 710 km^2 in land area, and has always needed to deal with the problem of land scarcity as a physical constraint. To cope with the competing needs of land as a result of the country's rapid growth and development, besides optimizing the use of its scarce land area through efficient and careful land use planning, Singapore also began reclaiming land from the sea since the 1960s (Koh & Lin, 2006). A map of Singapore showing land increase from 1973 to 2013 is shown in Fig 1.1. However, land reclamation has its problems, with Malaysia voicing her displeasure of Singapore's land reclamation works in Pulau Tekong and Tuas in 2002. Evidently, there is a limit to reclamation, and to deal with the issue of land scarcity, utilizing space underground was the other part of Singapore's strategy for space.

The Urban Redevelopment Authority of Singapore (URA) sees the effective use of underground space as both a strategic resource and a key enabler for cities to meet the many needs of an urban environment (Tng, 2012). In fact, to maximize land space, much of Singapore's infrastructure and utilities, particularly sewer systems has been placed underground since the first tunnel for a first sewage pipeline was constructed in 1983 (Hulme & Burchell, 1999). Since then, Singapore's Mass Rapid Transit (MRT) network has been mostly constructed underground, including the North-South line, the North-East line, Circle Line. New lines like the Downtown Line and the Thomson Line are entirely underground. The rail network, including future underground network expansion is shown in the Land Transport Authority's Transport Master Plan 2013, reproduced in Fig

1.2. Two of Singapore's newest expressways, including the Kallang-Paya Lebar Expressway and the Marina Coastal Expressway have also been constructed underground to free up land space above ground. The Public Utilities Board of Singapore (PUB) also announced in 2009 the construction of a Deep Tunnel Sewerage System (DTSS) 20 to 55 meters below ground, as a cost-effective and sustainable solution to meet Singapore's long term needs (Public Utilities Board, 2009), shown in Fig 1.3. Following that, in 2012, Singapore Power announced a $2 billion project to build two cross-island transmission cable tunnels, running 35km, 60m underground (Foo, 2012), shown in Fig 1.4.

Another more expensive option to increase the supply of land is to create large spatial underground space in rocks deeper into the ground. Currently, there are two caverns in Singapore - the Underground Ammunition Facility (UAF), and the recently opened Jurong Rock Caverns (JRC), South-east Asia's first commercial underground liquid hydrocarbon storage facility. The graphical depiction of the use of underground space in Singapore shown in Fig 1.5 illustrates how the different uses come together underground, to free up scarce land space above ground.

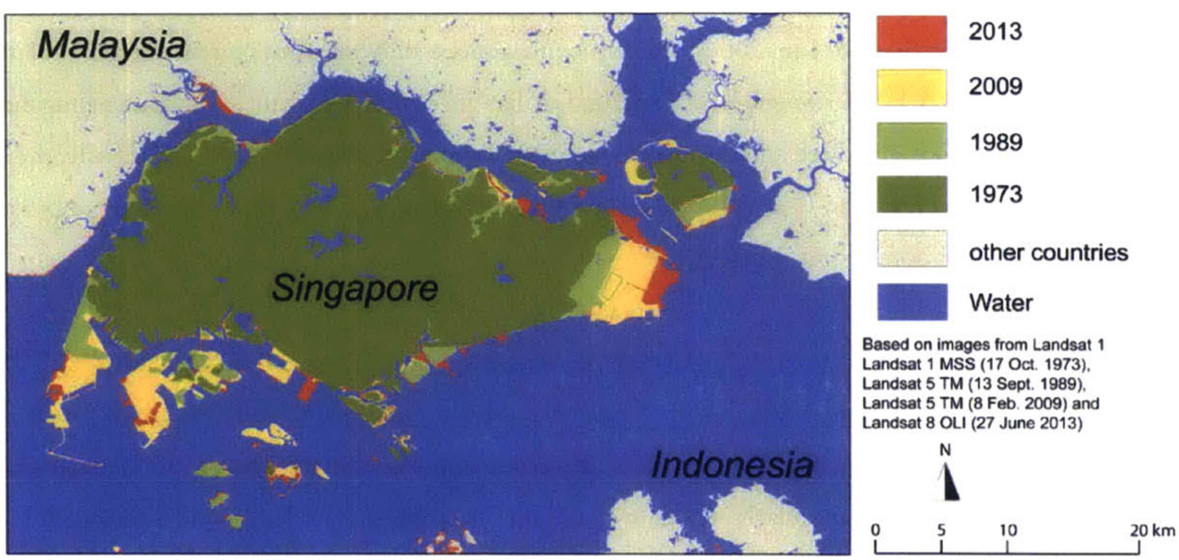


Fig 1.1: Map of Singapore showing land area increase from 1973 to 2013 (reproduced from Peduzzi, 2014)

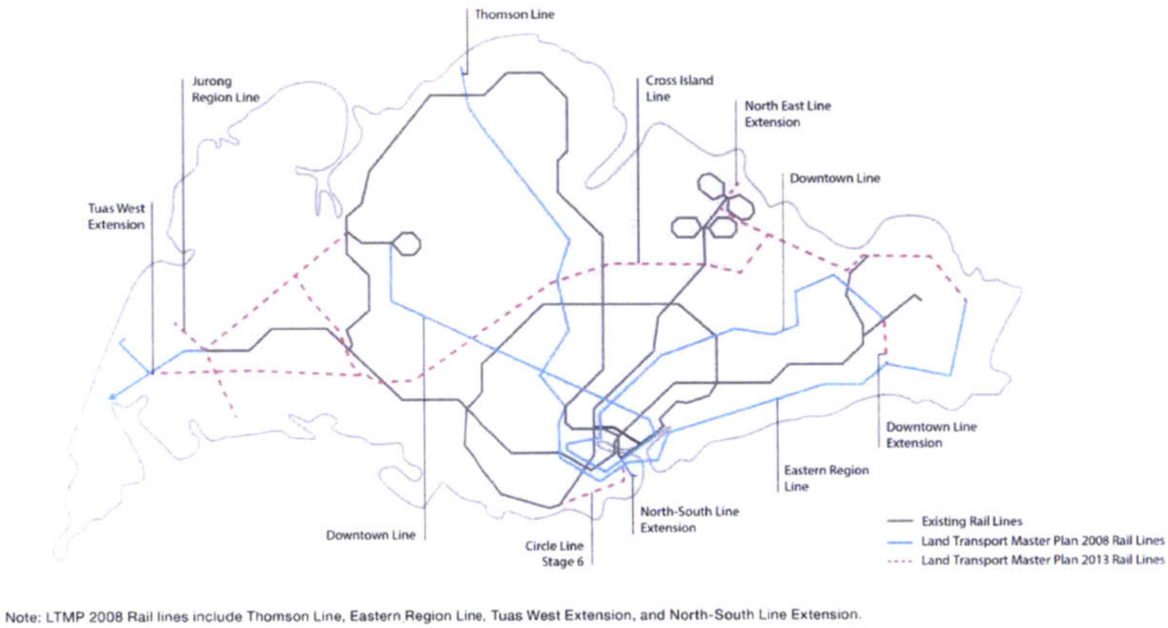


Fig 1.2: Rail network in Singapore, with plans for future rails underground beyond Downtown Line and Thomson Line (source: Land Transport Authority Singapore, Land Transport Masterplan 2013)

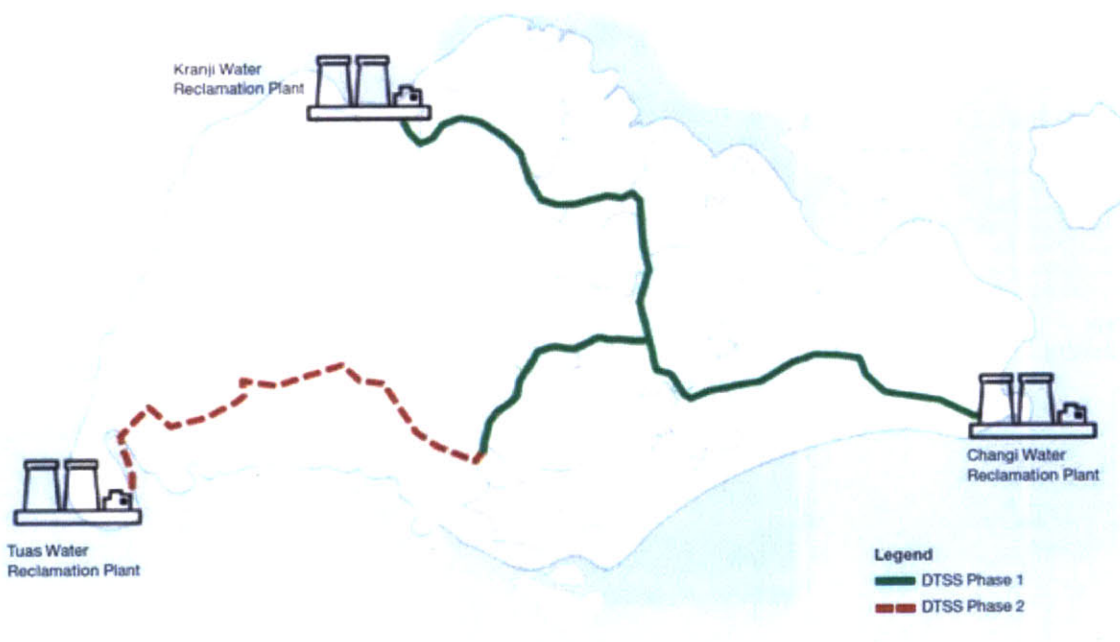


Fig 1.3: Deep Tunnel Sewerage System in Singapore (source: Public Utilities Board Singapore)

Fig 1.4: North-South and East-West Cable Tunnels in Singapore

(source: Singapore Power, shown in google maps)

Fig 1.5: Graphical depiction of use of underground space in Singapore

(modified and reproduced from Tng, 2012)

1.2 DEVELOPMENT OF ROCK CAVERNS IN SINGAPORE

As competing land use drives up the opportunity cost of land in Singapore, deep rock caverns will become increasingly attractive. The UAF and the JRC are showcases of the economic value and the feasibility of large scale underground developments in Singapore. Many studies have also been carried out to identify potential uses of underground caverns in Singapore. These studies, press releases on opportunities (Wong, 2015; Kok, 2013) and legislative changes indicating future planning and development of underground space in Singapore (Ministry of Law, 2015), suggest that the UAF and the JRC are unlikely to be the remain as the only two underground rock caverns in Singapore in the near future.

1.2.1 HISTORY OF ROCK CAVERNS

The idea of the use of rock caverns deep in the ground is not new. This is most evident from the fact that many underground cities, semi-underground settlements that are at least 1500 years old, exist in the Cappadocia Region of Central Turkey. One of those, the Derinkuyu Underground City, is an ancient multi-level underground city with 13 floors extending to a depth of approximately 85m, and large enough to accommodate 30,000 people (Aydan & Ulusay, 2012).

History of mining goes back more than several thousand years, and rock caverns have been used substantially for a wide range of functions. Nearly all hydropower stations in Norway in the past 40 to 50 years are built underground, and large rock caverns have been frequently used to store oil, water and food. More recently, Norway started constructing underground openings for public use - the most famous of which, is the Gjøvik Olympic Mountain Hall (Broch, Myrvang, & Stjern, 1996).

1.2.2 STUDIES ON UNDERGROUND SPACE

Activities related to rock cavern development in Singapore began in 1990, way before the UAF started construction in 1999, and the JRC began construction in 2009. These activities are summarized by Lui, Zhao and Zhou (2013), and reproduced in Table 1.1.

Table 1.1: Major activities on rock cavern development in Singapore since 1990 (Reproduced from Lui, Zhao, & Zhou, 2013)

Period	Major Activities and Development
1990-1994	Feasibility study of rock cavern construction in the Bukit Timah Granite by Public Works Department (PWD)/ Nanyang Technological University (NTU).
1995-1998	Feasibility study of rock cavern construction in the Jurong Formation by NTU/ PWD. First Tasks Force on promoting use of rock cavern was set up and led by URA, and the Tasks Force recommended Ministry of Defence (MINDEF) to take the lead. Feasibility study of the UAF by MINDEF/Defence Science and Technology Agency (DSTA). Establishment of Underground Technology and Rock Engineering (UTRE) program at NTU supported by DSTA.
1997-2000	Feasibility study of the Underground Science City (USC) by NTU/Jurong Town Corporation (JTC). Construction of the UAF started in 1999 by MINDEF/DSTA.
2001-2007	Feasibility studies of hydrocarbon storage caverns at the Jurong Island (JRC) by JTC and NTU. Other preliminary feasibility studies of underground space using rock caverns, e.g., Science Centre below Mount Faber, Jurong Bird Park extension into the Jurong Hill.
2007-2012	JRC construction started in 2009 and the Government set up inter-agency Underground Master Planning Task Force (UMPTF). Further feasibility study on the USC at Kent Ridge commissioned by JTC. Feasibility study on underground warehouse caverns at Tanjong Kling by JTC. Feasibility studies of several industrial usages of rock caverns by JTC/Ministry of National Development (MND). Nanyang Centre of Underground Space (NCUS) established at NTU in 2012. Underground space master planning study of the NTU campus. MND research and development call on Sustainable Urban Living.

The list of activities demonstrates that underground caverns is on the mind of planners since the 1990s, and it is instructive to briefly highlight a few of these studies to understand the extent of such activities.

1.2.2.1 Rock Caverns in Bukit Timah Granite

One of these early studies on the construction and utilization of rock caverns in Singapore was a comprehensive four-part study. The first part covered the Bukit Timah Bedrock resource that lies below central and northern Singapore, which is shown in the generalized geological map of Singapore, Fig 2.2, in the next Chapter. It described the geological investigations carried out and concluded the Bukit Timah granite is a valuable resource for cavern and underground space development in Singapore (Zhao, 1996). In the second part of the study, the potential uses, cost and benefits of underground caverns were discussed, including shelter and recreation, water collection storage and treatment, sewage treatment, oil and gas storage and power station (Zhao, Choa, & Broms, 1996). The costs of a crude oil storage and a warehouse scheme were also closely examined.

The third part of the study moves on to examine planning of the underground space for economic, social and environmental considerations, recognizing that constructing underground, as attractive as it is, is not unlimited (Zhao & Lee, 1996). Finally, in the fourth part of the study, two proposed cavern schemes for crude oil storage and warehouse-shelter proposals were considered to illustrate possibilities of putting in such caverns underground (Zhao & Bergh-Christensen, 1996).

1.2.2.2 Underground Science City below Kent Ridge Park

At the 13th World Conference of Associated Research Centres for Urban Underground Space in 2012, some findings of a three-year feasibility study on an Underground Science City (USC) with 40 linked rock caverns for research and development facilities and data centres was presented (Feng, 2012). The location of the USC is in Kent Ridge, shown in Fig 1.2. The study explored the geology of the site, orientation, rock mass quality and cavern geometry, and using different design methods to determine the temporary support. This showed that the construction of the USC is technically feasible (Schmid et.al., 2013).

From Table 1, it can be seen that the feasibility of the USC was first explored by NTU in the period of 1997 to 2000. The detailed NTU report covers engineering geology and feasibility, underground master planning and preliminary designs (Zhao, Cai, & Hefny, 2001). Even though the idea of the USC was not picked up at that point of the study, the recent government commissioned study by Schmid et.al. (2013) indicates the possibility of the USC's construction in the forseeable future, and is further indication that the Singapore government is ready to go into underground rock caverns in a big way.

1.2.3 THE UNDERGROUND AMMUNITION FACILITY

Besides feasibility studies, two underground caverns have been constructed and are functioning under the hustle and bustle of the city-state of Singapore. The first of which is the Underground Ammunition Facility (UAF), which started construction in 1999, and was commissioned in 2008. Taking more than 10 years to complete, the UAF was constructed in granite formation, and freed up 300 hectares of land above ground for other use (Wan, 2015). The location of the UAF is not disclosed because of military sensitivity, but is known to be in Bukit Timah Granite, shown in Fig 2.2, which is the same region Zhao (1996) conducted his study mentioned earlier.

Storing ammunition underground is an ideal function of a facility with huge benefits located underground. Locating the UAF underground requires 90 per cent less land to be restricted compared to locating it above ground, in order to ensure a safe distance to public access areas. Furthermore, locating ammunition storage underground reduce the associated risks because of the natural resistance of the granite.

1.2.4 THE JURONG ROCK CAVERNS

The first phase of the Jurong Rock Caverns (JRC), was officially opened by Lee Hsien Loong, the Prime Minister of Singapore on 2 September 2014. Planning for the JRC began in 2001, and construction of the first phase started in 2009. The opened first phase consists of five caverns at 27m high, 20m wide and 340m long, and is the first underground oil storage facility in South-East Asia. It frees up 60 hectares of usable land above ground. Two of the caverns are currently operational, with the other three expected to be operational by 2016. (Chia, 2014)

The total excavated rock volume for Phase I of the JRC is 3.8 million cubic meters, with 2 levels of excavation at 100m and 135m depth below ground level, costing S$950 million. Despite having extensive site investigation works both before and during construction, serious water inflow was experienced in certain areas during construction (Winn & Ng, 2013). The cost of the construction also exceeds the originally estimated amount of S$700 million by S$250 million (Cheam, 2009). The JRC is constructed in the Jurong Formation.

The locations of Jurong Island, where JRC is located, and Kent Ridge, where USC will be located, are shown on Fig 1.6.

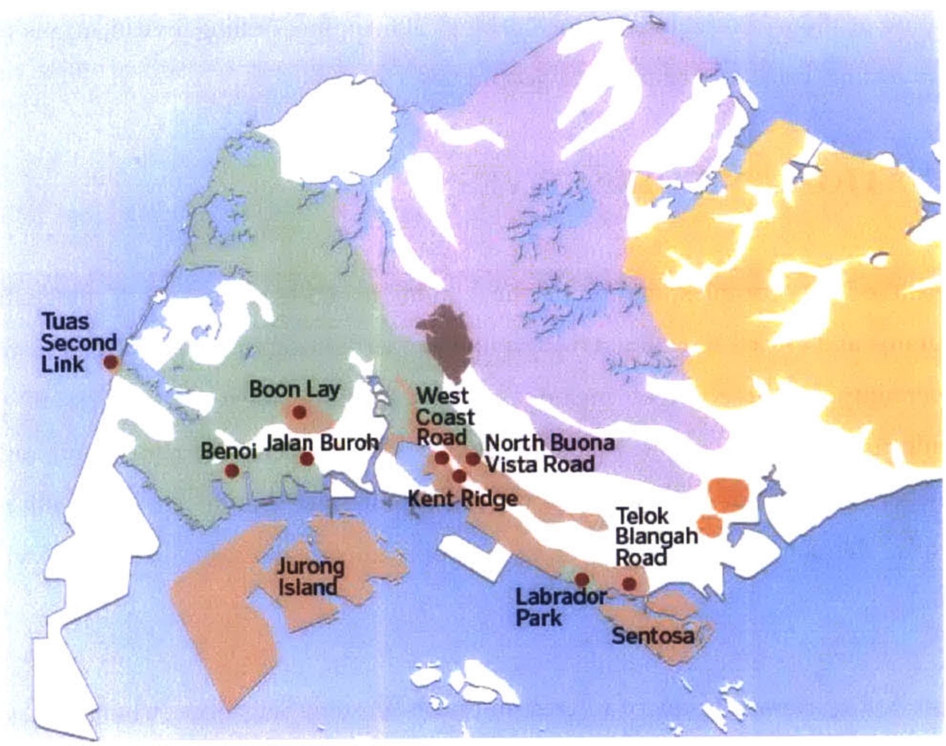


Fig 1.6: Locations of Jurong Island (JRC location), and Kent Ridge (USC location)
(modified and reproduced from *The Straits Times, 2014*)

1.3 THE SINGAPORE GEOLOGICAL OFFICE

For any form of geotechnical engineering design below the surface of the ground, knowledge of the geologic conditions and, subsequently the measurement and determination of geotechnical parameters required for design, are important stages to achieve adequate and successful design. Hence, the Singapore Geological Office (SGO) was set up in 2010 under the Building and Construction Authority of Singapore (BCA) to facilitate underground developments by creating a database on the country's geology.

In addition, in view of plans for cavern developments in future, SGO has also carried out extensive geological surveying and investigation work in the South-Western and North-Eastern parts of Singapore. Some of the works include surface geological mapping, drilling investigations to 200m depth with in-situ testing, rock core testing and geophysical surveys.

1.4 MOTIVATION AND OBJECTIVES

As a result of the complex nature of the conditions underground, it is inevitable that exploration programs and efforts to characterize ground properties cannot eliminate all uncertainties. Sources of uncertainty in these stages include spatial variability, measurement errors (sample disturbance, random errors, bias errors, statistical fluctuation), model uncertainty, and omissions (Einstein & Baecher, 1983). For cavern design in rocks, these uncertainties are especially relevant because, frequently, the actual geological condition is only known during construction when the ground is exposed.

The thesis will work on an area of interest in South-Western Singapore, where the SGO has placed four boreholes for site investigation. These include three 200m vertical boreholes and a 200m inclined borehole, with dip direction/dip of 65/70. The area of interest will be described in detail in Chapter Five.

To reduce the risk associated with the uncertainties from the site investigation, this thesis will probabilistically analyze and assess the rock mass parameters obtained from SGO's investigation work and draw relations between these rock mass parameters to the rock support design, as well as the implication on cost. The thesis aims to translate the analysis to simple decision maps to allow planners to decide on the location and depth of possible future caverns.

1.5 OUTLINE OF THESIS

Chapter One, the current chapter of interest, introduces the background to the development of underground space in Singapore, and describes various cavern studies and cavern constructions that have been done in Singapore. The chapter also highlights the work that the SGO, set up in 2010, has done in view of plans for cavern development in future. The author also defines the scope of the project and lists the objectives for the paper.

In Chapter Two, the geology of Singapore is described. Sources that have discussed the geology of Singapore are summarized, and provide an understanding of the geology and properties of the rock mass in the area of interest.

Chapter Three describes some of the commonly methods used for designing rock support for caverns, and highlights the methods – RQD and *Q*-system - that will be used in determining the rock support for caverns. General limitations of these methods will also be discussed.

Chapter Four discusses the use of estimated cost for rock support as an indicator for cavern feasibility and associated uncertainty. Sources of rock support cost are compared to give an indication of the expected cost for required rock supports for different ground classes determined from the RQD and *Q*-system method.

In Chapter Five, the area of interest is described and data from the boreholes of the BCA site investigation in an area in South-Western Singapore are analyzed and assessed probabilistically. The expected RQD and Q-value for different depths are obtained and related to the required rock support and associated cost. A decision map, the Depth Selector Map, will be created from the probabilistic assessment, to aid planners in the determination of location and depth of caverns. Spatial variation of RQD in the area of interest will also be probabilistically assessed.

Finally, Chapter Six presents the summary of the findings of the thesis, and highlights the limitations associated with the decision tools. Following this, the Chapter will discuss the applicability of the tools proposed, including extension of the method to different areas of interest and explore further work that can be done to improve the results for decision making.

CHAPTER TWO

SINGAPORE GEOLOGY

2.1 REGIONAL GEOLOGY OF THE MALAY PENINSULAR

Singapore lies at the southern tip of Peninsular Malaysia, close to the southern extremity of a southerly projection of the Eurasian tectonic plate. As described by Pitts (1984), Singapore is "just north of, but generally now away from the influence of the Java Trench, which is part of the northerly termination of the Indian Plate", and "in the geologic past, Singapore has come under the influence of the developing Indonesian Island Arc". Also, Pitts (1984) highlighted that the northerly-dipping subduction zone is migration south-wards, away from Singapore.

Fig 2.1 shows a geological subdivision of the Malaysian Peninsula and Singapore into Western, Central, and Eastern Belts. The Bukit Timah Fault in Singapore marks the boundary between Eastern and Central Belts, and is shown in the generalized geological map of Singapore in Fig 2.2. At this boundary, 220 to 200 million years old late Triassic Jurong Formation continental red beds and shallow marine conglomerates from the Central Belt, sandstones and limestone lie above 244 to 230 million year old granite from the Eastern Belt. (Hutchison & Tan, 2009) Fig 2.3 shows a paleogeographic reconstruction of Central Singapore in red-bed Upper Triassic Times.

2.2 GEOLOGICAL DESCRIPTIONS OF FORMATIONS

An overview of the main geological formations and the essential geological and geotechnical information are essential to the understanding of the implications the geology of Singapore has to the design and construction of underground caverns.

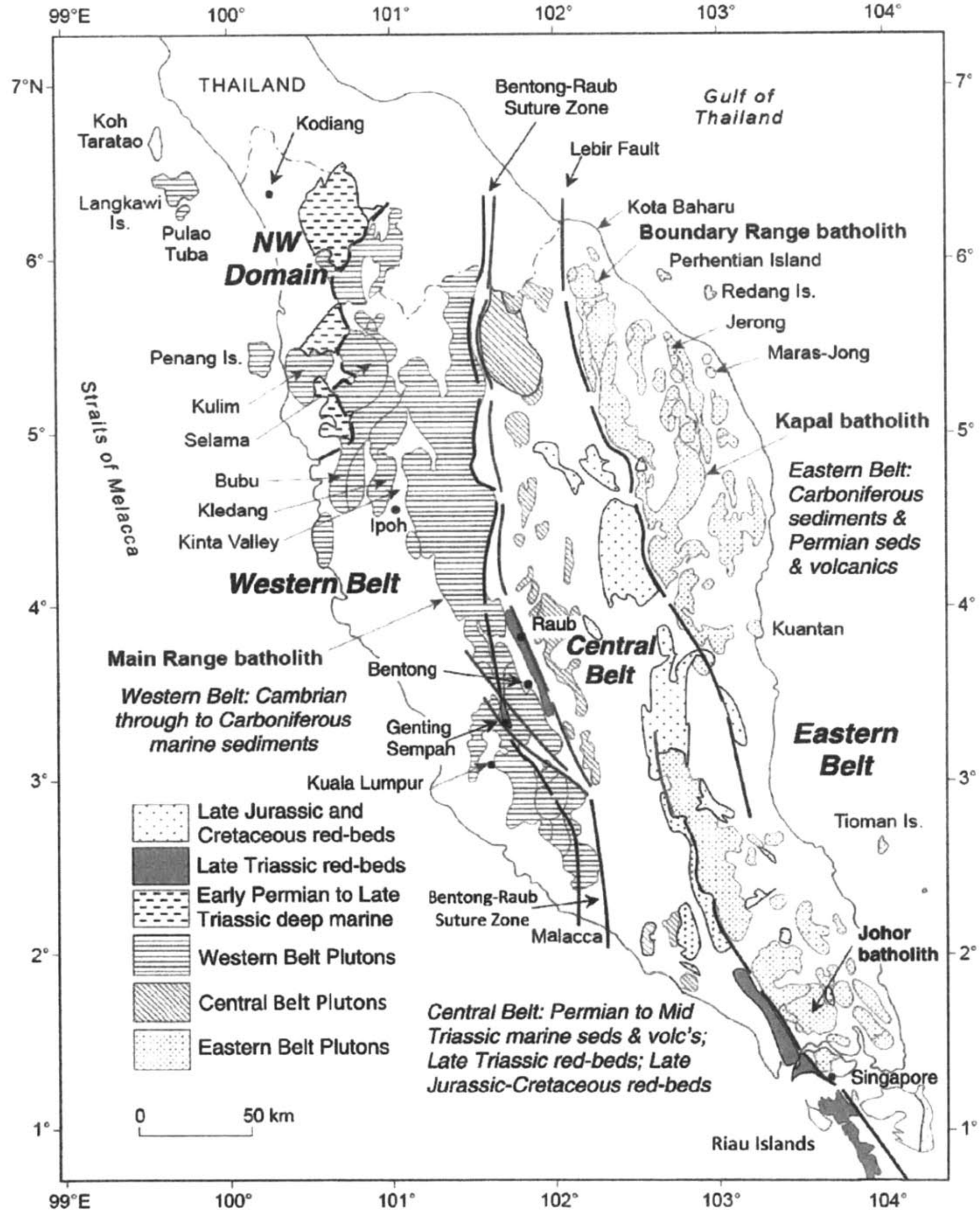


Fig 2.1: Regional geological map of Peninsular Malaysia
(reproduced from Oliver & Prave, 2013 after Tate et.al., 2008)

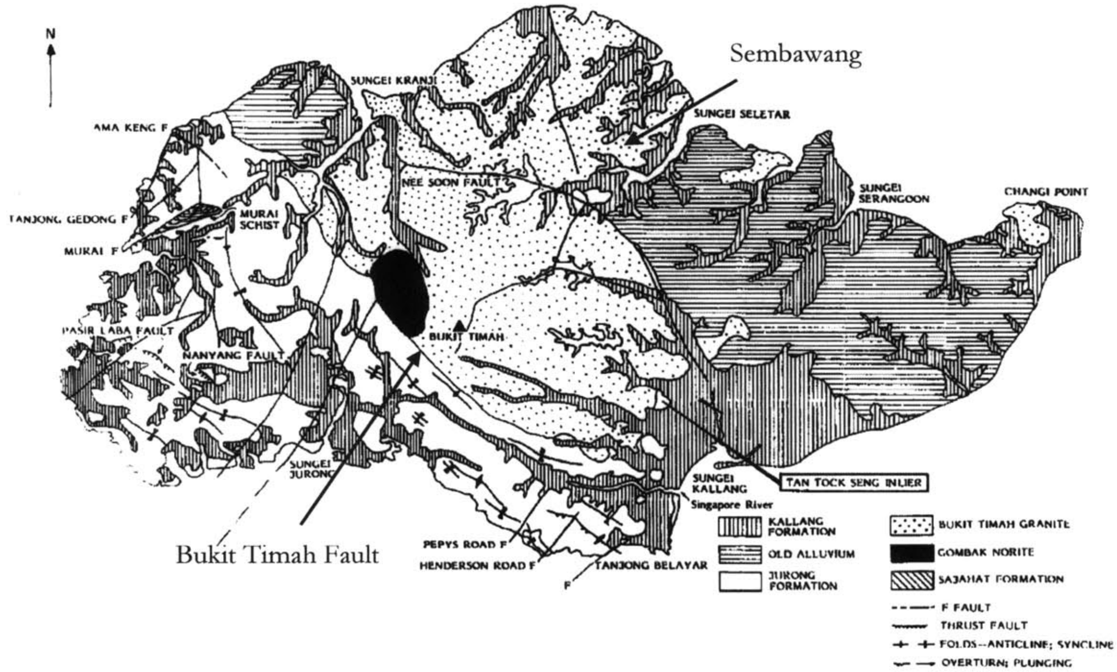


Fig 2.2: Generalized geological map of Singapore (reproduced and modified from Pitts, 1984)

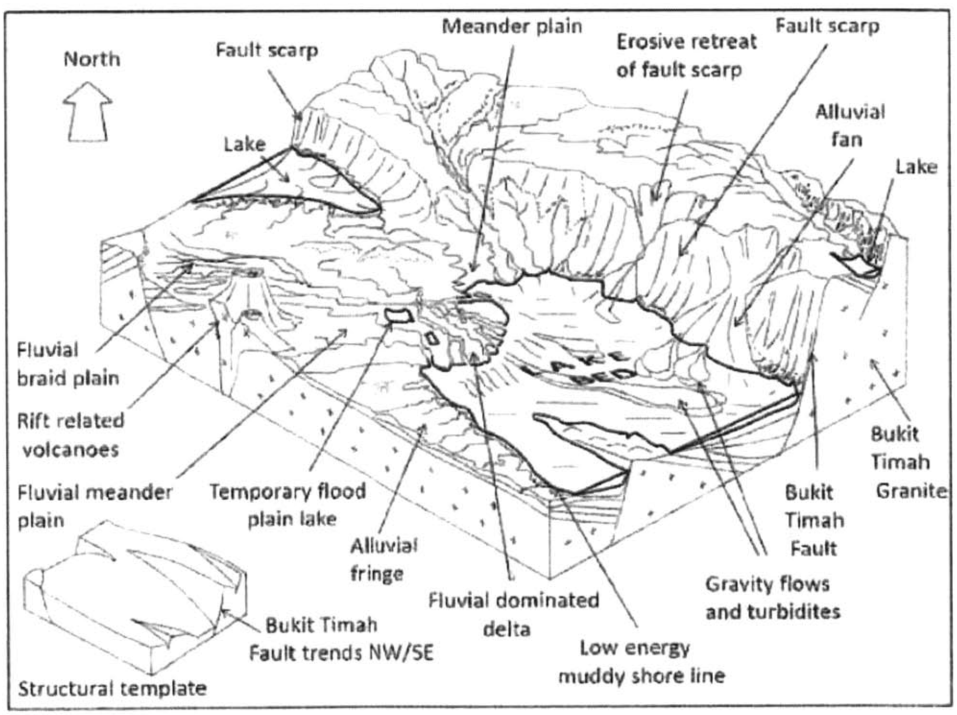


Fig 2.3: Paleographic reconstruction of Central Singapore island in red-bed Triassic times. Lakes and rivers occupy a half graben in front of the active Bukit Timah Fault Scarp (reproduced from Oliver & Prave, 2013, after Sladen, 1997)

As described by Sharma, Chu & Zhao (1999), the geologic materials of Singapore can be broadly classified into four main types, with other less common geological formations like the Sajahat Formation and the Tekong Formation. The four main types of geologic materials are:

a) the igneous rocks consisting of the Bukit Timah granite and the Gombak norite in the north and central-north;
b) the sedimentary rocks of the Jurong Formation in the west and southwest;
c) the Quatenary deposits of the Old Alluvium in the east; and
d) recent deposits of the Kallang Formation of the alluvium member..

The generalized geological map of Singapore in Fig 2.2, shows the various geological formations. Descriptions of each of the four main types of geologic materials, and the Sajahat Formation, are summarized from Pitts (1984), Sharma, Chu and Zhao (1999), Rahardio et.al. (1994), Zhao et.al. (1999) and Zhao et.al. (1995).

2.2.1 BUKIT TIMAH GRANITE AND GOMBAK NORITE

The Gombak Norite is an association of noritic and gabbroic rock which outcrops in a restricted area, and is coarse-grained and has high proportions of plagioclase. It is a basic intrusive rock that has very high strength. The Bukit Timah Granite is mainly an acidic igneous rock formed during the Triassic period. Bukit Timah Granite is grey, medium to coarse grained (2 to 5mm) and consist of mainly feldspar, some quartz, biotite and hornblende. The pink variety of orthoclase is also present. Typical weathering profile of residual soil from Bukit Timah Granite is shown in Fig 2.4.

It has been suggested that the granite has intruded into the widespread sedimentary sequence of the Jurong Formation. It extends about 8km from north to south and 7km east to west, which makes up about one-third of the Singapore Island and the whole of Pulau Ubin. The weathering of the Bukit Timah Granite has been rapid and extensive, with an average depth of weathering of 30m, which is likely attributed to the chemical decomposition under the humid tropical climate.

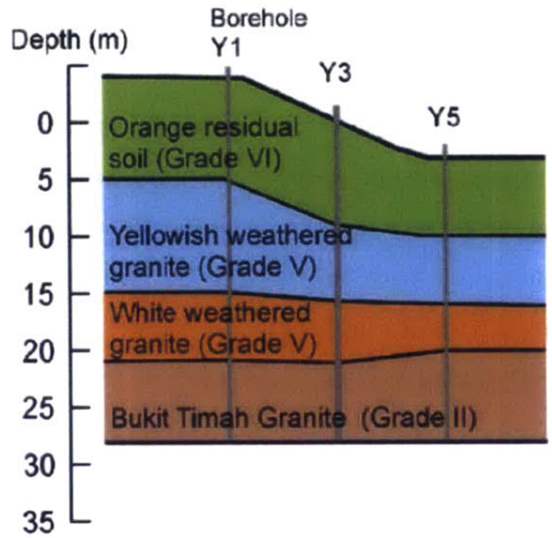


Fig 2.4: Typical weathering profile of residual soil from Bukit Timah Granite (Yishun slope) (reproduced from Rahardjo et. al., 2004)

2.2.2 JURONG FORMATION

The Jurong Formation is an extensive rock formation covering west and southwest Singapore Island. Formed in the late Triassic to early Jurassic age, the Jurong Formation has been divided into six facies members. Although each facies is suggested to have deposited in similar environments, the relationships of the members remain unclear, and are often confused. Because of the transitional nature of the deposition, part marine and part terrestrial, the sedimentary rock types of the Jurong Formation vary widely vertically and laterally. The bulk mineral composition of the Jurong Formation includes quartz, clay, feldspars and other altered igneous rock minerals. There's a wide spectrum of gradation found in the Jurong formation, from fine-grained mudstone to boulder conglomerates.

A limestone unit, the Pandan limestone, is reported to be found in the Jurong Formation, with observations at Pasir Panjang, Tuas, Buona Vista and Kent Ridge. The age of the limestone is dated to late Triassic. It was suggested that it is possible that all the limestones were deposited in the same period, and it is likely that the limestones form the lower part of the Jurong Formation. Another unit, the Fort Canning Boulder Bed, a bed of hard sandstone boulders up to 6m across, set in a stiff reddish brown and grey silty matrix, is considered by many to be part of the Jurong Formation. The Bed covers a large part of the Central Business District, and because of its variable soil and rock lithology, has caused problems for deep excavations and foundations for multi-storey buildings.

The Jurong Formation has been intensely folded, with structural strike oriented NW-SE and faults aligned parallel with or perpendicular to the strike direction. In the Western part of Singapore, the sedimentary rocks of the Jurong Formation are covered by weathered layers of a few meters to a few tens of meters, and at some locations, recent deposits. A typical weathering profile of residual soil from the Jurong Formation is shown in Fig 2.6.

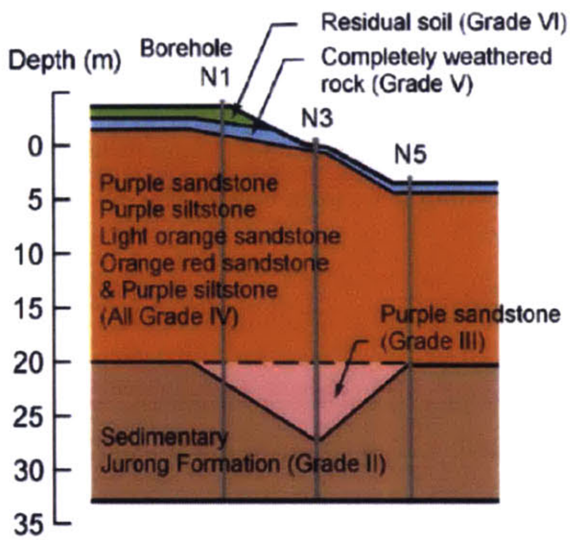


Fig 2.5: Typical weathering profile of residual soil from the Jurong Formation (NTU-CSE slope) (reproduced from (Rahardjo et. al., 2004)

2.2.3 OLD ALLUVIUM

The Old Alluvium consists of alluvial deposits of medium dense to very dense clayey coarse sand, fine gravel and lenses of silt and clay, and forms an extensive sheet of weakly consolidated to unconsolidated sediments across the eastern side of Singapore, and a smaller area in the North-West. It has a maximum recorded thickness of 195 meters in Singapore, but is more commonly of the order of 50 meters thick (Pitts, 1984). Three weathering grades for Old Alluvium was devised by Burton (1964), and in the weathered zone, there is almost complete destruction of ferro-magnesian minerals, complete alteration of feldspars to kaolin and dissolution of some quartz (Sharma, et.al., 1999).

2.2.4 KALLANG FORMATION

The Kallang Formation consists of deposits with marine, fluvial, littoral, coral reef and estuarine origins, and are found in coastal areas and low-lying valleys around the island. The Kallang Formation is known to consist of two distinct units of marine clay, which consist mostly of kaolinite with a flocculate structure (Sharma et al., 1999). When civil and engineering work is done in this formation, large settlements can occur as the marine clay has low cohesive strength and the natural water content is close to the liquid limit (Pitts, 1984).

2.2.5 SAJAHAT FORMATION

Occurring in the north-eastern corner of Singapore and Pulau Ubin, the Sajahat Formation was formed during the lower Paleozoic and is the oldest rock of Singapore (Sharma et al., 1999). It consist of sandstone and mudstones, and is seldom encountered during civil engineering activity, and it is known to exist below the Old Alluvium in the eastern part of Singapore (Pitts, 1984).

There has not been any documented study of site and cavern feasibility studies in this area, even though the Singapore Geological Office (SGO)'s current site investigation encompasses this area.

2.3 ENGINEERING PROPERTIES FROM PREVIOUS CAVERN FEASIBILITY STUDIES

In the previous chapter, in Table 1.1, it was highlighted that many early feasibility studies were done for rock caverns Singapore. In the literature, engineering properties of the igneous rocks in the Bukit Timah Granite, and the sedimentary rocks in the Jurong Formation, including relevant considerations for cavern developments have also been discussed, and will be summarized in the following section.

2.3.1 BUKIT TIMAH GRANITE

The UAF was built in Bukit Timah Granite, and in the studies before, it was concluded that the Bukit Timah Granite has good potential for underground cavern construction (Zhao et.al., 1995). The findings of Zhao et.al. (1995) will be presented in this section.

2.3.1.1 Rock strength and deformability

Laboratory results done for the feasibility study shown the Bukit Timah Granite has an average uniaxial compressive strength of around 190 MPa, a Young's modulus of 83.6 GPa, a Poisson's ration of 0.25, a tensile strength of 16.6 MPa, and a point load index of 8.9 MPa. Test results are shown in Table 2.1. The degree of weathering affects the strength of the granite, with the uniaxial compressive strength of moderately weathered rock only about 40% of fresh granite, as shown in Fig 2.6. Typical deformation characteristics of the Bukit Timah Granite material under uniaxial compression are shown in Fig 2.7. The natural joints have a peak shear angle of 35-45^0, and their shear strength are shown in Fig 2.8.

Table 2.1: Mechanical Properties of fresh Bukit Timah Granite (Reproduced from Zhao et.al., 1995)

Property	Range	Average
Density (kg/m^3)	2430 – 2650	2610
Uniaxial compressive strength (MPa)	159 – 232	186
Young's modulus (GPa)	67 – 131	84
Poisson's ratio	0.15 – 0.31	0.25
Point load index (MPa)	7.5 – 10.7	8.9
Brazil tensile strength (MPa)	9.9 – 11.7	11.4
Four-point flexure tensile strength (MPa)	14.6 – 18.6	16.6
Schmidt rebound hardness	60.4 – 67.6	64.5
Compressional wave velocity (m/s)	5490 – 6270	5790

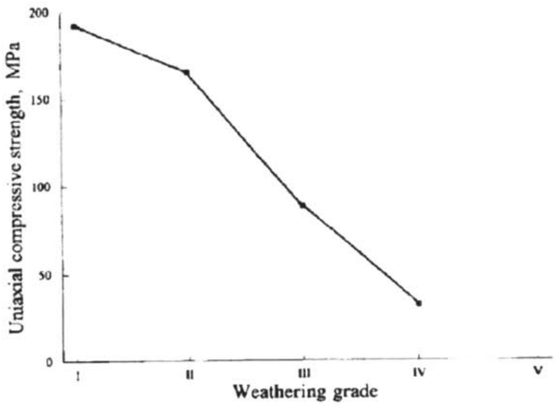


Fig 2.6: Change of strength of the Bukit Timah Granite with weathering

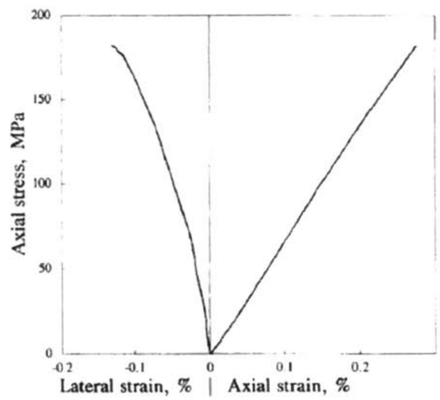


Fig 2.7: Typical deformation characteristics of the Bukit Timah Granite

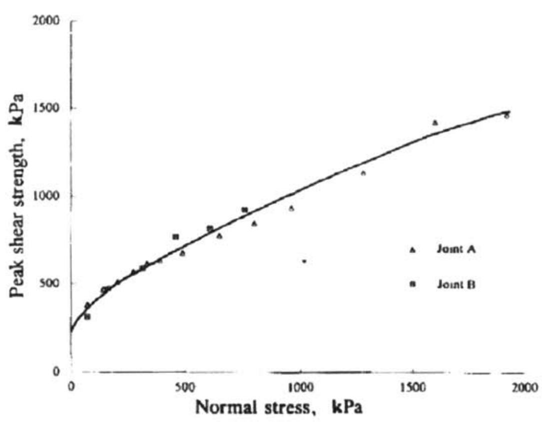


Fig 2.8: Shear strength of natural joints of the Bukit Timah Granite

(Fig 2.6 to 2.8 reproduced from Zhao et.al., 1995)

2.3.1.2 Discontinuities

Generally, there are three to four sets of joints in the granite rock masses. The dominant joint set is sub-vertical, with the strike at NNW-SSE. Secondary sets have strikes at NNE-SSW and NW-SE. Joints are widely spaced, and the surfaces of the joints are generally planar, closed and rough. RQD is consistently close to 100%. There is a fault zone observed in Sembawang, North-East of Singapore and shown in Fig 2.2, with a width of about 40m with a strike direction of N-S. The joint rosette of joint system observed in the study is reproduced in Fig 2.9.

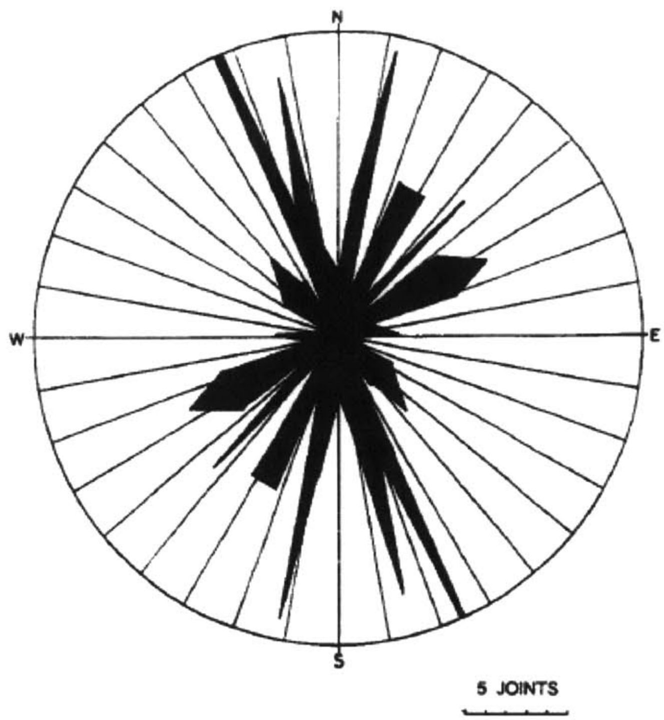


Fig 2.9: Joint rosette of joint system observed in the study (reproduced from Zhao et.al., 1995)

2.3.1.3 Permeability

Rock mass permeability was measured using Lugeon tests, and average rock mass permeability is in the range of 10^{-7} to 10^{-9} m/s. With the increase in effective stress and joint density with depth, permeability of the Bukit Timah Granite is expected to decrease.

2.3.1.4 Rock Mass Quality

Rock Mass Quality was assessed using the *Q*-system, that takes into account the joint orientation and frequency, rock lock size, rock strength, in-situ stress, and ground water conditions. It was found that Q-values of the Bukit Timah Granite is generally in the range from 10 to 400, with small areas of weak zones. The distribution of the *Q*-values for the Bukit Timah Granites is shown in Fig 2.10.

The results showed that about 98% of logged length of the rock mass can be classified as good and above, while 77% is classified as very good and excellently good. This suggests that large unsupported caverns can be constructed in Bukit Timah Granite. An SRF of 1.0 has been assumed adopted for depths greater than 50m, assuming medium stress conditions in the study. However, it should be noted that there are problems with obtaining *Q*-values from just borehole logs. The *Q*-system and its limitations will be described and discussed in detail in Chapter Three.

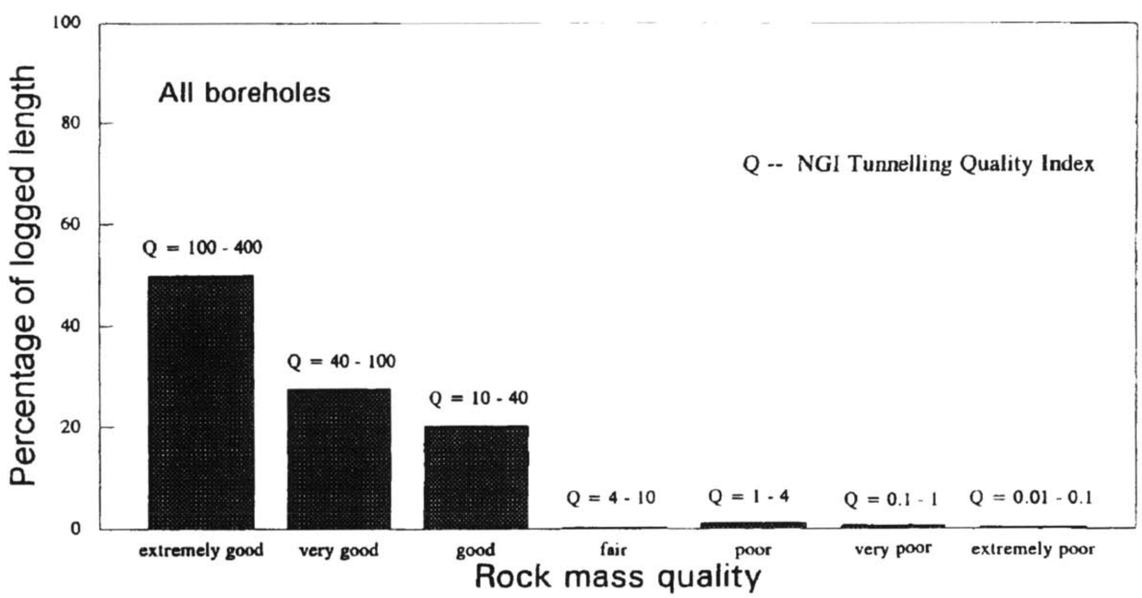


Fig 2.10: Distribution of assessed *Q*-values in the Bukit Timah Granite
(Reproduced from Zhao et.al., 1995)

2.3.2 JURONG FORMATION

The JRC is located within the Jurong Formation, and the Underground Science City under Kent Ridge, if constructed, will also be in the Jurong Formation. The area of interest in Western Singapore for this study is also in Jurong Formation. Hence, it is of particular relevance to look at past studies of the rock properties and their comments on cavern feasibility. In general, the sedimentary rocks of the Jurong Formation display a wide range of engineering properties which depend upon the rock types, the extent of weathering and the fractures in them. Different sites have a better potential for housing underground cavern developments than others (Zhao et.al., 2001; Zhao et.al., 1999). The layout of geological investigation of the Jurong Formation in their feasibility studies is shown in Fig 2.11.

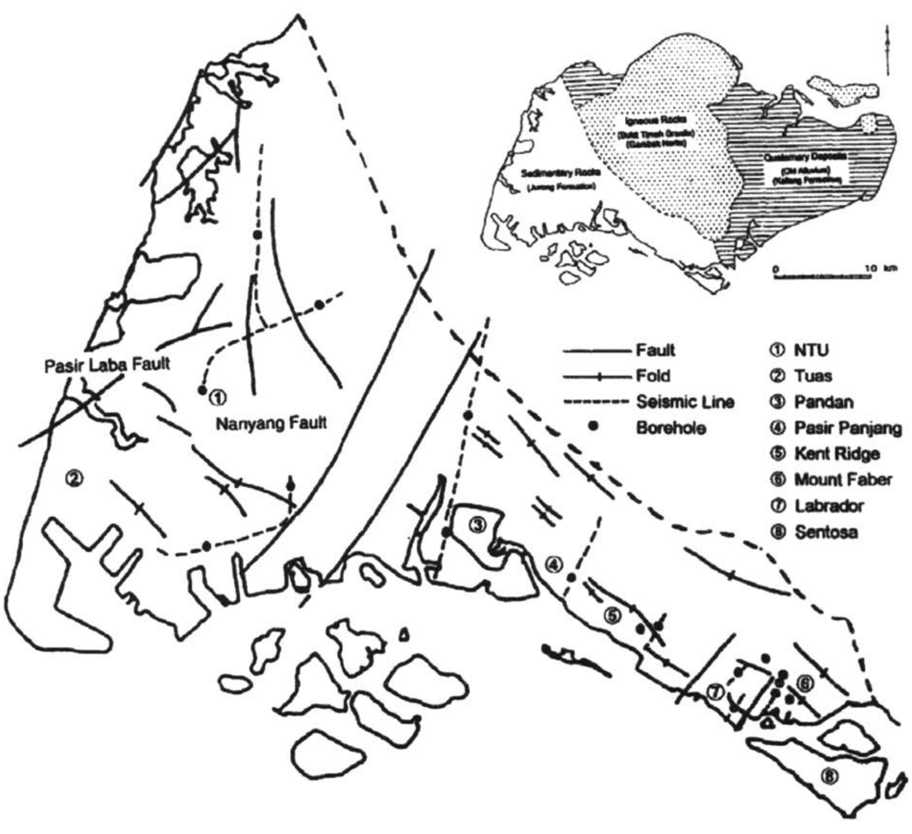


Fig 2.11: Layout of geological investigation of the Jurong Formation from previous feasibility studies (Reproduced from Zhao et.al., 1999)

2.3.2.1 Rock Strength and deformability

Different rock types – siltstone, sandstone, conglomerate, tuff and marble – are encountered in the Jurong Foundation. Laboratory tests indicate that the rock type and degree of weathering affects the rock strength (Zhao et al., 2001). The point load index strength, uniaxial compression strength and Young's modulus for each rock type are summarized in table 2.2.

Table 2.2: Summary of rock strength and Young's modulus of rock (Reproduced and modified from Zhao et.al., 2001)

Property	Siltstone	Sandstone	Conglomerate	Tuff	Marble
Uniaxial compressive strength (MPa)	0.1 – 3.6	5 – 117	23 – 42	28 – 79	15 – 52
Young's modulus (GPa)	0.01 – 16	-	4 – 60	5 – 30	3 – 30
Point load index (MPa)	0.01 – 4.45	0.01 – 24	0.04 – 12.5	0.02 – 7.89	0.4 - 8

2.3.2.2 Discontinuities

Zhao et al. (2001) describe the most prevalent joints are steeply inclined sub-vertical 70^0 to 85^0, moderately to widely spaced joints, generally within the relatively stronger rocks such as sandstone, conglomerate and tuff. Other joint sets were inclined shear joints with dip angles in the range from 40^0 to 70^0. RQD values generally range from 20% to 80% with a weighted average value of about 35%. A summary of the RQD values along the depth of one borehole is shown in Fig 2.12.

2.3.2.3 Permeability

Permeability characteristic of the rock mass is affected by the fracturing of the rock mass in the Jurong Formation. Again, using Lugeon tests, carried out in conglomerate, sandstone and tuff, the permeability was found in the range of 10^{-7} to 10^{-9} (Zhao et al., 2001).

2.3.2.4 Rock Mass Quality

Using the Q-system to assess the rock mass qualities, about 47% of the logged rock cores are rated as fair (Q = 4 – 10) to good (Q = 10-40) or very good (Q = 40 – 100). Poor quality rocks are usually mudrocks or highly faulted rocks (Zhao et al., 1999). Again, an SRF of 1.0 has been assumed and adopted for depths greater than 50m, assuming medium stress conditions. A summary of the *Q*-values along the depth of one borehole is also shown in Fig 2.12.

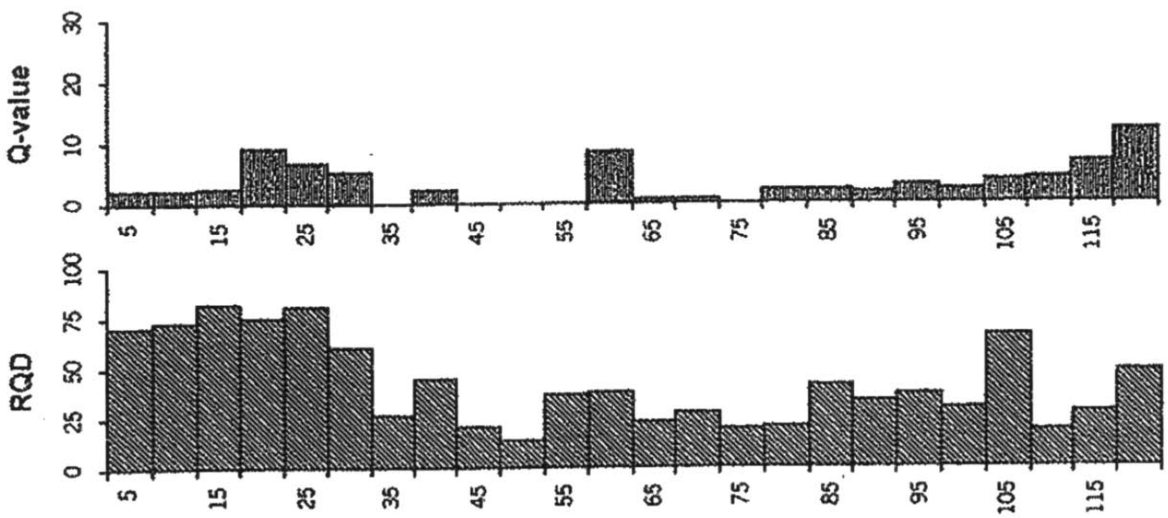


Fig 2.12: Summary of RQD and *Q*-values along the depth of one borehole (Reproduced from Zhao et.al., 2001)

CHAPTER THREE

EMPIRICAL ROCK SUPPORT DESIGN METHODS

3.1 INTRODUCTION

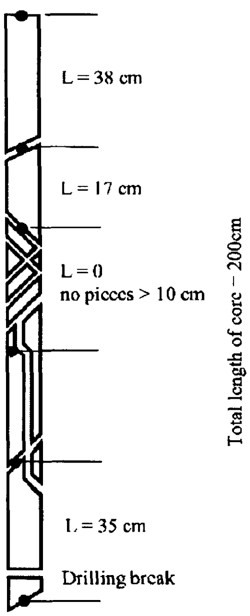
Empirical methods have always been important in underground construction as compared to many branches of engineering, possibly because underground construction happens in an unknown environment with wide variability (Steiner & Einstein, 1980). Empirical methods for rock support design have gained widespread use worldwide, and some of these methods include the Rock Quality Designation (RQD) method, and geomechanical rock mass classifications like the Q-system, proposed by Barton, Lien and Lunde in 1974, and Bieniawski's RMR classification established in 1976.

Empirical methods are easy to apply, and it has been proposed that during the feasibility and preliminary design stages of a project, when there is a lack of detailed information, use of an empirical rock mass classification scheme can be of considerable benefit (E Hoek, Kaiser, & Bawden, 2000). Moreover, these methods can be used in all phases of design, with the design being constantly updated after more information of the ground has been gathered. However, when using empirical methods, Bieniawski (1987) recommends that at least two methods should be applied, and in general, empirical design methods should not be used to replace more elaborate design procedures, and must be accompanied by sound judgement and practical experience.. (Bieniawski, 1987; Palmstrom & Broch, 2006; Stille & Palmström, 2003).

The study done in this thesis will look at two of the empirical methods, the RQD and the Q–system, to derive required rock support systems. This chapter will cover the basis for the two empirical methods, how the rock support recommendations are derived, and the applicability and limitations that should be kept in mind when these methods are applied.

3.2 ROCK QUALITY DESIGNATION (RQD) METHOD

RQD is an index for assessing rock quality quantitatively, and is a more sensitive index of the core quality than the core recovery method. According to Deere, the RQD is based on a modified core recovery procedure that takes into account only hard and sound pieces of core which are at least 10cm long. The RQD hence, indirectly measures the number of fractures and implicitly include other features like weathering and alteration. In summary, RQD is calculated as follows, and illustrated in Fig 3.1:



$$RQD = \frac{sum\ of\ length\ of\ core\ pieces\ \geq 10cm}{total\ run\ of\ core} * 100\ (\%)$$

For example on the left,

$$RQD = \frac{38 + 17 + 20 + 35}{200} * 100 = 55\%$$

Fig 3.1: Procedure for measurement and calculation of RQD
(Based on Deere, 1989, modified and reproduced from Hoek et al., 2000)

The procedure for the determination of the RQD gives low values for rock where recovery is poor. Rocks with poor core recovery generally imply poor quality rock. However, if the poor recovery is a result of poor drilling equipment and techniques, then RQD will not give an appropriate indication. As such, for RQD determination, the International Society for Rock Mechanics (ISRM) recommends core size of at least NX (size 54.7mm) drilled with double-tube core barrel using a diamond bit, with proper supervision.

Deere (1968) showed that there is a reasonably good relation between the numerical values of RQD and the general quality of rock for engineering purposes, which is shown in Table 3.1.

Table 3.1: RQD and rock quality relationship

RQD (%)	Description of Rock Quality
0-25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

3.2.1 Determination of rock support using RQD

Based on RQD, Deere et.al. (1969) estimated the required support quantities, shown in Table 3.2. The support quantities are given for two different construction methods, the drill and blast method, and the machine boring method. In Table 3.2, three alternative support systems are provided, which include steel sets, rock bolts and shotcrete, with recommendations for a combination of these support systems in certain circumstances. As with all empirical methods, the relationship are based on several simplifying assumptions, listed in Deere et.al. (1969), reproduced from Steiner and Einstein (1980), Table 3.5.4:

(1) The RQD adequately describes the quality of the rock;

(2) The support systems are installed as close to the face as possible; for steel sets and for rock bolts this would be about 2 to 4 feet, and for shotcrete essentially zero. Furthermore it is assumed that the support systems are properly installed, i.e. lagging and blocking is tightly placed behind steel sets and rock bolts are properly tensioned.

(3) The tunnel has a cross-section (either horse-shoe or circular) with the height approximately equal to the width.

(4) The tunnel is approximately 20 to 40 feet in width.

(5) The natural stresses in the ground are low enough that stress concentrations around the periphery generally do not exceed the compressive strength of the rock.

From Table 3.2, it is also stated that the support recommendations are based on 1969 technology in the United States.

Steiner and Einstein (1980) described the developments of the RQD-support relations by Deere, his students and collaborators at the University of Illinois, and detailed the studies by them, including Coon (1968), Deere, Merritt and Coon (1968), Cecil (1975) and Merritt (1975). A brief summary of these developments is provided below.

The initial work done by Coon (1968), which was also reported in Deere et.al. (1968), consisted of 14 tunnels, and is summarized in Table 3.3. Cecil (1975) studied a total of 97 tunnel sections in detail and attempted to correlate RQD, width and support requirements. The comparison of the support recommendations for Coon and Cecil is shown in Fig. 3.2. It is evident that many of Cecil's cases fall into the area of maximum rock support, which he classified as "anomalies", highlighting that geologic features including (1) softening clay materials, (2) thin clay coatings, and (3) single-sets of steeply dipping closely spaced tight joints "may have absolutely no relationship to the rock quality designation". While (1) and (2) may lead to instability in rock with RQD more than 75%, (3) is often stable at RQD values lower than 20%.

Merritt (1972) advanced the RQD-support span relations by considering about 60 case studies. Merritt's studies included steelset and bolt supported tunnels, but no shotcrete support. In particular, Merritt's results showed the "continued development of the RQD-support relation and a continuous increase in the range of applicability, in particular the use of rock bolts for a wider range of RQD" (Steiner & Einstein, 1980). Merritt's result is shown in Fig. 3.3.

Table 3.2: RQD-Support Relations (Based on Deere et.al., 1969, and reproduced from Steiner and Einstein, 1980, Table 3.5.3)

GUIDELINES FOR SELECTION OF PRIMARY SUPPORT FOR 20-FT TO 40-FT TUNNELS IN ROCK

Rock Quality	Construction Method	Alternative Support Systems: Steel Sets: Rock Load (B = Tunnel Width)	Steel Sets: Weight of Sets	Steel Sets: Spacing[c]	Rock Bolts[a] (Conditional use in poor and very poor rock): Spacing of Pattern Bolts	Rock Bolts[a]: Additional Requirements and Anchorage Limitations[a]	Shotcrete[b] (Conditional use in poor and very poor rock): Total Thickness: Crown	Shotcrete[b]: Total Thickness: Sides	Shotcrete[b]: Additional Support[b]
Excellent[d] RQD > 90	Boring machine	(0.0 to 0.2)B	Light	None to occasional	None to occasional	Rare	None to occasional local application	None	None
	Drilling and blasting	(0.0 to 0.3)B	Light	None to occasional	None to occasional	Rare	None to occasional local application 2 to 3 in.	None	None
Good[d] RQD = 75 to 90	Boring machine	(0.0 to 0.4)B	Light	Occasional to 5 to 6 ft	Occasional to 5 to 6 ft	Occasional mesh and straps	Local application 2 to 3 in.	None	None
	Drilling and blasting	(0.3 to 0.6)B	Light	5 to 6 ft	5 to 6 ft	Occasional mesh or straps	Local application 2 to 3 in.	None	None
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)B	Light to medium	5 to 6 ft	4 to 6 ft	Mesh and straps as required	2 to 4 in.	None	Provide for rock bolts
	Drilling and blasting	(0.6 to 1.3)B	Light to medium	4 to 5 ft	3 to 5 ft	Mesh and straps as required	4 in. or more	4 in. or more	Provide for rock bolts
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)B	Medium circular	3 to 4 ft	3 to 5 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	4 to 6 in.	4 to 6 in.	Rock bolts as required (~4-6 ft cc.)
	Drilling and blasting	(1.3 to 2.0)B	Medium to heavy circular	2 to 4 ft	2 to 4 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.	6 in. or more	6 in. or more	Rock bolts as required (~4-6 ft cc.)
Very poor RQD < 25 (Excluding squeezing and swelling ground)	Boring machine	(1.6 to 2.2)B	Medium to heavy circular	2 ft	2 to 4 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium sets as required
	Drilling and blasting	(2.0 to 2.8)B	Heavy circular	2 ft	3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Medium to heavy sets as required
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft	2 to 3 ft	Anchorage may be impossible. 100 percent mesh and straps required.	6 in. or more on whole section		Heavy sets as required

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock.

[a]Bolt diameter = 1 in., length = ⅓ to ¼ tunnel width. It may be difficult or impossible to obtain anchorage with mechanically anchored rock bolts in poor and very poor rock. Grouted anchors may also be unsatisfactory in very wet tunnels.

[b]Because shotcrete experience is limited, only general guidelines are given for support in the poorer quality rock.

[c]Lagging requirements for steel sets will usually be minimal in excellent rock and will range from up to 25 percent in good rock to 100 percent in very poor rock.

[d]In good and excellent quality rock, the support requirement will in general be minimal but will be dependent on joint geometry, tunnel diameter, and relative orientations of joints and tunnel.

Table 3.3: Summary of initial case studies for RQD-support relation
(Based on Deere et.al., 1968, and reproduced from Steiner and Einstein, 1980, Table 3.5.6)

Project	Width of Opening, ft.	Support	RQD or Velocity Index
1 Pigeon River No. 1	36	Unsupported	87
2 Pigeon River No. 2	36	8 in. WF 10 to 4 ft	29
3 Tehachapi Site 3	21		see Fig.3.5.8
4 Tehachapi Site 1	28	8 x 8 in. 6 ft o.c., and bolts 5 ft o.c.	54-80
5 Straight Creek	13	Heavy support to unsupported	see Fig.3.5.9
6 Cavity I, NTS	Hemisphere with radius of 60 ft	Top 32 ft - bolts, 3 ft o.c. Mid 24 ft - bolts, 3 ft o.c. Bot 16 ft - bolts, 6 ft o.c.	72 90
7 Cavity II, NTS	Hemisphere with radius of 60 ft	Top 32 ft - bolts, 3 ft o.c. Mid 24 ft - bolts, 3 ft o.c. Bot 16 ft - bolts, 6 ft o.c.	69
8 Cavity III, NTS	Hemisphere with radius of 35 ft	Top 24 ft - bolts, 3 ft o.c. Mid 16 ft - bolts, 3 ft o.c. Bot 8-16 ft - bolts, 6 ft o.c.	75
9 Adit at Two Forks	5		78
10 Adit at Yellowtail Dam	8		
11 Adits at Dworshak Dam	5		81
12 Diversion Tunnel at Dworshak Dam granite gneiss 960 ft	30	47% ribs, 53% bolts	Estimated 85
Schistose gneiss 760 ft	11	77% ribs, 23% bolts, ribs 5 ft o.c.	Estimated 70
13 Shaft, East Coast	20	Temporary timber and bolts - concrete lining w/ i 20 ft of face	44-93
14 Tunnel, NTS	10	Unsupported	75

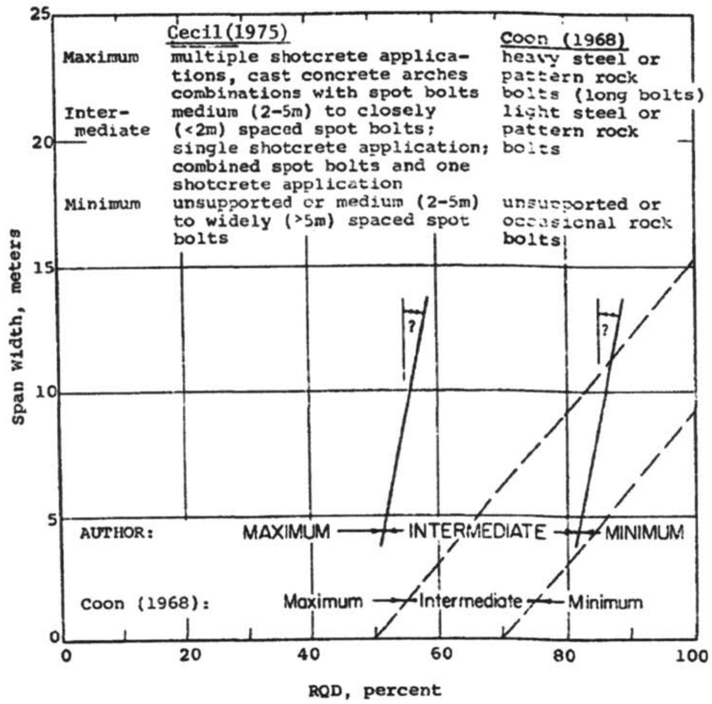


Fig 3.2: Comparison of Cecil's and Coon's support region boundaries

(based on Cecil, 1975, reproduced from Steiner and Einstein, 1980, Figure 3.5.11b)

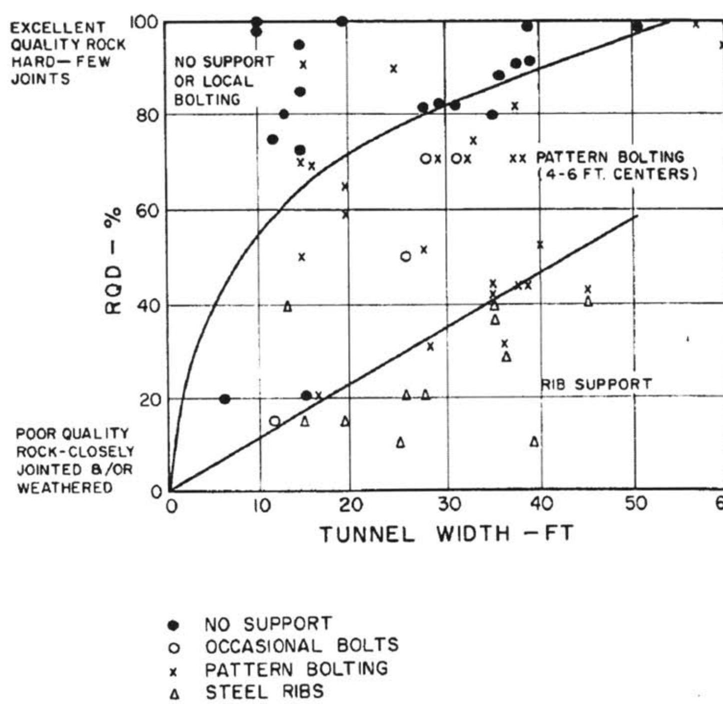


Fig 3.3: RQD support relations for bolt and steel set supported tunnels

(based on Merritt, 1972, reproduced from Steiner and Einstein, 1980, Figure 3.5.13a)

3.2.2 Applicability and Limitations of deriving rock support from RQD

RQD is easy to assess, and support prediction based on Table 3.2 is easy to perform for a first approximation of required support systems, and is useful for designers to link information from rock cores to anticipated underground construction characteristics. However, it should be noted that the RQD alone is not all-emcompassing, and does not account for all factors that influence the stability and support requirements. In particular, if ground information is available, it should be taken into account. Furthermore, RQD is highly sensitive to sampling line orientation due to preferential orientation distribution of discontinuities, and cannot account for size and length of the discontinuities (Palmstrom & Broch, 2006).

Assumptions of the RQD method, listed by Deere et.al. (1969), were also discussed earlier in this chapter. One of the most important limitations as a result of these assumptions for the support recommendations, is that the relationship is empirically defined for tunnels of 20 to 40 feet (about 6 to 12 meters) in diameter. Hence, the RQD based support prediction might not remain conservative when the RQD-support relation in Table 3.2 is applied to underground design beyond that defined.

3.3 BARTON ET.AL.'S *Q*-SYSTEM

The *Q*-system for ground classification was proposed by Barton, Lien and Lunde of the Norwegian Geotechnical Institute (NGI) for estimating rock support in tunnels, from evaluating an approximately 200 case histories of tunnels and caverns. The numerical value of the index *Q* is defined by six parameters, namely:

(1) RQD, the rock quality designation,
(2) Jn, the joint set number,
(3) Jr, the joint roughness number,
(4) Ja, the joint alteration number,
(5) Jw, the joint water reduction factor, and
(6) SRF, the stress reduction factor.

The classification of the individual parameters used for the evaluation of the index *Q* is shown in Table 3.4. The index *Q* is then defined by the six parameters in the following equation:

$$Q = \frac{RQD}{Jn} * \frac{Jr}{Ja} * \frac{Jw}{SRF}$$

Barton et.al. (1974) offer an explanation to the meaning of the parameters used, pointing to the three quotients in the equation to determine *Q* as crude measures of (1) relative block size (RQD/Jn), (2) inter-block shear strength (Jr/Ja) and (3) active stresses (Jw/SRF).

3.3.1 Index *Q* and rock support design

Although the RQD is rather straightforward to determine, Jn, Jr and Ja require more experience and may be difficult to determine (Steiner & Einstein, 1980). In fact, collection of field data for these parameters in evaluating the index *Q* is usually done by logging rock exposures. Hence, the index *Q* needs to be updated as the ground is further exposed during construction (Barton et al., 1994). In some cases, where the rock mass is not uniform, or when the shear zones are less than 0.5 meters and occur frequently, a wall length of up to 50 meters may be needed to obtain an overall picture of the rock mass quality (Singh & Goel, 1999).

From the *Q*-system, nine different rock mass quality classes are defined, from "exceptionally poor" to "exceptionally good". The numerical value of the index *Q* varies on a logarithmic scale from 0.0001 to a maximum of 1,000. An estimation of the rock support system can be made with an additional dimension - the ratio between the span (for roof support) or height of the opening (for wall support) and an excavation support ratio (ESR), from the chart shown in Fig. 3.4. Hoek (2000) suggests that the value of ESR is related to the intended use of the excavation and the degree of security demanded of the support system installed to maintain the stability of the excavation. The excavation category and corresponding ESR value, as suggested by Barton et.al. (1974), are shown in Table 3.5.

Table 3.4: Classification of the individual parameters used for index Q (based on Barton et.al., 1974, reproduced from Hoek, 2000) (page 1 of 3)

DESCRIPTION	VALUE	NOTES	
1. ROCK QUALITY DESIGNATION	***RQD***		
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),	
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.	
C. Fair	50 - 75		
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently	
E. Excellent	90 - 100	accurate.	
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	3		
D. Two joint sets	4		
E. Two joint sets plus random	6		
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)	
G. Three joint sets plus random	12		
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)	
J. Crushed rock, earthlike	20		
3. JOINT ROUGHNESS NUMBER	J_r		
a. Rock wall contact			
b. Rock wall contact before 10 cm shear			
A. Discontinuous joints	4		
B. Rough and irregular, undulating	3		
C. Smooth undulating	2		
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is	
E. Rough or irregular, planar	1.5	greater than 3 m.	
F. Smooth, planar	1.0		
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having	
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for	
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	minimum strength.	
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)		
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle,
B. Unaltered joint walls, surface staining only	1.0	25 - 35	are intended as an approximate guide
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	to the mineralogical properties of the alteration products, if present.
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. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

Table 3.4: (cont'd) Classification of the individual parameters used for index Q (Based on Barton et.al., 1974, reproduced from Hoek, 2000) (page 2 of 3)

4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		***SRF***	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

Table 3.4: (cont'd) Classification of the individual parameters used for index *Q* (Based on Barton et.al., 1974, reproduced from Hoek, 2000) (page 3 of 3)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			***SRF***	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When σ_1/σ_3 > 10, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3 J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Table 3.5: Excavation category and corresponding Excavation Support Ration (ESR) (after Barton et.al., 1974)

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

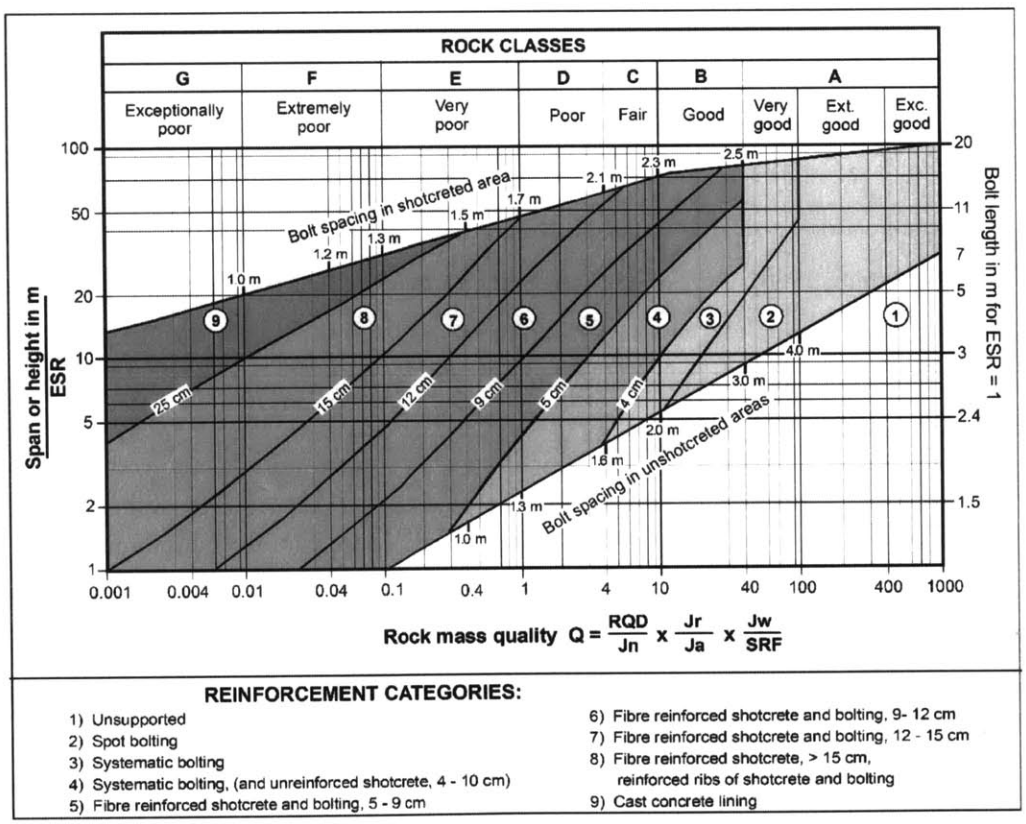


Fig 3.4: The *Q* support chart

(based on Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006)

3.3.2 Applicability and Limitations of deriving rock support from *Q*

The *Q*-system has been widely used for the design of rock support for underground rock tunnels, as well as the 62 meter span Gjøvik Olympic Mountain Hall in Norway (Barton et al., 1994; Broch, Myrvang, & Stjern, 1996). However, the *Q*-system has its limitations, and must be understood prior to its application.

One of the major limitation of the *Q*-system is that the values for the parameters Jn, Jr and Ja are difficult to quantify. Therefore, unlike the RQD, it is difficult to translate from borehole logging directly to a rock support design. Values of Jn, Jr and Ja from borehole coring will not be able to give an accurate representation of the quality of rock mass, and could be erroneous when determining the rock support system required. It has been mentioned earlier in Chapter Two, studies done in Singapore evaluated cavern feasibility by determining the index *Q* from borehole loggings. It is perhaps acceptable for the purpose of preliminary feasibility study, but the difficulty of obtaining the abovementioned parameters, especially from borehole loggings, must be kept in mind when evaluating the results of the study, so that critical assessments and more extensive site explorations can be carried out to assess the parameters with more certainty.

The *Q*-system does not give direct consideration to joint orientation (Palmstrom & Broch, 2006; Steiner & Einstein, 1980), although it has been commented that the other joint parameters play a greater role than joint orientation when rock supports are determined, possibly because excavations are normally adjusted to avoid the maximum effect of unfavorably oriented major joints (Singh & Goel, 1999).

The active stress quotient (Jw/SRF) has also been pointed out to be complicated. SRF, in particular, may have the same value in different types of ground, which would not be a problem if other parameters sufficiently differentiate the ground. However, in the case of swelling and squeezing ground (SRF = 5 to 15), the support requirements need to be substantially different, but the SRF and the other five parameters can be essentially the same with other less complex ground (Steiner & Einstein, 1980). Palmstrom & Broch (2006) highlighted that it is their impression that SRF seemed a sort of "correction factor" or "fine-tuning factor", rather than a factor expressing active stresses that will give an appropriate rock support.

It should also be noted that even though the Q-value scale in Fig. 3.4 goes up to a "Span or height in m / ESR" value of 100 and a Q value of 1000, it is unlikely that the full range of the figure can be used, simply because the empirical studies used as a basis for the Q-system do not cover cases that have such extreme values. It has also been argued, through a critical evaluation of the parameters, that the Q-system is only good for the range approximately $0.1 < Q < 40$, and a span of 3 to 30 meters (Palmstrom & Broch, 2006), illustrated in Fig. 3.5.

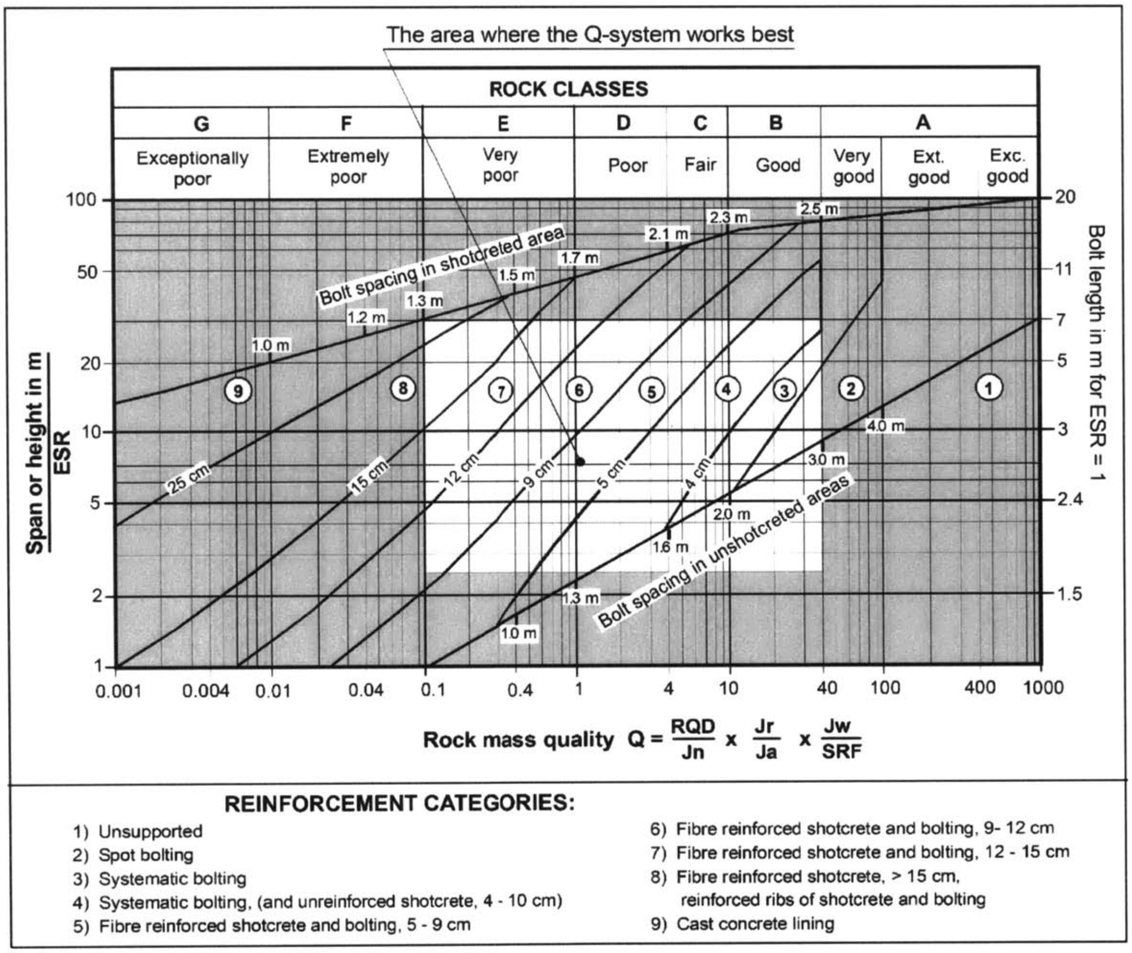


Fig 3.5: Limitations of the Q support chart. Outside the unshaded area supplementary methods/ evaluations/ calculations should be applied.

(based on Palmstrom et.al., 2002, reproduced from Palmstrom and Broch, 2006)

From the discussion of the RQD method and the *Q*-system, it can be seen that even though empirical rock support design methods are useful for preliminary evaluation of the rock support required for a particular rock mass, application of each method has its limitations. These limitations should be understood by studying the assumptions of the method, and its application must be accompanied by a sound judgement, practical experience, and a thorough, detailed consideration of ground conditions (Steiner & Einstein, 1980).

CHAPTER FOUR

COST OF CAVERN CONSTRUCTION

4.1 INTRODUCTION

Central to the success of infrastructure projects is cost estimation. In underground infrastructure projects, cost estimation can be said to be of greater importance due to the variability of the ground conditions leading to greater uncertainty. As mentioned before, for cavern designs in rocks, these uncertainties are especially relevant because, frequently, the actual geological condition is only known during construction when the ground is exposed.

The Decision Aids for Tunnelling (DAT), a computer-based tool which has been widely adopted and applied, compute tunnel construction cost, time and the resources employed to reflect uncertainties in the information used, including geologic and geotechnical ground conditions (Einstein, 1999; Min, Kim, Lee, & Einstein, 2008). Similarly in cavern construction, poor ground conditions will affect rock support amongst other aspects and hence cost. Distributions of the rock support estimates will reflect the uncertainties in the ground conditions, and will be primarily employed in the thesis as an indicator of cavern feasibility.

This chapter describes the assumed cavern size, the construction method and rock support components included in the cost estimation. By comparing construction cost of actual caverns, an estimate for the rock support components is obtained. Based on the assumptions of the cavern size, ground classes are established using the RQD method and *Q*-system, and the cost of the rock support derived from these empirical rock support methods for each ground class can be estimated.

4.2 CAVERN ASSUMPTIONS AND CONSTRUCTION

For the purpose of this thesis, a cavern 20m wide, 30m in height and 100m in length is considered, a simplified representation of the cavern is shown in Fig 4.1. Excavation of the cavern will be assumed to be conducted by drill and blast methods, and is assumed to be constructed with one heading and two benches, each at 10m in height, also shown in Fig 4.1.

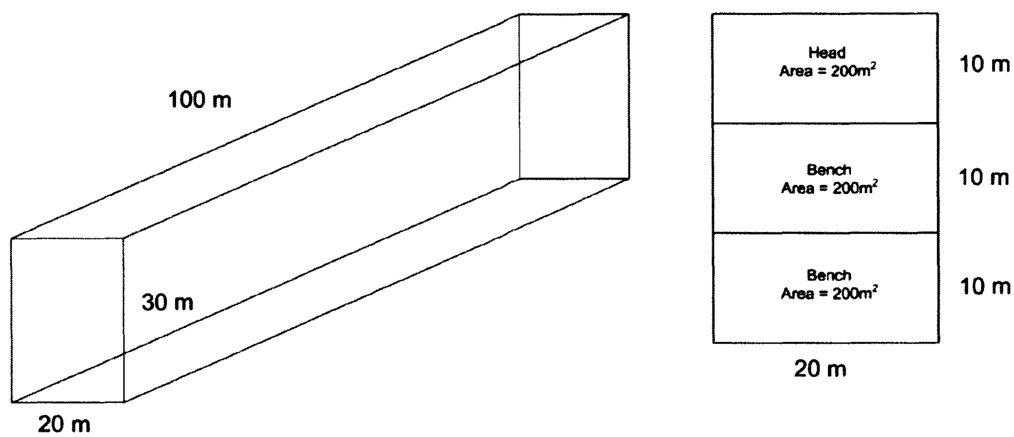


Fig 4.1: Simplified representation and cross-section of cavern of interest

The primary support requirements are assessed using the empirical rock support design methods, namely RQD approach and the *Q*-system, as discussed in Chapter Three. The rock support system derived from the two methods will include the following rock support elements:

(1) Fiber reinforced shotcrete;

(2) Rock bolts; and

(3) Grouting.

As the thesis is a feasibility study to identify suitable locations for housing caverns without a specific cavern scheme in mind, the estimated cost in this chapter is not the total construction cost of the cavern, but only the sum of the cost of these rock support elements, and the excavation cost of the cavern. Depending on the cavern scheme and usage, a more detailed cost estimate including the access tunnels and shafts, transportation of equipment into and mucking out of the cavern at depth, land acquisition, operating cost, and other cost components relating to the complexity of the cavern must be considered.

4.3 COST COMPARISON OF ROCK SUPPORT ELEMENTS

Cost information of rockbolts and fiber reinforced shotcrete can be found in the existing literature. The cost of rockbolts in Canada is reported in Goris, Nickson, & Pakalnis (1994) and Soni (2000), and the cost of rockbolts and reinforced shotcrete in Sweden can be found in Almefeldt (2012). The cost information for rockbolts, fiber reinforced shotcrete, grouting and excavation cost in Austria and in Singapore are also obtained from cavern construction projects in the respective countries. With this information, the cost of the rock support elements are compared so that an indicative cost of rock support can be estimated for the purpose of this thesis, as shown in Table 4.1. Although the cost information has been reported in different currencies and different years, the cost information referred to in this section has been converted to reflect Singapore Dollars in 2015 (SGD$'15)[1].

Cost differences can be observed in different countries, which is not surprising given that cost is affected by a wide range of factors, including the method of construction, type of equipment used, and labor cost. From Table 4.1, it can be observed that the rockbolt cost per meter is about the same between Canada and Sweden, but cost less in Austria and more in Singapore. The difference in cost for the fiber reinforced shotcrete is significant, with the cost in Austria at $58 per square meter of 10 cm thick shotcrete, compared to $107 in Singapore and $143 in Sweden. For grouting, the cost in Austria is $185 per square meter, while in Singapore, it only cost about $64.

The cost of excavation generally differs between heading excavation and bench excavation, with headings excavation costing more. The cost of cavern excavation decreases with the quality of the ground. For the project in Austria, the quality of the ground is indicated through the length of the heading that was excavated in one cycle, i.e. a shorter heading implies poorer ground conditions; for the project in Singapore, the quality of the ground is indicated through a straightforward classification, i.e. Ground Class 1 (GC1) is the best quality ground, and Ground Class 5 (GC5) is the worst. The cost of excavation per cubic meter for headings and benches in different ground conditions, does not differ too much between Austria and Singapore. In general, the cost derived from the project in Singapore will be used in the estimation of the cost of the rock support system, in order to reflect local cost conditions for the construction of the cavern.

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[1] This is done by converting the reported currency to the equivalent Singapore Dollar in the reported year through the exchange rate, and adjusted to the annual average rate of inflation through the years to SGD$'15.

4.4 COST ESTIMATION OF ROCK SUPPORT SYSTEMS FOR DIFFERENT GROUND CLASSES

Required rock support systems need to be determined to estimate cost. Minimum rockbolt length is determined empirically, from the recommendation of Lang (1961), where the minimum rock bolt length is taken to be 0.25B for spans of B ranging from 18 to 30 meters, and 0.2H for height of H greater than 18 meters. For our case, the minimum rock bolt length will hence be 6 meters. As there is no information to determine the amount of grouting required, it is assumed that the volume of grouting required based on conductivity tests for all ground classes is 80% of the surface area, and a meter in depth. The rest of the rock support components required for the rock support system can be derived from the empirical rock support methods, the RQD method and *Q*-system.

Based on Table 3.2 for the RQD method, the ground classes and the associated rock support systems are derived and shown in Table 4.2. As the RQD method is empirically defined for tunnels of about 6 to 12 meters in diameter, the ground classes has been modified to take into account this limitation. Poor ground class is defined to be less than 60, instead of 25 to 50; Fair ground class is defined to be 60 to 80 instead of 50 to 75; Good ground class is defined to be 80 to 90 instead of 75 to 90; Excellent ground class remains as greater than 90. Similarly, for the *Q*-system, the ground classes and the associated rock support system can be derived from Fig 3.4, using an ESR value of 0.8, shown in Table 4.3. For the same ESR value of 0.8, using the chart for Q-system, for a span of 20 meters, the required bolt length can be found to be between 5 and 7 meters, which agrees with the 6 meters determined earlier. The cost estimates for each ground class are also shown in Table 4.2 and 4.3. It can be noted that the cost for Ground Classes 1 to 3 are comparable between the two empirical methods, while for Ground Classes 4 and 5, the cost is significantly greater.

A detailed table of the cost estimation for the rock support system and excavation of the cavern for Ground Class 4 of the *Q*-system is shown in Table 4.4, to itemize the components that are included in the estimation, and to illustrate the methodology adopted in estimating cost for the rock support systems. It can be seen from Table 4.4 that the cost estimation depends on the cavern assumptions made earlier in the chapter, including cavern geometry and construction method, and must be modified when these assumptions change.

Table 4.1: Cost comparison of rock support elements across countries (page 1 of 2)

Rock bolt cost per meter		**Country**	**SGD$'15**
Soni (2000)	Cable bolt 4m	Canada	$28.30
Goris et.al. (1994)	12.2m twin strand cable bolt	Canada	$28.60
Almefelt (2012)	Bolt 4m Combi-coated	Sweden	$28.61
Austrian Project	IBO Anker / SWELLEX Anker	Austria	$17.57
Singapore Project	Rock bolts (5m)	Singapore	$44.25
Fiber reinforced shotcrete per square meter		**Country**	**SGD$'15**
Almefelt (2012)	Shotcrete 10 cm with fibre	Sweden	$143.05
Austrian Project	Shotcrete 5cm (Sidewalls)	Austria	$20.39
	Shotcrete 10cm (Sidewalls)		$40.99
	Shotcrete 15cm (Sidewalls)		$44.60
	Shotcrete 20cm (Sidewalls)		$59.47
	Shotcrete 25cm (Sidewalls)		$74.34
	Shotcrete 30cm (Sidewalls)		$89.21
	Shotcrete 5cm (face)		$29.31
	Shotcrete 10cm (face)		$58.41
	Shotcrete 5cm (invert)		$11.68
	Shotcrete 10cm (invert)		$23.15
	Shotcrete 15cm (invert)		$34.83
	Shotcrete 20cm (invert)		$46.30
Singapore Project	Shotcrete 5cm (crown/sidewall inc. fibre)	Singapore	$53.43
	Shotcrete 8cm (crown/sidewall inc. fibre)		$85.45
	Shotcrete 10cm (crown/sidewall inc. fibre)		$106.87
	Shotcrete 15cm (crown/sidewall inc. fibre)		$160.30
	Shotcrete 20cm (crown/sidewall inc. fibre)		$213.62

Table 4.1 (cont'd): Cost comparison of rock support elements across countries (page 2 of 2)

Grouting per cubic meter		**Country**	**SGD$'15**
Austrian Project	**Normal Grouting**	Austria	$184.79
Singapore Project	**Grouting and Drilling works**	Singapore	$63.50
Excavation per cubic meter		**Country**	**SGD$'15**
Austrian Project	Heading (>3.0-3.5m)	Austria	$30.80
	Heading (>2.5-3.0m)		$42.48
	Heading (>2.0-2.5m)		$53.95
	Heading (>1.5-2.0m)		$65.63
	Heading (>1.0-1.5m)		$77.31
	Heading (<=1.0m)		$108.11
	Bench + Invert (>3.5-4.0m)		$15.51
	Bench + Invert (>3.0-3.5m)		$23.15
	Bench + Invert (>2.0-3.0m)		$27.82
	Bench + Invert (>1.0-2.0m)		$33.98
	Bench + Invert (<=1.0m)		$46.30
	Invert		$16.99
Singapore Project	Heading (GC1)	Singapore	$29.42
	Heading (GC2)		$30.60
	Heading (GC3)		$34.13
	Heading (GC4)		$51.79
	Heading (GC5)		$69.44
	Bench(GC1)		$23.54
	Bench (GC2)		$25.89
	Bench (GC3)		$29.42
	Bench (GC4)		$44.72
	Bench (GC5)		$62.38

Table 4.2: Modified Ground Classes from RQD method and associated rock support system

GC	*Modified RQD range*	*Rock Classes*	*Reinforcement Categories*	*Cost/ m^3 (SGD$'15)*
5	-	Fault zone	20cm fibre reinforced shotcrete 6m bolts at spacing 0.6m	156.75
4	<60	25 to 50 (Poor)	15.2cm fibre reinforced shotcrete 6m bolts at spacing 0.9m	89.40
3	60 to 80	50 to 75 (Good)	12.7cm fibre reinforced shotcrete 6m bolts at spacing 1.5m	51.98
2	80 to 90	75 to 90 (Very good)	7.6cm fibre reinforced shotcrete 6m bolts at spacing 1.8m	38.72
1	>90	>90 (Excellent)	5cm fibre reinforced shotcrete 6m bolts at spacing 2.3m	30.40

Table 4.3: Ground Classes from *Q*-method and associated rock support system

GC	*Q-value range*	*Rock Classes*	*Reinforcement Categories*	*Cost/ m^3 (SGD$'15)*
5	-	Fault zone	20cm fibre reinforced shotcrete 6m bolts at spacing 1.0m	93.81
4	<0.4	G to F (Ext. to Exc. Poor)	15cm fibre reinforced shotcrete 6m bolts at spacing 1.2m	69.99
3	0.4 to 4	D to E (Poor to Very Poor)	12cm fibre reinforced shotcrete 6m bolts at spacing 1.5m	50.98
2	4-40	B to C (Good to Fair)	9cm fibre reinforced shotcrete 6m bolts at spacing 2.1m	37.82
1	>40	A (Very Good – Exc. Good)	5cm fibre reinforced shotcrete 6m bolts at spacing 2.5m	29.37

Table 4.4: Computation of cost of rock support per m^3 for Ground Class 4 of *Q*-system, including grouting and excavation cost

Ground Class:	4	
Empirical Method:	Q-System	
1) Rock bolt cost	= number of bolts used * length of bolts * cost of rock bolt per m	
	= (Total surface area/area per rock bolt) * length of bolts * cost of bolt / m	
	= ((20 + 30 * 2) * 100 / (1.2 * 1.2)) * 6 * 44.25	
	=	**$ 1,475,125.47**
2) Shotcrete cost	= Total surface area * cost of shotcrete per m2 (15cm)	
	= (20 + 30 * 2) * 100 * 160.30 =	**$ 1,282,417.59**
3) Grouting cost	= 80% * Total surface area * 1m depth * cost of grouting per m3	
	= 0.8 * (20 + 30 * 2) * 100 * 63.50 =	**$ 406,400.00**
4) Excavation cost	= Volume of head * cost of excavating heading per m3 (GC4)	
	+ Volume of benches * cost of excavating benches per m3 (GC4)	
	= 200 * 100 * 51.79 + 400 * 100 * 44.72	**$ 1,035,756.21**
Total Cost		$ 4,199,699.27
Total Volume (m3)		60000
Cost per m3		**$ 69.99**

Cost estimates have been done in the previous studies of cavern feasibility in Singapore. Zhao et.al (1996) indicated the cavern construction cost with cross-section of 415 m^2 (24 by 19 m), reproduced in Table 4.5. They also reported that, for a cavern scheme used for oil storage, the estimated cost is about $125 per cubic meters. Zhao et. al. (1999) gave indicative unit cost for cavern excavation and support in Jurong Formation, reproduced in Table 4.6. Taking into account inflation rate, the SGD$34 to $85 estimate in Table 4.5 is comparable to the estimate in Table 4.2 and 4.3, while the SGD$50 to $120 estimate is greater. As the cost estimate in Table 4.6 is based on a cavern scheme including shafts and access tunnels, it is reasonable that the cost estimate is greater than the estimates in this thesis.

Table 4.5: Cavern construction cost with cross section of 415m^2 (24 x 19 m) (reproduced from Zhao et.al., 1996)

	Cost per m³ (Singapore Dollar)		
Rock quality	**Excavation**	**Rock support**	**Total cost**
Good	30	4	34
Fair	33	15	48
Poor	54	31	85

Table 4.6: Indicative unit cost for cavern excavation and support in Jurong Formation (reproduced from Zhao et.al., 1999)

Rock Mass Quality		Suitability of Rock Mass for Cavern Development	Indicative Unit Cost for Cavern Excavation & Support S$/m³
Q-value	**Description**		
0.01 - 1	Extremely poor to very poor	Rock mass generally unsuitable for cavern construction	-
1 - 4	Poor	Cavern development technically feasible, although rock support costs relatively high	80 - 120
4 - 10	Fair	Suitable for cavern development	60 - 90
10 - 40	Good	Highly suitable for cavern development	55 - 70
40 - 1000	Very good to extremely good	Excellent for cavern development	50 - 60

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CHAPTER FIVE

PROBABILISTIC ASSESSMENT OF ROCK PROPERTIES

5.1 INTRODUCTION

In most geotechnical engineering designs, the complex nature of the conditions underground makes it inevitable that exploration programs and efforts to characterize ground properties are unable to establish ground properties with certainty. Also, ground investigations in rock are expensive, and as a result rock mass properties often rely on a few observations over a large volume of rock mass. In the empirical methods discussed in Chapter Three, like in the *Q*-system, single values are given to each input parameter, producing a *Q*-value for obtaining rock support design. Application of the empirical methods in such a manner cannot capture the variability associated with the rock mass (Kim & Gao, 1995).

Due to the stochastic nature of the geometry of the rock masses and the variability of their mechanical properties, uncertainties in the rock mass properties are commonly dealt with using stochastic models (Dershowitz & Einstein, 1988; Kim & Gao, 1995; Sari, Karpuz, & Ayday, 2010). In this chapter, for the area of interest in South-Western Singapore, a probabilistic analysis will be carried out on the RQD and the other input parameters for the *Q*-system that are obtained from boreholes placed within the area of interest, accounting for both depth and spatial variability. In particular, to account for spatial variability in geology, the rock mass properties will be analyzed based on interpreted subsurface profiles provided by SGO.

5.2 DATA FROM BOREHOLES

As mentioned in Chapter One, the SGO carried out extensive geological surveying and investigation work in the South-Western and North-Eastern parts of Singapore, including surface geological mapping, drilling investigations to 200m depth with in-situ testing, rock core testing and geophysical surveys. The approximate area in SW Singapore where investigation works are carried out is shown in Fig 5.1. The zone of interest for this thesis lies is in this approximate area, where the SGO placed four boreholes for site investigation – three 200m vertical boreholes and a 200m inclined borehole, with dip direction/dip of 65/70, shown in Fig 5.2. Fig 5.2 includes an approximate scale.

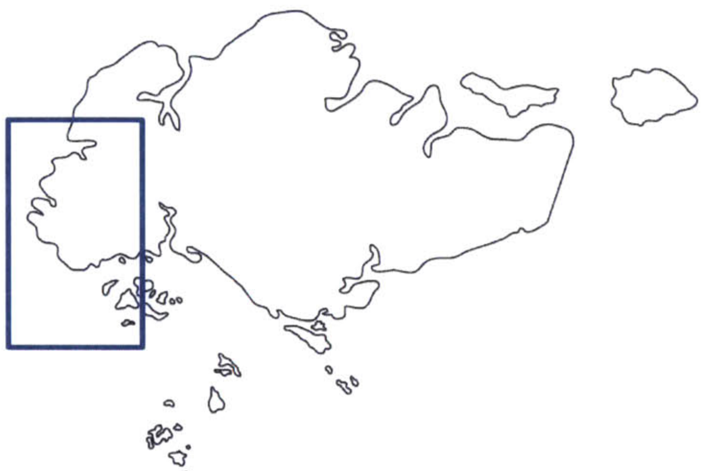

Fig 5.1: Approximate area of SW Singapore where investigation works are carried out

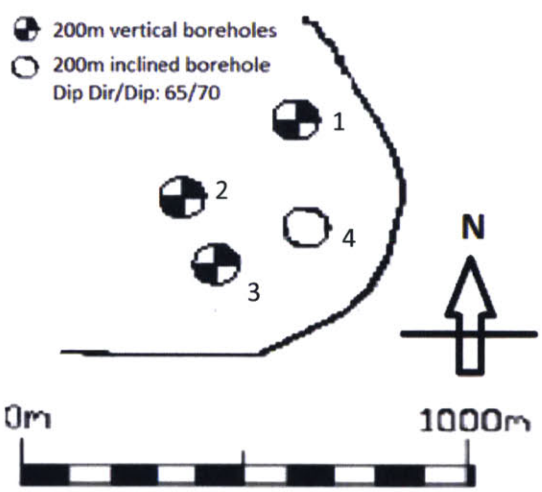

Fig 5.2: Map of borehole locations for area of interest (source: SGO)

Rock core observations were carefully logged by engineering geologists, including rock core features like RQD, discontinuity orientation, aperture, infilling and weathering and alteration properties. Borehole Image Processing, i.e. the continuous scanning and processing of borehole wall images was also conducted during the boring. To derive rock support using the *Q*-system, the engineering geologists logging the boreholes also provided an interpretation of the parameters Jn, Jr, Ja and Jw. Together with the RQD from the boreholes, these parameters will form the basis of the probabilistic assessment carried out on the rock mass properties.

5.3 INTERPRETED SUBSURFACE PROFILES

For the area of interest, SGO has provided the interpreted subsurface profiles of two cross-sections, labelled cross section A and cross section B, as shown in Fig 5.3. The interpreted subsurface profiles of cross-sections A and B are shown in Fig 5.4 and Fig 5.5 respectively.

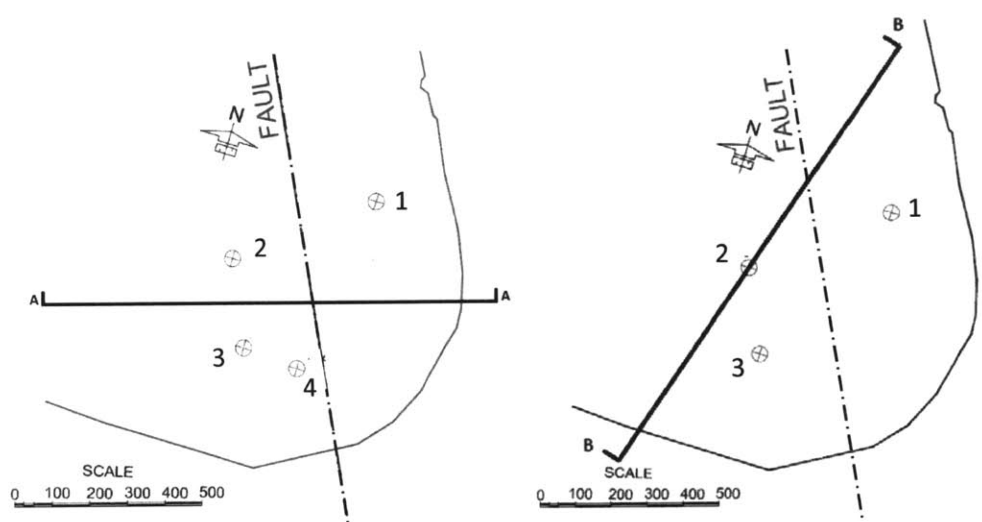

Fig 5.3: Cross-sections with interpreted subsurface profiles in area of interest (source: SGO)

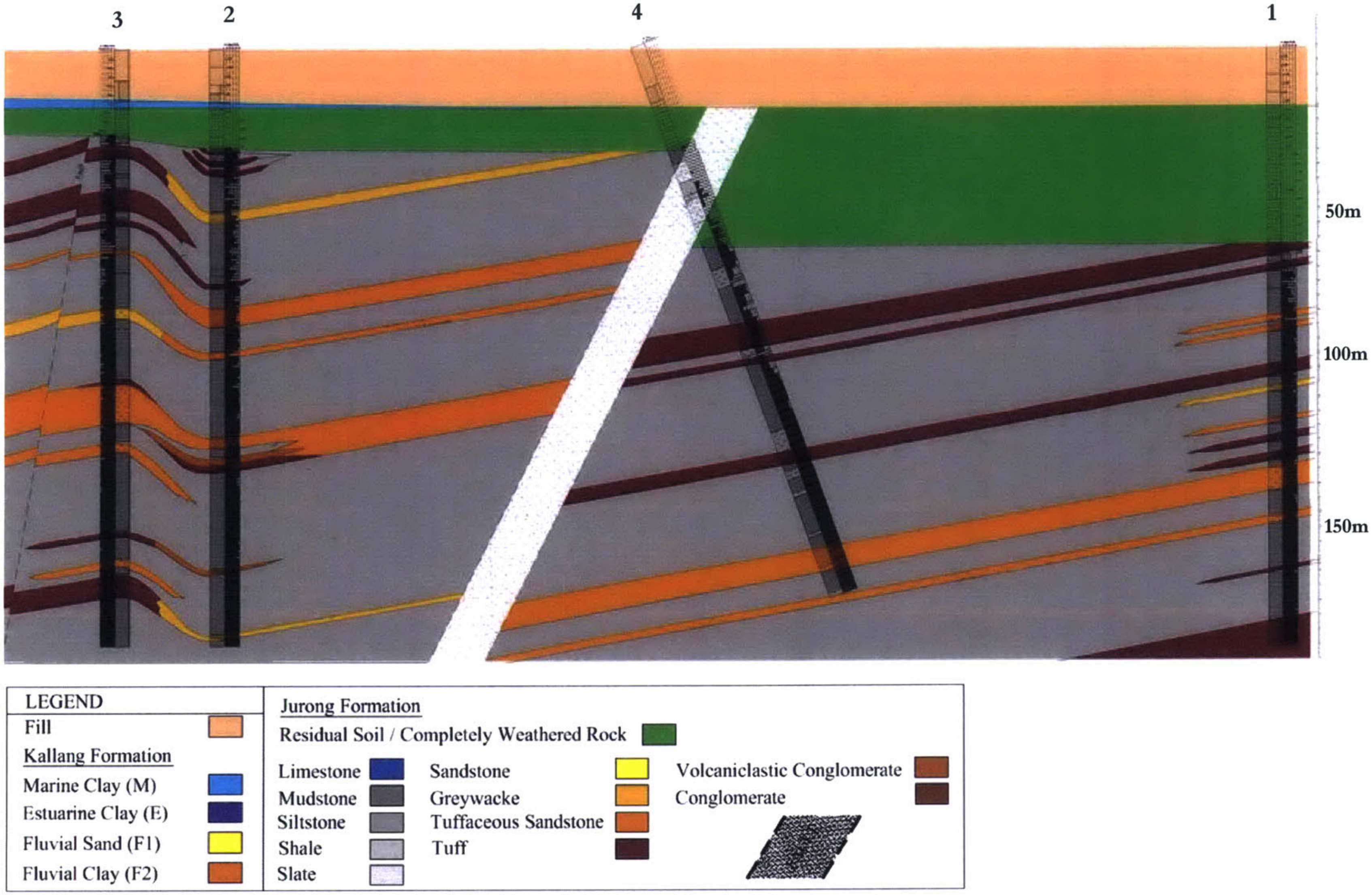

Fig 5.4: Interpreted subsurface profile across Section A-A (source: SGO)

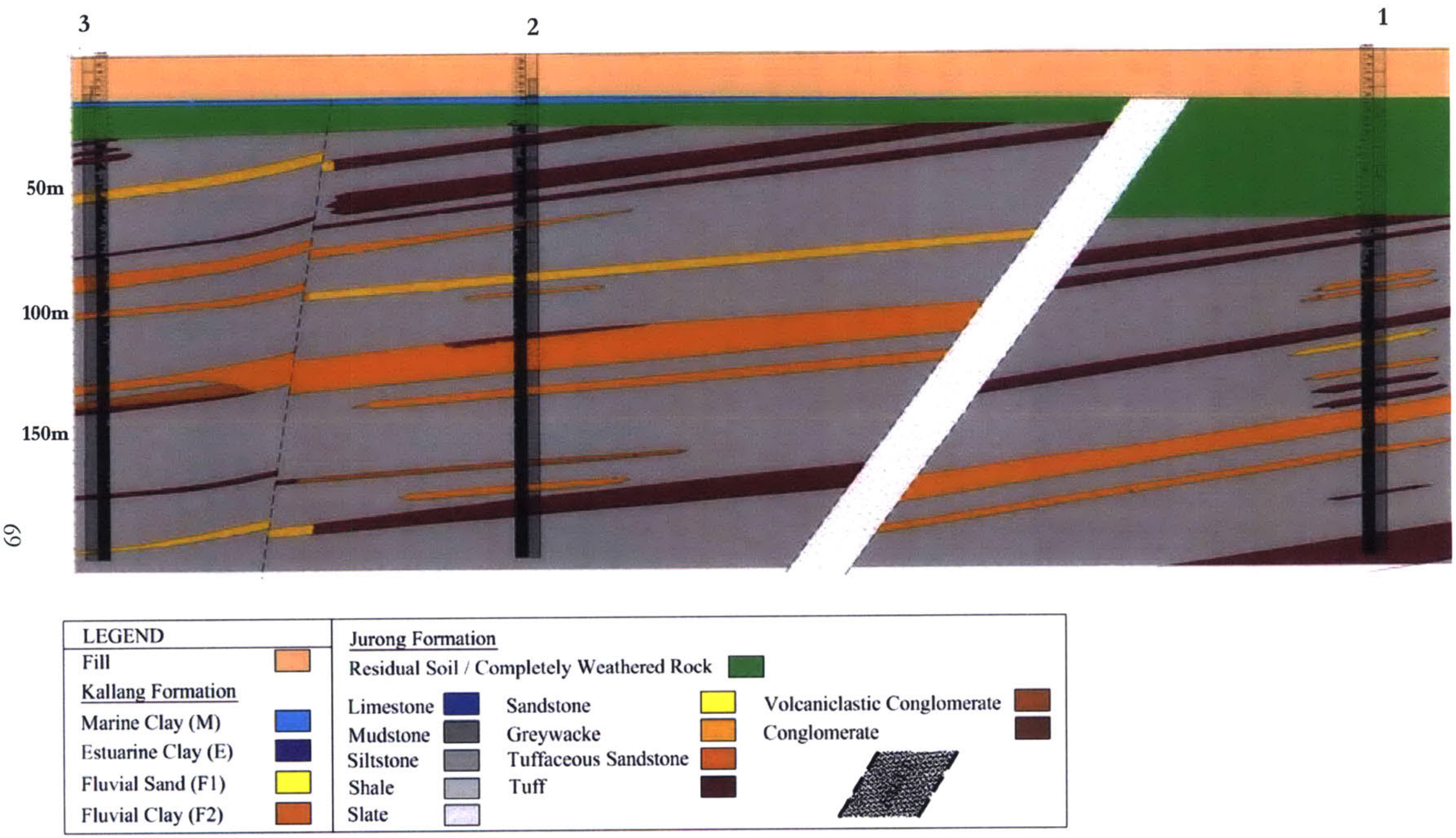

Fig 5.5: Interpreted subsurface profile across Section B-B (source: SGO)

5.4 DEPTH VARIABILITY

To capture the variability of the *Q*-value and the RQD with depth, RQD and the other parameters needed to determine the *Q*-value logged in the boreholes are analyzed separately in intervals of 40 meters, i.e. the rock mass property at a depth of 80 meters will be based on the analysis of the RQD and other *Q*-parameters that are logged in the boreholes from depth 60 meters to 100 meters. Depth of the inclined borehole is corrected based on its dip angle. The 40 meters interval is chosen to encompass the assumed cavern in this analysis, which has a height of 30 meters.

5.4.1 PROBABILISTIC ASSESSMENT OF RQD BY DEPTH

For each interval of 40 meters, the RQD logged from the boreholes is plotted on a histogram. An example for the interval 60 to 100 meters is shown in Fig 5.6. The observed frequency can be transformed into discrete probabilities, and then classified into the ground classes proposed in Table 4.2. For example, of the 156 observations for the depth interval of 60 to 100 meters, 22 observations showed RQD values from 81 to 90 (GC2), yielding a probability of 14%. A summary of the statistics for the RQD values, including the probability of each ground class, is shown in Table 5.1. A plot of the mean, median and mode is shown in Fig 5.7.

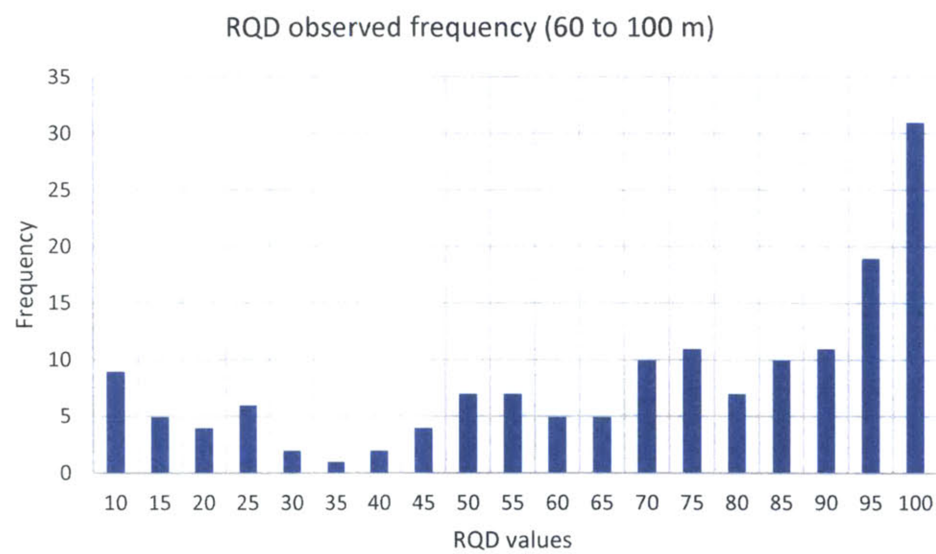

Fig 5.6: Observed frequency of RQD values (60 to 100m)

5.4.2 PROBABILISTIC ASSESSMENT OF *Q*-VALUE BY DEPTH

Similarly, besides the RQD, all other parameters needed to obtain the *Q*-value are plotted on histograms, to obtain the probability of each value of the parameters. Using the formulae to calculate *Q* as discussed in Chapter Three, the probability of the different ground classes proposed in Table 4.3, based on the calculated *Q*-values can be obtained. A summary of the statistics for the *Q*-values, including the probability for each defined ground class, is shown in Table 5.2. A plot of the mean, median and mode is also shown in Fig 5.8. It is observed that the mode of the *Q*-value increased drastically from depth 170 meters to 182.5 meters. By observing the range of values obtained for the different parameters needed to calculate *Q*, it was found that the *Q*-value is particularly sensitive to the value of Jn.

Comparing the statistics of the RQD and the *Q*-value, the general trend that the ground improves with depth agrees between the two methods. However, it is evident that the ground class distribution differs. The most evident difference is that the generated RQD suggests greater occurrence of GC4 at lower depths, while at greater depths, RQD predicts GC1 with higher probability. The expected cost at different depths based on the probabilities of ground occurrence for both the RQD approach and the *Q* method are shown in Fig 5.9, which illustrates ground improvement with depth. However, the different probabilities of the various ground classes are not shown in Fig 5.9, and hence planners will not be able to know the variability and uncertainty that they face when they select the depth to locate caverns.

5.4.3 DEPTH SELECTOR MAP

To provide planners with the variability information required to make decision on the depth where the cavern is to be located, based on the probabilistic analysis and assessment of the RQD and *Q*-values derived from the borehole logs, the probability of each ground class at each depth are shown in Fig 5.10 and Fig 5.11 for RQD and the *Q*-system respectively, called Depth Selector Maps (DSM). In the DSM, the vertical scale is the depth in meters, and the horizontal scale is the cost of rock support, grouting and excavation per cubic meter. The probability of each ground class at each depth, and its corresponding cost can be easily read from the DSM by making use of the color scale, as it is done in a contour map. For example, the RQD DSM, it can be seen that if a planner wants at least 80% probability of GC1 and GC2, the cavern will be located a depth of at least 110 meters.

The horizontal axes of the DSMs capture the idea that the cost per cubic meter does not increase linearly across the different ground classes. The cost increase from GC1 to GC2 is less significant than the cost increase from GC2 to GC3, which in turn is less significant than the cost increase from GC3 to GC4. It can also be noted that even though the two DSMs for RQD and *Q*-system appear quite different, the depths selected for locating caverns using the two DSMs generally agree well. For about 70% chance of locating the cavern in at least GC2, both DSMs suggest at least a depth of 80 meters. Also, for both DSMs, the cavern will be located a depth of at least 110 meters for at least 80% probability of GC1 and GC2, and to achieve a greater than 90% of probability of GC1 and GC2, at least 140 meters depth is required.

The simplified statistical approach to obtain the expected probability for each ground class for the *Q* method, used in this probabilistic assessment for depth variation serves as an indicative comparison with the RQD approach. However, it must be noted that the approach is based on the assumption that the parameters used to obtain the *Q*-value are independent of each other, which is not the case. For instance, the parameters are all likely to be dependent on the nature of discontinuities and rock strength. To capture the dependency of the parameters, the parameters need to be examined in much greater detail, and more sophisticated statistical methods need to be employed, which is beyond the scope of this thesis. Therefore, further probabilistic assessment of the rock mass will be solely based on the RQD approach.

Table 5.1: Statistics of generated RQD value and probability of ground class occurrence

			Statistics			Probability			
Depth range (m)		Mean depth (m)	Median	Mean	Mode	GC4	GC3	GC2	GC1
40	80	60	65.00	56.67	10.00	45%	31%	8%	16%
50	90	70	70.00	64.12	100.00	37%	28%	10%	25%
60	100	80	80.00	70.72	100.00	29%	24%	14%	33%
70	110	90	85.00	75.56	100.00	24%	21%	13%	41%
80	120	100	95.00	81.76	100.00	17%	17%	14%	52%
90	130	110	95.00	87.05	100.00	10%	14%	15%	61%
100	140	120	100.00	91.06	100.00	8%	9%	11%	72%
110	150	130	100.00	95.29	100.00	1%	8%	10%	81%
120	160	140	100.00	97.62	100.00	0%	4%	7%	89%
130	170	150	100.00	98.43	100.00	0%	2%	5%	93%
140	180	160	100.00	98.36	100.00	0%	3%	4%	93%
150	190	170	100.00	99.12	100.00	0%	1%	2%	97%
160	205	182.5	100.00	99.21	100.00	0%	1%	2%	97%

Table 5.2: Statistics of generated *Q*-value and probability of ground class occurrence

			Statistics			Probability			
Depth range (m)		Mean depth (m)	Median	Mean	Mode	GC4	GC3	GC2	GC1
40	80	60	3.39	6.23	1.32	5%	52%	42%	1%
50	90	70	5.28	9.50	2.64	3%	40%	55%	3%
60	100	80	7.26	11.56	8.80	1%	31%	65%	3%
70	110	90	8.80	15.05	17.60	1%	23%	70%	6%
80	120	100	14.08	22.65	17.60	0%	13%	72%	15%
90	130	110	16.72	24.45	17.60	0%	10%	72%	17%
100	140	120	17.60	27.89	35.20	0%	8%	69%	22%
110	150	130	23.47	28.95	46.93	0%	5%	69%	26%
120	160	140	23.47	26.41	46.93	0%	4%	69%	27%
130	170	150	23.47	34.60	46.93	0%	3%	61%	36%
140	180	160	26.40	39.61	46.93	0%	2%	59%	39%
150	190	170	35.20	52.15	46.93	0%	1%	50%	49%
160	205	182.5	46.93	60.61	140.80	0%	0%	46%	54%

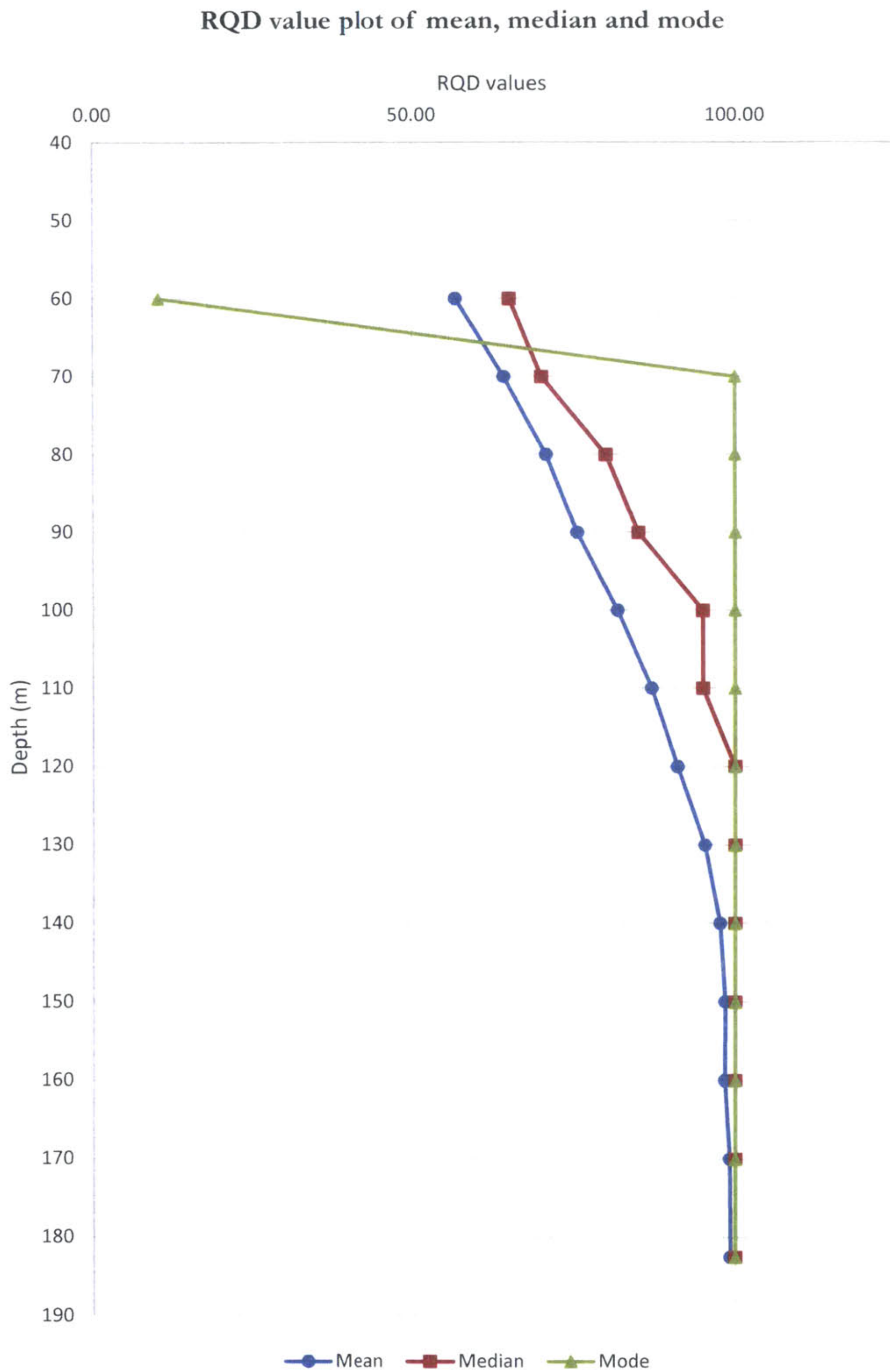

Fig 5.7: RQD value plot of mean, median and mode

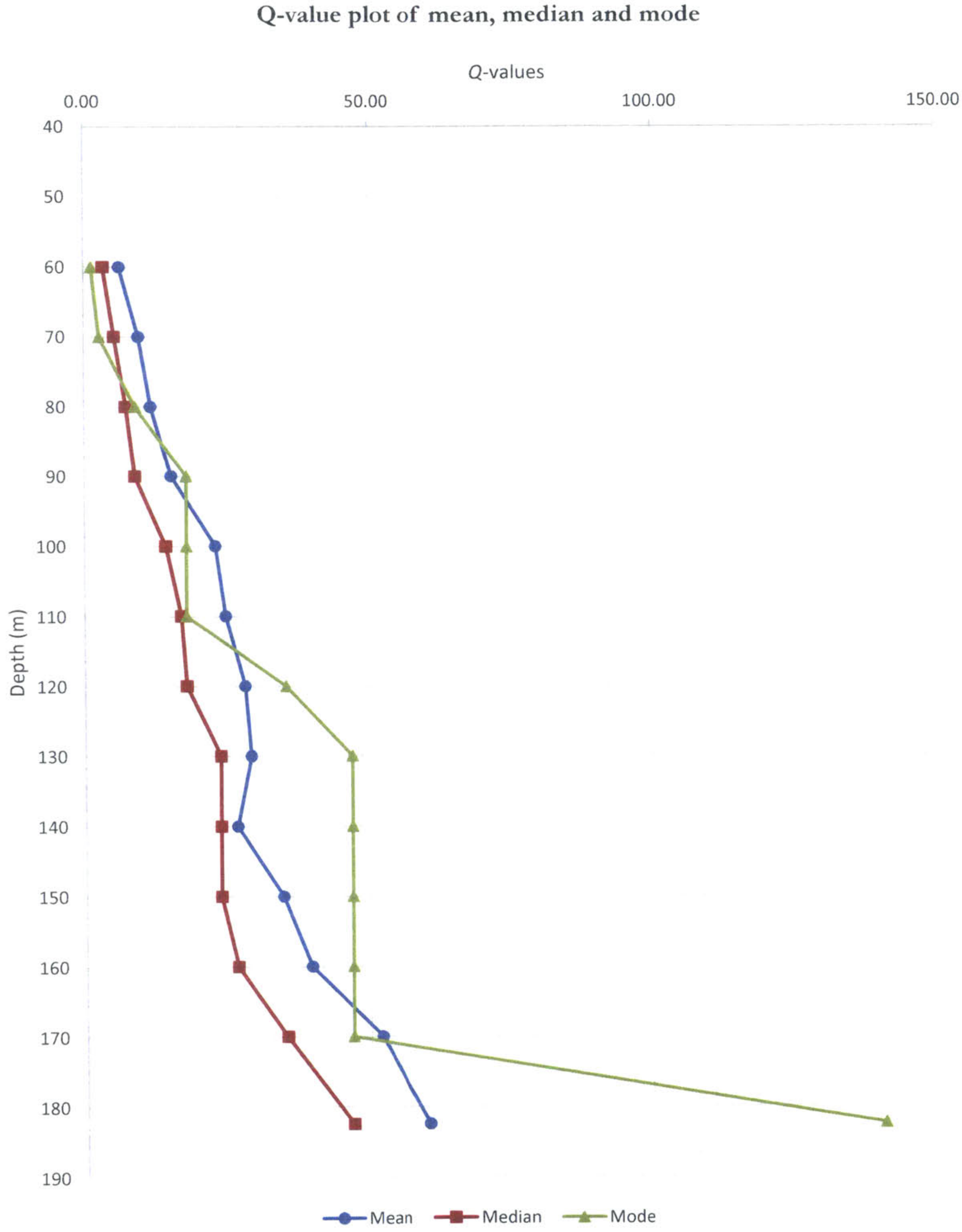

Fig 5.8: *Q*-value plot of mean, median and mode

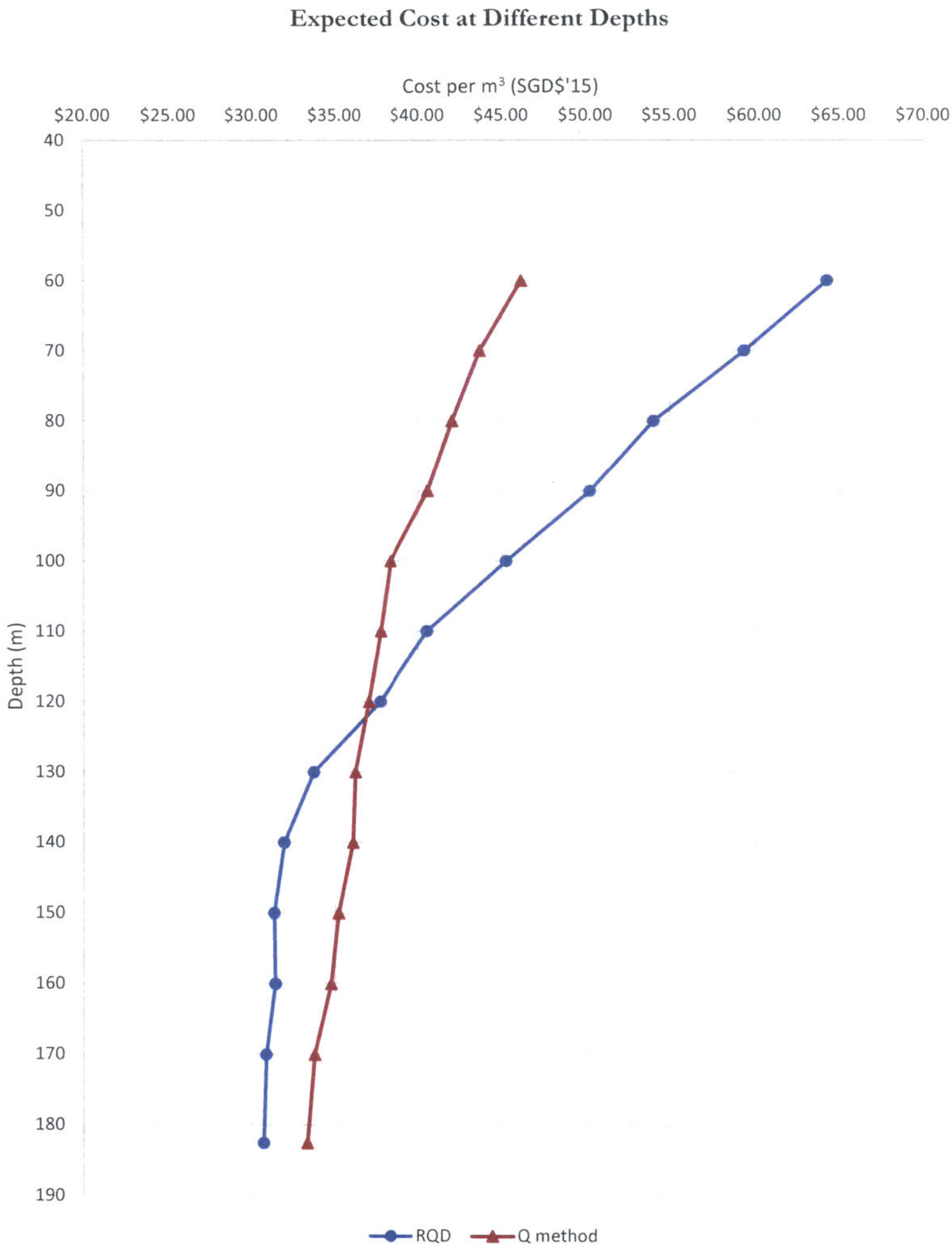

Fig 5.9: Expected cost at different depths for both RQD approach and *Q* method

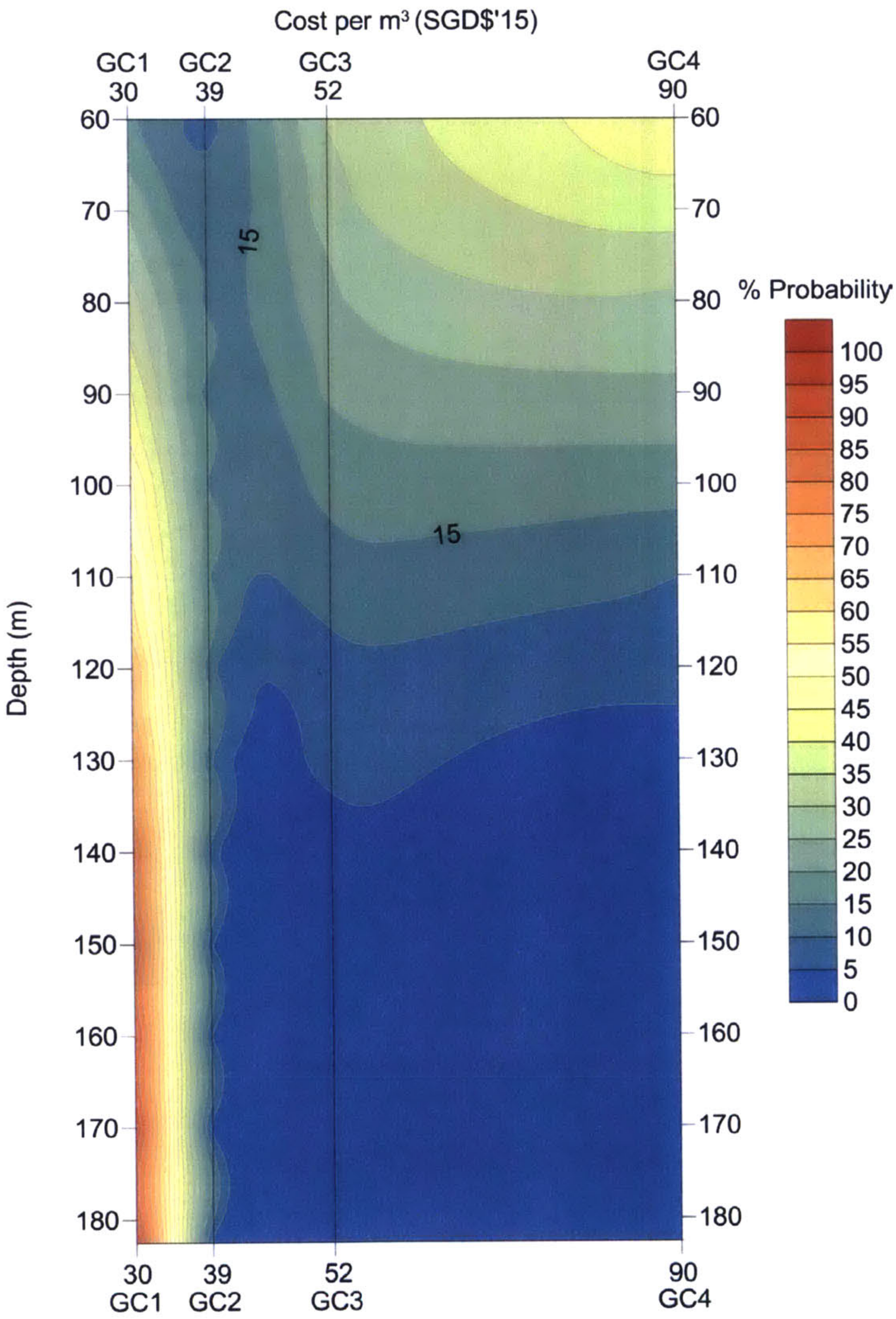

Fig 5.10: Depth Selector Map (RQD)

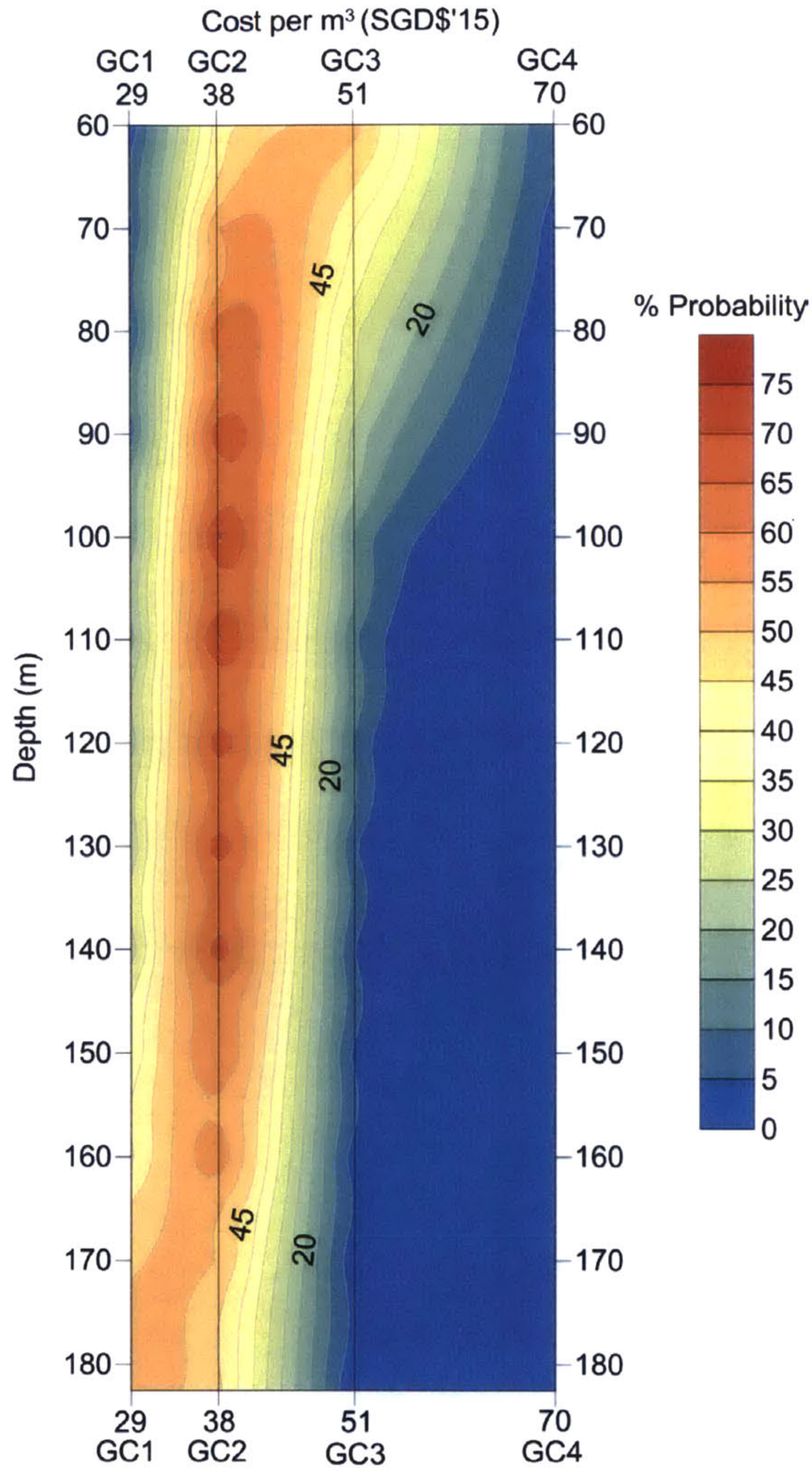

Fig 5.11: Depth Selector Map (*Q*-system)

5.5 SPATIAL VARIABILITY

Spatial variability can be assessed by classifying the area of interest into different zones that could possibly differ based on the interpreted subsurface profile of the area. From the interpreted subsurface profiles, shown in Fig 5.4 and 5.5, the area can be divided into three zones. The division and naming of the three zones are shown in Fig 5.12a and Fig 5.12b. There is insufficient information to determine the size of the fault zone (Zone 3) and its effects with depth for an effective assessment, and hence will not be included in this spatial variability assessment. RQD values obtained from boreholes 2 and 3 will be assessed to determine the ground class variability at different depths for Zone 1. Similarly, RQD values obtained from boreholes 1 and 4 will be assessed to determine the ground class variability at different depths for Zone 2. The results for the two zones will be compared to determine the degree of spatial variability across these two zones.

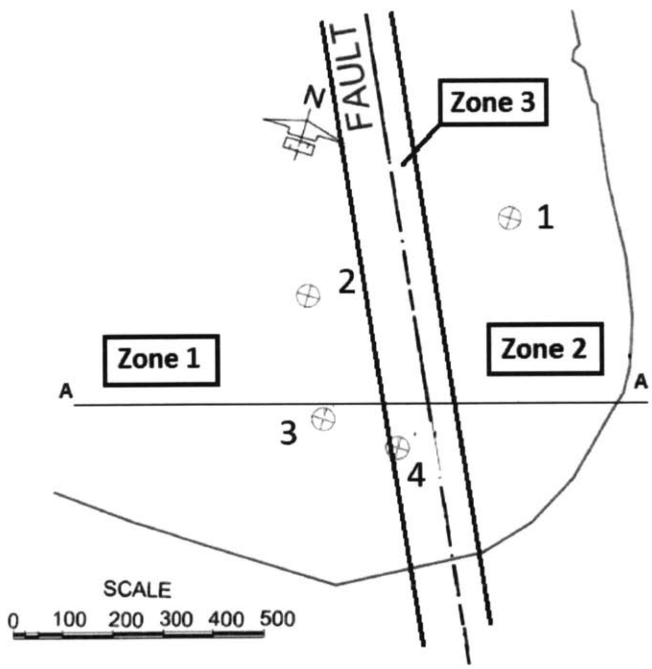

Fig 5.12a: Simplified zonal divisions for spatial variation (plan view)

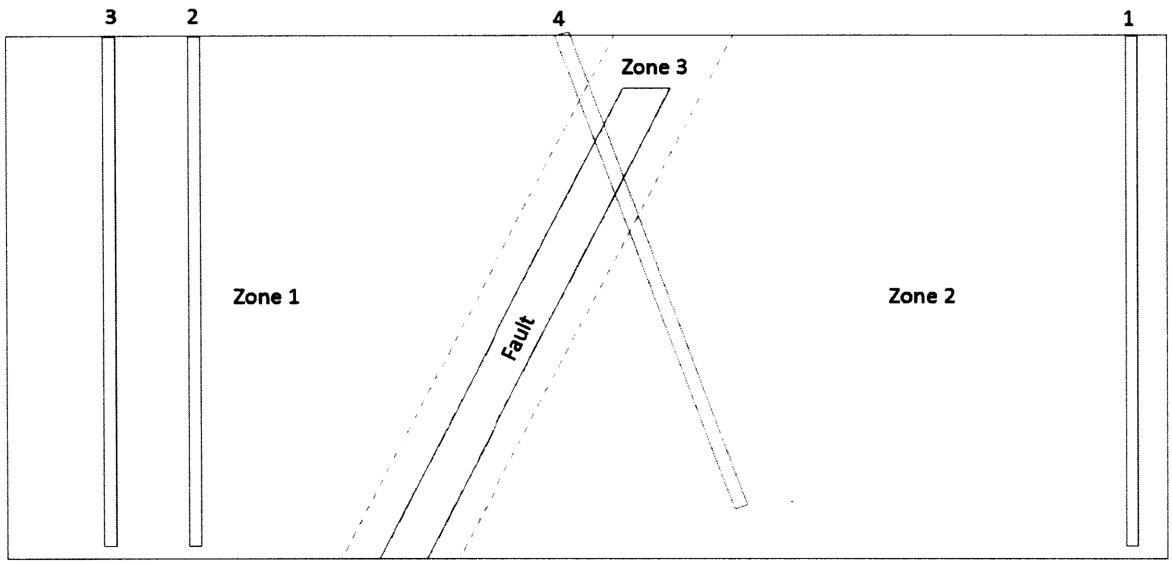

Fig 5.12b: Simplified zonal divisions for spatial variation (cross-section A-A)

As before, for each interval of 40 meters, observed RQD of Zone 1 (boreholes 2 and 3) and Zone 2 (boreholes 1 and 4) are analyzed, and transformed into discrete probabilities of the ground classes proposed in Table 4.2. The expected cost at the different depths is then plotted in Fig 5.13. From Fig 5.13, it is evident that the ground improves with depth in both zones, and it costs less to construct the cavern in Zone 1 at depths of 60 to 130 meters. The difference between the costs of constructing the cavern in the two zones becomes insignificant at about 130 meters.

DSMs are plotted for both Zone 1 and Zone 2 to illustrate the variability, shown in Fig 5.14 and Fig 5.15 respectively. From the DSMs, it can be clearly seen that to achieve 60% probability of GC1 and GC2 occurrence, a depth of 80 meters will be selected for Zone 1 and 110 meters will be selected for Zone 2. To achieve 80% probability of GC1 and GC2, a depth of 100 meters will be selected for Zone 1 and 120 meters will be selected for Zone 2. At depths of about 130 meters, there is no significant advantage putting the cavern in Zone 1 over Zone 2, which agrees with the expected costs shown in Fig 5.13.

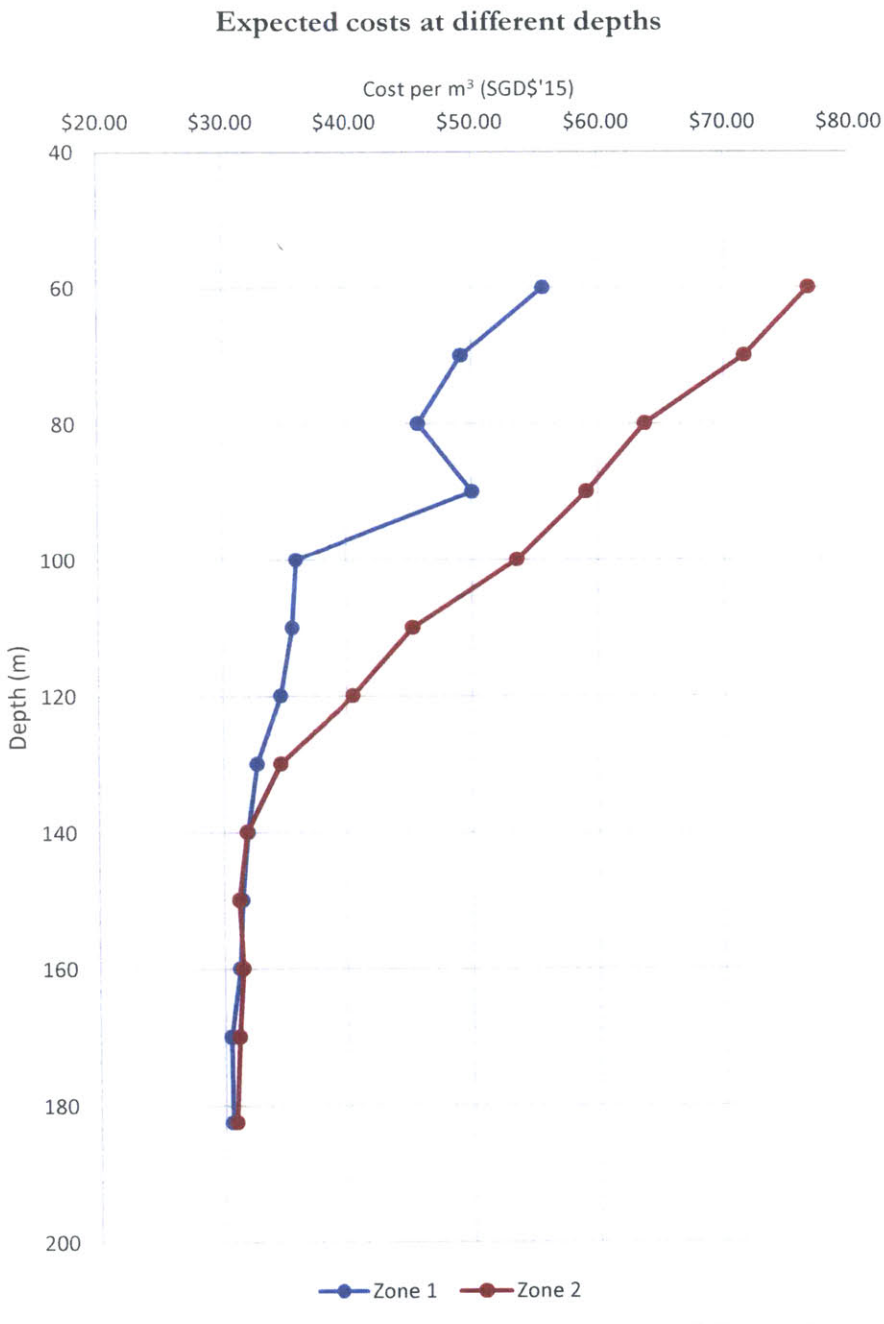

Fig 5.13: Expected cost at different depths for Zone 1 and Zone 2

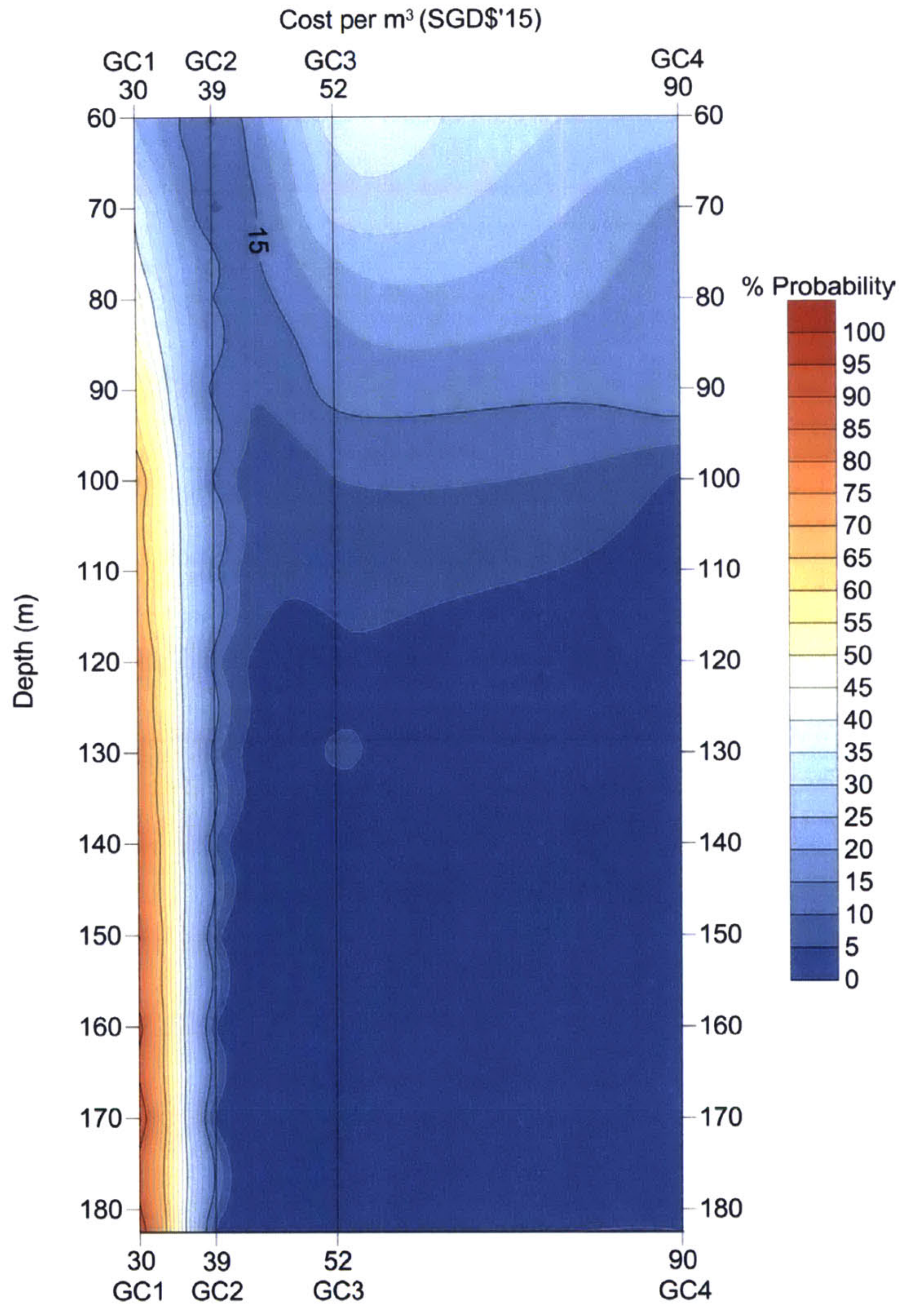

Fig 5.14: Depth Selector Map (RQD) for Zone 1

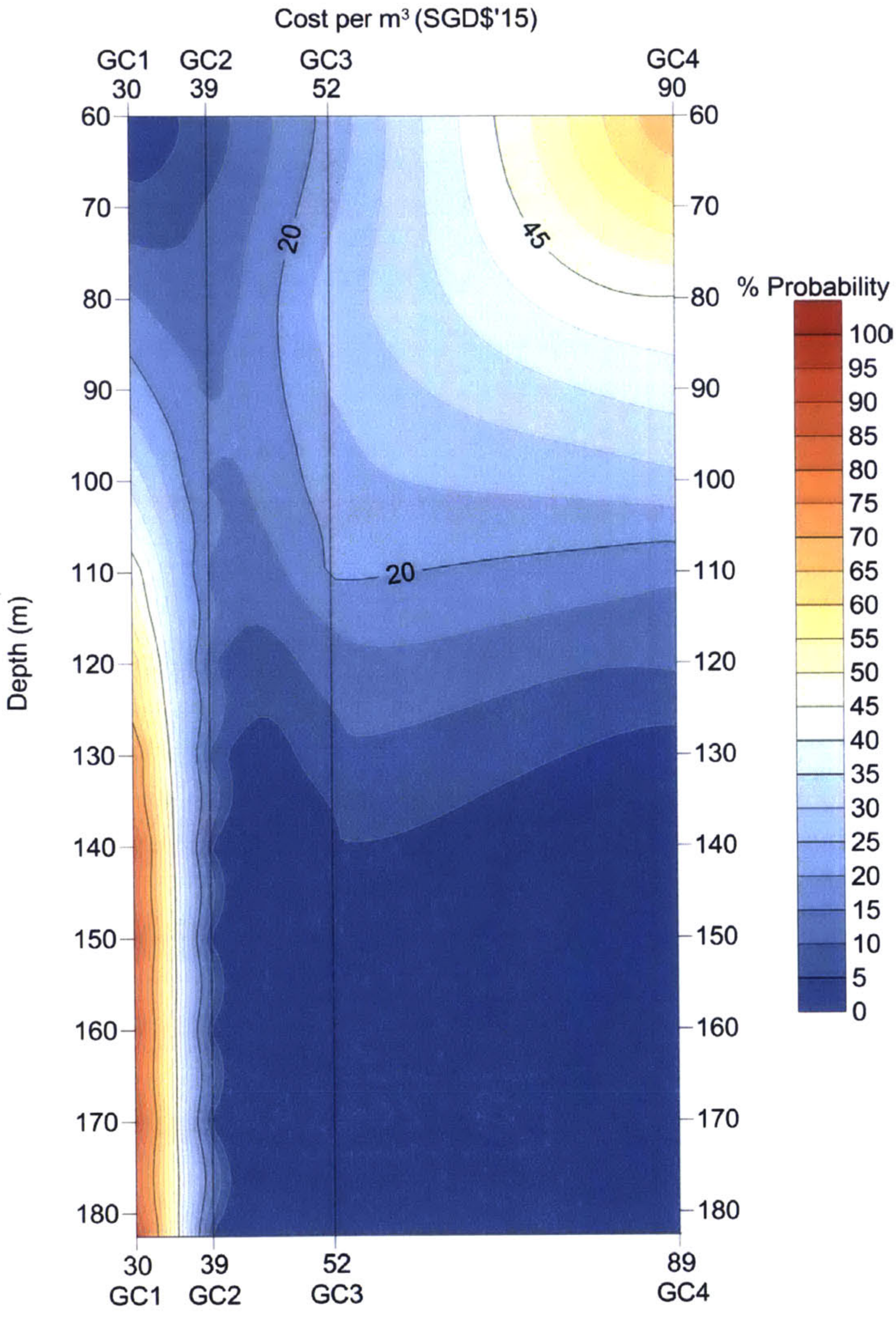

Fig 5.15: Depth Selector Map (RQD) for Zone 2

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CHAPTER SIX

CONCLUSIONS

6.1 CONCLUDING REMARKS

Singapore conducted various cavern studies since the 1990s, and has since constructed two caverns. The rising opportunity cost of land has made utilizing underground space a feasible and attractive option, and there are indications that more cavern development could be underway in the foreseeable future. In view of plans for cavern development, the Singapore Geological Office was set up in 2010, and has done much site investigations in different regions of Singapore. The study done in this thesis concentrates on an area of interest within South-Western Singapore, where the Singapore Geological Office has sunk four boreholes that are each 200 meters in depth.

Through the use of the empirical methods, the RQD and *Q*-system, rock support can be estimated for different ground classes for an assumed cavern size of 20 meters in width, 30 meters in height and 100 meters in length. The cost per cubic meter of cavern construction, which includes grouting, the rock support system, and the excavation, is then estimated. To account for the variability of the ground in the area of interest, probabilistic analyses and assessments of the rock mass parameters derived from the boreholes were carried out. Discrete probabilities were obtained from observed frequencies, and depth and spatial variability are assessed. Depth Selector Maps (DSM) are created to give planners an indication of the ideal location of a cavern, both in depth and spatially, by providing them with an indication of the variability of the ground so that planners can take the associated uncertainty into consideration when making decisions.

The DSMs created for the area of interest using both the RQD and the *Q*-system suggest that for about 70% chance of at least GC2, the cavern should be located at a depth of 80 meters. Locating the cavern at a depth of at least 110 meters yields at least 80% probability of GC1 and GC2, and for greater than 90% probability of GC1 and GC2, a depth of 140 meters is required. Spatial variability of the area of interest is assessed by dividing the area of interest into two zones, and it was found that in general, Zone 1 has greater probability of better ground classes from 60 to 130 meters. To achieve 80% probability of GC1 and GC2, a depth of 100 meters will be selected for Zone 1 and 120 meters will be selected for Zone 2. At depths of about 130 meters, there is no significant advantage putting the cavern in Zone 1 over Zone 2, where both Zones have about 80% chance of GC1 occurrence.

6.2 LIMITATIONS OF STUDY

Readers utilizing the results must note certain limitations of this study. Firstly, the cost of the rock support for each ground class are based on the two empirical methods, RQD and the *Q*-system, which has certain limitations. The RQD-support relations are empirically defined for tunnels 6 to 12 meters in diameter, but has been extrapolated and applied to an assumed cavern size of 20 meters width. *Q* parameters need to be derived from exposed faces, and cannot be determined purely from borehole logs. Furthermore, SRF is assumed to be 2.5 for this study, but can be complicated to determine. Also, it has been found that the *Q*-value is sensitive to the value of Jn, which is difficult to assess and quantify even in exposed faces. Moreover, as discussed briefly in Chapter Five, the probabilistic assessment using the *Q*-method is problematic because the parameters used to determine *Q* are not independent. The results in this thesis are acceptable for a preliminary feasibility study, but when actual design of any cavern scheme gets underway, these limitations must be kept in mind and the design should not be purely based on such empirical methods.

Secondly, the cost of cavern construction in this study only includes rock support by bolts and shotcrete, and grouting, and excavation for a simple assumed cavern size. There are many other important costs that are affected by the location of the cavern at different depths and in different ground classes, which must be taken into account after the actual cavern scheme is developed. These costs include shafts and access tunnels construction, transportation of equipment underground, and mucking, which could make locating caverns at depth more expensive even though the results of this thesis suggests otherwise.

6.3 APPLICABILITY AND POSSIBLE EXTENSION OF WORK

The DSMs can be easily re-created for other areas by probabilistically assessing the data available from the ground investigations, using the method discussed in Chapter Five. The current DSM can, and should also be updated when more ground information is available. For the purpose of this thesis, the probabilistic analysis of the spatial variability is based on a small dataset, and hence only divides the area of interest into three zones, where only two of the zones are considered. With more ground investigation, the size of the fault zone could be determined and assessed. Spatial variation can also be considered with higher resolution by dividing the area of interest into more zones. All the rock mass information and variability can be combined and reproduced in a 3D representation with ArcGIS or similar programs, with a database that can also be easily updated when more ground investigation information becomes available.

The RQD approach, which has been used extensively in this thesis, should be compared with other methods. Further studies could also improve the statistical approach to carry out probabilistic analyses of Q to capture the correlation of the parameters. With detailed logs done for the boreholes, it is also possible that other methods like RMR and GSI could be used for comparison with the results of this thesis.

Furthermore, in this study, only costs associated with the different ground classes have been considered. As done in the Decision Aids for Tunnelling (DAT), construction cost, time and resources can also be employed to reflect uncertainties, and incorporated into the results to give a more complete picture in the consideration of cavern feasibility. The Depth Selector Map is a useful tool that incorporates probabilistic assessment of rock mass properties to give planners a quick first indication of where to locate caverns. With more ground investigation information and incorporation of other factors that reflect uncertainties, it can play a significant role in Singapore's quest for utilizing space underground in the most cost-efficient way.

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