Rock Engineering Practice for Development of Underground Caverns in Singapore

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ABSTRACT: This paper presents the state-of-the-art practice in rock engineering for the development of rock caverns in Singapore based on two very significant rock cavern projects. It discusses the key components of rock engineering related to the planning, design and construction of large-span tunnels and caverns in rock. It covers technologies and strategies for site investigations and geological modelling, rock mass classification and empirical methods for rock support design, numerical analysis, instrumentation and monitoring, as well as key issues related to rock excavation such as water control and grouting, and safe use of explosives, and blasting vibrations. Two major cavern facilities, one in granite rock, and one in sedimentary rock, will be used as examples. It will be demonstrated that safe and cost-effective rock cavern construction can be achieved with a well-planned and executed rock engineering programme, supported with the innovative use of technologies, design optimisation, risk sharing, and collaboration among the client, consultant, and contractor.

1 INTRODUCTION

Studies of rock cavern developments in Singapore started in the 1980s (Broms, 1989), with feasibility studies for different applications (Lui et al., 2012; Zhou & Cai, 2011). The series of feasibility studies completed in the 1990s covered different geological formations in Singapore and applications (Broms & Zhao 1993, Zhao et al 1994, Zhao and Lee 1996). Table 1 summarises the major activities related to rock cavern development in Singapore.

The major breakthrough in cavern development in Singapore was the construction of the Underground Ammunition Facility (UAF) in rock caverns in the Bukit Timah granite, which was started in 1999 and was commissioned in 2008 (The Straits Times 1999, 2008). The UAF demonstrated the viability and significant benefits of using rock caverns to create space and improve safety, and served as a catalyst and case study for many later studies and thinking on the use of rock cavern space (e.g., Zhao et al 1999a, 1999b, 2000, 2001, 2004). The success of the UAF also gave a major boost to the second major rock cavern project, the Jurong Rock Cavern project (JRC) for oil storage (Zhao et al 2004). Phase 1A of JRC was officially opened by Singapore's Prime Minister in Sept 2014 (The Straits Times 2012, 2014).

The JRC is a cavern complex with cavern size typically 20m wide, 27m high, and 300m long, located at about 150 m below ground in the sedimentary rocks of the Jurong Formation (Kar & Ng, 2012). The current phase has a storage capacity of 1.5 million m^3.

Table 1. Summary of major activities on rock cavern development (Lui et al. 2012)

Period	Major Activities and Development
1990 – 1994	• Feasibility study of rock cavern construction in the Bukit Timah Granite by PWD/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 MINDEF to take the lead. • Feasibility study of the UAF (underground ammunition facility) by MINDEF/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/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 – 2013	• JRC (Jurong Rock Caverns for hydrocarbon storage) shaft construction started in 2007 and cavern construction in 2009 • Government set up inter-agency Underground Master Planning Task Force • 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/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
2013 – present	• Feasibility study for underground automated good mover system and Jurong West Underground Caverns • Feasibility study for underground cavern development at Gali Batu • Feasibility study for land optimized use for Kranji Water Treatment Plant • Feasibility study for underground drainage and reservoir system (UDRS)

2 GEOLOGY AND SITE INVESTIGATIONS

2.1 Geology and major rock formations

The Singapore rocks consist mainly of four major geologic formations (Lee & Zhou, 2009): (1) Sajahat Formation of metamorphic quartz sandstone and mudstone, (2) Bukit Timah granite, (3) Gombak norite, and (4) Jurong Formation sedimentary rocks.

The Sajahat Formation (S) is probably the eldest rock formation in Singapore. It is variably metamorphosed sedimentary rocks comprising quartzite, sandstones, and argillite. The Sajahat Formation (S) is found mainly at Pulau Sajahat, Sajahat Kechilkong, along north coast of Pulau Tekong and to the east at Tanjong Renggam.

The Bukit Timah granite (BT) forms the major basement rock of Singapore and covers about one-third of the Singapore island, with distribution at central main island, Pulau Ubin island, and the surrounding sea area. The term granite is used in a general sense for the entire suite of acid rocks including granite, adamellite, granodiorite, and the acidic and intermediate hybrids (mainly of granodioritic and dioritic composition) which resulted from the assimilation of basic rock within the granite.

The Gombak Norite (GN) is a body of noritic and gabbroic rock with exposure on the western side of the Bukit Timah Granite on the Singapore Island. The unit is named after Bukit Gombak where noritic and gabbroic rocks are well exposed in a number of quarries. The noritic and gabbroic rocks are coarse-grained and plagioclase-rich with varying amounts of clino- and orthopyroxene minerals appearing as interstitial grains giving an intergranular texture to the rock.

The Jurong Formation (JF) sedimentary rock is the youngest rock formation in Singapore. It overlies the above basement rock formations, and extensively covers one-third of the main island area, at south and southwest Singapore. Extensive areas in the Jurong Formation have been affected by dynamic metamorphism resulting from tectonic activity. The grade of metamorphism is low, and it is still possible to determine the facies that has been affected.

2.2 Site investigations for cavern construction

Extensive site investigations (SI) were carried out to assess suitability of the sites for construction and to obtain reliable data for design during the various phases of the projects. The SI works formed an integral part of the planning and engineering design process, and were guided by an overall geological (especially structural geology) model which helped minimize the cost of SI with more targeted work and reduce geological uncertainties.

A successful combination of conventional soil boring, diamond coring drilling, and modern geophysical methods provided reliable information about the geology and rock mass properties of the site. Some of the geophysical techniques included surface seismic refraction and reflection surveys, reverse seismic vertical profiling, electric resistivity profiling, cross-hole tomography, borehole radar, borehole geophysical logging, borehole camera and acoustic imaging, and core orientation (Zhou, 2001). Table 2 shows a summary of the type of SI work carried out in these projects.

In the case of the JRC, horizontal directional coring (HDC) was used for the first time in Singapore (Wong et al., 2012), as the cavern facilities were located beneath the seabed, and access for SI works were limited (Figure 1). HDC uses a specially-designed direction-steerable core barrel and accurate direction measurement tool together with wire-line core drilling rig, making it possible to perform continuous rock coring along the planned tunnel alignment. This technique has subsequently been used in SI works for tunnel alignment in the undersea cable tunnel between Jurong Island and the main island. However, the HDC's accurate directional control is not feasible in soils. It requires that rocks must be weathering GIII (for Bukit Timah Granite) or SIII (For Jurong Formation) or better. If weakness zones or fault zones or soil zones are encountered during HDC drilling, grouting may have to be carried out to avoid both drilling fluid loss and hole collapse.

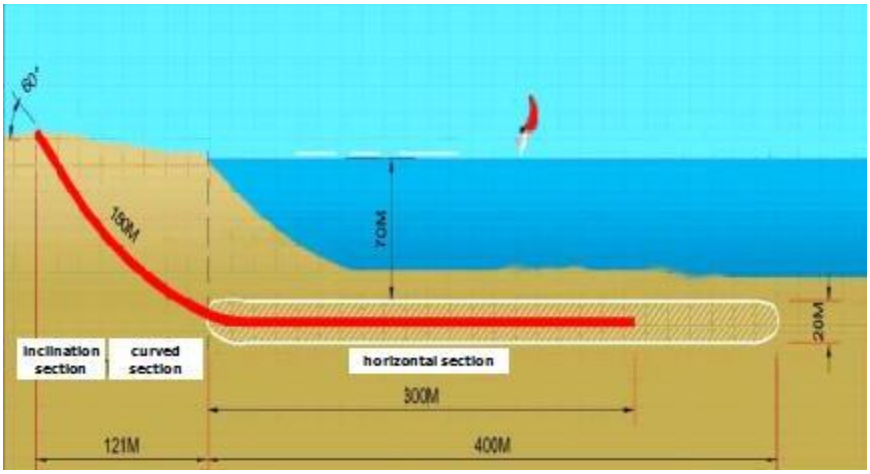


Figure 1. Horizontal directional coring used for the Jurong rock cavern at Jurong Island. The HDC trajectory is usually divided into three sections: 1) initial inclined straight section, 2) curve section, and 3) horizontal section (along tunnel alignment).

It is important to highlight the importance of combining borehole drilling with geophysical surveys, and of using complementary methods as cross checks for the interpretation of the geological model. Figure 2 shows a composite section in granite rock with seismic refractions, electrical resistivity profiling, calibrated with both vertical and inclined boreholes. The overlapping scope of such methods reduces uncertainty thus giving better resolution and confidence of the SI results.

Site investigations were planned and provisioned for the various phases of the engineering process, namely:

- Preliminary site investigations to establish overall feasibility during planning
- Main phase investigations based on conceptual layout and selected method of tunnelling for the purpose of detailed preliminary design
- Supplementary investigations during construction to allow for contingency and checking any unexpected geological conditions.

Table 2. Summary of main site investigations carried out

Type	Methods	Objective and type of data
Drilling	Soil boring; diamond core drilling	Overburden; rock cores for laboratory testing & logging; rock mass classification & quality
Surface geophysical surveys	Seismic refraction, reflection; electric resistivity tomography	Main geological structures; overburden depth; rock mass quality
Borehole surveys and *in-situ* testing	Temperature logging; seismic logging; borehole camera & acoustic imaging; cross-hole tomography; borehole radar; impression packer; Lugeon test; rising head/falling head test.	Ground temperature; seismic velocities; joint data; permeability; geological structures
Laboratory tests	Point load; uniaxial/triaxial compression; Brazil tensile; 3-point flexural; shear test of rock joints	Mechanical properties of intact rock and rock joints
In situ stress	Hydraulic fracturing; 3-D overcoring	Hydraulic fracturing; 3-D overcoring (during construction)
Other methods	Core orientation Marine seismic surveys Horizontal Directional Coring (HDC)	Joint orientation Seabed depth and geological layers Rock core, geological information & fault zones along cavern alignment

2.3 In situ stress measurements

In situ stress measurements were carried for both the UAF and JRC projects using hydraulic fracturing during site investigations. In the case of the UAF project, additional measurements by 3D over-coring were also made during construction as a validation (Zhao et al., 2005; Zhou et al, 2003). Results of these measurements show that the horizontal stress in both rock formations is higher than the vertical stress, with the ratio of the minimum and maximum principal horizontal stresses to the vertical stress between 2-3. Results of the in situ stress measurements are compared to data from other countries of the world as compiled by Hoek (2007), as shown in Figure 3.

In situ stresses are important boundary conditions for design analysis. For competent rock conditions at relatively shallow depths with respect to the strength of the rock, a relatively high horizontal stress is considered conducive to roof stability because rock is generally very strong in compression and relatively weak in tension. This favorable stress condition is reflected in the Stress Reduction Factor (SRF) in the Q-system (Barton et al., 1974, 1992), and can be used to optimize the geometry of the cavern & rock support measures and minimse cost of rock excavation for unnecessary arch height during detailed design.

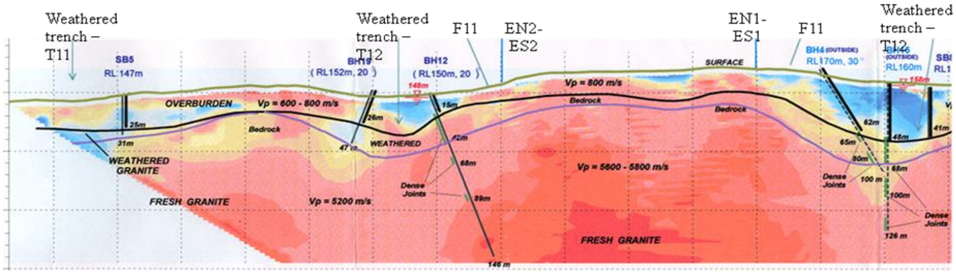


Figure 2. Composite geological profile with the electrical resistivity and seismic refraction surveys, and calibrated with vertical and inclined boreholes.

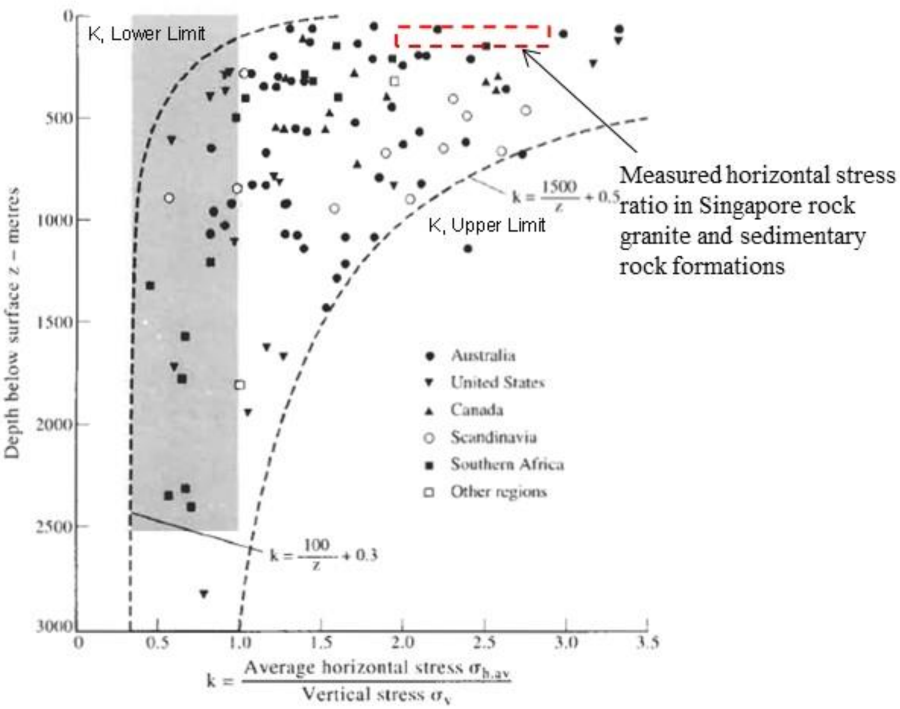


Figure 3. Comparison of horizontal stress ratios (after Hoek, 2007)

2.4 Geological baseline and risk sharing

In rock engineering, geological uncertainties represent a major source of geological risks, and present a major challenge for both construction and contractual risk management. Unlike in conventional civil engineering projects, where the engineer can design and then specify the type of construction material, in rock engineering the engineer has to deal with construction materials and conditions that will never be fully understood until the tunnel is excavated. As such, disasters, disputes and cost claims in large underground rock engineering projects are common. Such disputes can be minimised through a good site investigation program and risk sharing between the owner and contractor.

While such uncertainties and the need to manage the associated risks are generally recognised by owners (to varying degrees) and contractors, the contractual approach to managing such geological risks can vary greatly. Various concepts for risk sharing such as the Geotechnical Baseline Report (GBR) have emerged over the years. A similar concept in Norway is the Engineering Geology Report (EGR). In essence, these reports present the reference conditions on which all planning, preliminary design, and cost estimates are based. Typically the client and the contractor agree on the performance criteria for the expected geological conditions, and provisions are made for differing conditions which are unexpected. The use of GBR (in JRC) and EGR (for the UAF) made it possible for risk sharing and allowed a more realistic pricing by the contractor (Zhou & Cai, 2007).

The baseline report contains the general geological setting, geological structures, rock head elevations, rock properties and rock mass classification & properties, hydro-geological conditions, expected performance and design for planned tunnel dimensions.

Tables 3 and 4 show some typical rock mechanics properties and rock mass classifications in the Engineering Report for the UAF.

Table 3. Summary of rock joint properties of the Mandai granite rock

Joint conditions	Friction Angle (^{o})	Cohesion, *C* (Kpa)
Freshly fractured and dry	45.6	258
Freshly fractured and saturated	42.6	172
Freshly fractured and dry (weathered rock)	36.8	183
Natural and dry	36.5	266
Natural and saturated	33.4	108
Mineral filled and dry	32.5	71
Mineral filled and saturated	27.3	52
Weathered and dry	27.6	200
Weathered & saturated	20.1	136

Table 4. Statistical distribution of Q values from core logging of Mandai granite

Q Value	Rock Mass Quality	Percent, %
0.01 – 0.1	Extremely poor	1.9
0.1 – 1.0	Very Poor	3.7
1 – 4	Poor	5.8
4 – 10	Fair	13.6
10 – 40	Good	51.8
40 – 100	Very Good	19.3
> 100	Extremely Good	3.8

3 EMPIRICAL ROCK ENGINEERING DESIGN

3.1 Empirical design using the Norwegian Q System

The rock engineering process can be summarized as follows:

- Geological modelling and site investigations
- Preparation of baseline report
- Preliminary design based on rock mass classifications (using the Q method). These are supported by numerical analysis
- Final support design based on tunnel mapping after excavation
- Instrumentation and monitoring to validate design assumptions and tunnel performance.
- Updating of geological model, re-modelling based on mapped joint data, and refinement or optimization of support design

Both projects adopted the empirical design method using the Q-system developed in Norway by Barton and his colleagues (Barton, 1974; Barton & Grimstad, 1992; Barton & Grimstad, 2014).

Based on the rock mass classification, the preliminary rock support (or reinforcement) are defined according to the design dimensions of the tunnel and other parameters. During construction, the excavated tunnel section is firstly scaled, and then mapped by an engineering geologist. The final support is then prescribed according to the Q value calculated from the mapped section, and approved by the QP (design). Submissions to BCA for rock support design before commencement of construction are based on different rock classes/Q values works expected to be encountered during excavation works. These rock classes are derived from the site investigation works.

3.2 Rock reinforcement and design

The key concept of the active design makes use of the rock mass as the main structural material, and the reinforcement is used to help the rock mass support itself. In both projects, the primary and permanent rock support is a combination of rock bolts and fibre-reinforced shotcrete without any use of concrete lining except in very poor rock conditions.

The rock bolts used for the UAF are called the CT-bolts (combination tube). The bolt has a mechanical anchor, which can provide 5-ton anchor force for immediate support after the bolt is inserted into the drill hole (Zhou, 2002). It also helps to improve construction safety as the risks of a normal rebar sliding out of the drill hole were eliminated. The installed bolts were then fully grouted with cement grout. The rock bolt has a diameter of 22 mm, and yield strength of 220 KN.

Corossion protection for rock support is a key consideration for the long-term stability and maintanability of the rock cavners. For the UAF, CT bolts had double corrosion protection with galvinisation of the bolts and cement grouting between the bolts and rock. The mechanical anchor ensured that the rock bolt was centered properly in the borehole and allowed for an even distribution of the cement grout around the bolt.

For the JRC project, the ground water was much more corrosive because of the high salt content. For this purpose, fully grouted fibre-glass rock bolts were used.

Table 5 shows a summary of typical dimensions of the large tunnels and caverns. The rock reinforcement design is a combination of rock bolts and steel-fiber reinforced shotcrete, based on the Q-system. Tables 6 and 7 show a summary of rock reinforcement used for the three typical tunnel types and storage caverns in the JRC.

Table 5. Summary of typical tunnel dimensions

Tunnel Parameters	Type I	Type II	Type III
Width (m)	10	15	30
Wall height (m)	4.5	6.5	8.5
Arch height (m)	8.1	11.2	13.5
Area (m^2)	62	115	275

Table 6. Summary of rock support for typical tunnel dimensions *

Class	Q	Type I	Type II	Type III
A	>40	Spot. 40mm	Spot. 40mm	Spot. 40mm
B	10-40	L3(2.4). 40mm	L4(2.4). 40mm	L5(2.4). 40mm
C	4-10	L3(2.2). 40mm	L4(2.2). 40mm	L5(2.2). 50mm
D	1-4	L3(1.9). 50mm	L4(1.9). 50mm	L5(1.9). 75mm
E	< 1	L3(1.5). 75mm	L4(1.5). 75mm	L5(1.5). 100mm

*Notes:

a) L3(2.4) = rock bolt length of 3 m at 2.4m center-to-center spacing.

b) A minimum shotcrete thickness of 40 mm to all crowns for safety reasons.

Table 7 - Rock support design for storage cavern at different Q value in JRC

Q value	Crown / End wall			Side wall / End wall		
	Bolt density	Embedded bolt length (m)	Unreinforced shotcrete thickness (mm)	Bolt density	Embedded bolt length (m)	Unreinforced shotcrete thickness (mm)
>40	Spot bolting	4.4	80	Spot bolting	5.3	80
10-40	1 bolt / 5.2 m2	4.4	80	Spot bolting	5.3	80
4-10	1 bolt / 4.4 m2	4.4	60 SFRS only	1 bolt / 5.2 m2	5.3	90
1-4	1 bolt / 2.8 m2	4.4	110 SFRS only	1 bolt / 4 m2	5.3	100 SFRS only
<1	1 bolt / 1.9 m2	4.4	140 SFRS only	1 bolt / 2.5 m2	5.3	130 SFRS only

SFRS - Steel Fiber Reinforced Shotcrete

3.3 Other considerations in support design

The support design of pattern rock bolt and shotcrete using the Q-chart is for the general conditions of the rock mass according to the rock mass classification. It does not consider specific geological conditions such as unstable wedges or weak zones between competent rock mass. In such cases, additional considerations must be given to the identified conditions. For the unstable wedges, rock bolts must be designed with sufficient length to make sure they are anchored into the stable rock, and the total support capacity of the combined effects of shotcrete and rock bolts must be able to support the assumed weight load of the wedge.

Steel fire reinforced shotcrete was used in combination with rock bolts throughout the project for tunnel support as the final reinforcement. Shotcrete failure can occur in punching shear, debonding from the rock, or bending, depending on the loading condition. It can be easily demonstrated that even with a nominal shear strength of 2 Mpa and bonding strength of 0.5 Mpa between the shotcrete and rock, shotcrete can provide sufficient support for most potentially unstable rock blocks and wedges.

For heavily fractured or weakness zones, usually 1-2 m wide, spiling (temporary bolts installed before excavation) will allow blasting to take place without the roof falling. Support of such weakness zone should be anchored to the competent rock on both sides of the weak zone. Spiling bolts installed ahead of the excavation are used to prevent collapse of the roof during excavation.

For a nominal shear strength of about 2 Mpa, a 1 m by 1 m area of shotcrete with a 50 mm thickness can support a dead weight of about 40 tons, assuming that the shotcrete has good bonding with the rock. Using wet shotcrete, it is quite easy to achieve a bonding strength of 0.5 Mpa on good granite rock with proper washing of the rock surface, which is normally specified in tunnel contracts. To achieve a 40-ton weight strength, the total area of good bonding required would be about 0.8 m^2. This analysis, however, does not take into account the frictional force on the rock wedge due to cohesion and more importantly, the normal stress from the relatively high horizontal stress, and sometimes rock ridges (non-continuous joints), and is therefore conservative in most practical cases.

Similar analysis can be done for the weakness zone. For example, assume a 2-m wide weakness zone continuous across the tunnel, and a 20-m high loose rock column. The weight load of this column (per linear meter across the tunnel) would be about 2x20x2.65 = 106 tons. Assuming a shear strength of 2 Mpa, the thickness required to add a 0.5 value to the safety factor (to support a weight of 0.5x106=53 tons) would be 530 KN/(2 m x 2Mpa) = 13.2 cm. Similarly, the bonding area required to support this load can be analysed. If rock bolts were applied to both sides of the fractured zone, each bolt having a design capacity of 250KN tensile capacity, the safety factor would be further improved. In this case, it would be reasonable to assume zero friction because of the highly fractured nature of the weakness zone, although in reality some friction force will be able to hold the rock column.

From the above simple analysis, it is clear that shotcrete plays a significant role in supporting potentially unstable wedges. In the commercial software UNWEDGE, most analysis will have a very high factor of safety against unstable wedges once shotcrete is included in the analysis.

At tunnel intersections or portal area, the rating parameters for the Q are adjusted to account for the reduced confinement and more degree of freedom for rock blocks to move. At tunnel intersections, the effective span is used as the tunnel span for the support design.

3.4 Water control

Water is said to be the "smartest construction material" due to its ability to find any possible passage to the tunnel opening. Thus, the concept for water control in the UAF was one of controlled drainage rather than water proofing, with the latter being more expensive and difficult to achieve long-term durability. The water control was based on a maximum allowable seepage rate by way of pre-injection grouting. The seepage criteria (typically from 5 to 15 l/100m tunnel) were decided based on usage of the excavated space, a balance between construction (and grouting) cost and pumping cost during operations, and ground water and environmental considerations above the facility. In order to minimize risks of surface subsidence due to water draw down, the overall seepage rate into the underground facility had to be estimated and controlled to within the overall surface recharge rate of the surface catchment area with considerations for seasonable variations in rain fall.

For environment control in the storage caverns where stringent moisture control was required, an internal PVC tent was used to keep the moisture away. In the traffic tunnels and other areas, seepage water from the crown is diverted to the side drain. No other water proof was used. The main requirement was that the shotcrete should be properly drained. In some places, shallow holes were drilled into the rock to relieve the water and to ensure the water does not move to other places.

This concept of water control eliminates the risk of water seepage in water proofing, since there is no buildup of water pressure because of the drained concept. However, for the storage caverns, the installation of internal tent requires some additional excavation volume.

For the UAF project, the permeability of the rock mass consisting of competent rock and joints is in the range of 10^{-8} - 10^{-9} m/sec. Only limited pre-grouting was required in tunnel sections near the weathered trenches. The technical specifications called for probe drilling when excessive water inflow was encountered at the blasting face. The probe hole was drilled to 25m from the face. Pre-grouting was required whenever the combined water inflow at exceeded 15 l/min in three boreholes. Depending on the expected water pressure at the location of grouting, the stop pressure specified was generally in the range of 30-50 bar. The objective of the pre-grouting was not to stop the water completely, but to reduce it to a reasonably low level so that pumping of seepage water becomes economically feasible.

Other considerations for injection grouting include: a) overlapping between successive pre-grouting rounds had to be at least one advance length (about 5m); and b) the grouted zone should be thicker than the length of the rock bolts to prevent penetrating of rock bolts into water zones after grouting during bolt installation. This would influence the look-out angle of the grout holes. Alternatively, rock bolt lengths may have to be reduced and the support supplemented by thicker shotcrete layers.

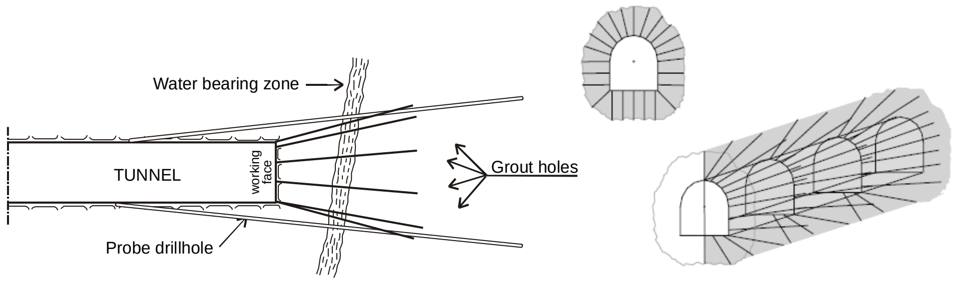


Figure 4. a) Probe holes and grout holes. b) Overlapping of successive grout rounds

In the case of the JRC, there was substantially more water inflow, especially during shaft sinking, which penetrated layers of highly weathered sedimentary rock at between 60m to 90m below ground . This condition proved quite challenging (Kim et al., 2012). Extensive umbrella grouting around the shaft was carried out to minimize the groundwater inflow and meet the maximum allowable seepage. For the excavation of the operation tunnels at 100m depth, similar grouting technique used at the UAF project was adopted to control the water inflow. For the oil storage caverns at greater depths, water control for the storage caverns was based on the water curtain concept, where ground water at 10 bar is used to confine the oil products in the cavern. As such, the allowable water seepage rate was designed to be substantially higher, but the actual water inflow was lower than anticipated as competent rock with low permeability was encountered at most areas of the storage caverns during the excavation. The key challenge during construction of the caverns was to maintain the necessary water curtain pressure to prevent loss of water pressure.

3.5 Numerical analysis

During detailed design, numerical modelling using UDEC and 3DEC (3D UDEC) was used to verify the support design, and test sensitivity of design parameters, plan cavern excavation sequence, and provide input prediction for instrumentation and monitoring during construction. The discrete element program UDEC has been widely used to model problems in jointed rock mass (e.g., Itasca, 1998; Cundall 1980; Fairhurst & Pei 1990; Pinnaduwa et al. 1994; Barton et al. 1994; Shen & Barton 1997). In UDEC modelling, rock joints are modelled explicitly, and the properties of rock joints have to be input. Rock joint properties include joint orientation, joint spacing, joint friction angle, joint normal stiffness (kn), and joint shear stiffness (ks).

The variation of input parameters and excavation sequence to the models yielded valuable information on the sensitivity of the design parameter and cavern performance to the variation of the in-situ conditions. Calculations were initially based on data from results from site investigation, and continually updated with data from tunnel mapping and in-situ observation during excavation. Figure 5 shows the 3D UDEC calculations for a typical storage cavern in the JRC.

Another type of numerical analysis, which is perhaps even more important given the mostly competent rock and relatively shallow depths, is the identification and analysis of potentially unstable wedges formed by rock joints, otherwise known as structurally controlled failures. Even in the best rock conditions, rock falls can occur if there are sufficient joints and free surfaces created by the excavation. Thus, the analysis of unstable wedges is an important part of the design process. The program UNWEDGE, developed by Rocscience, has been used extensively for such analyses.

These analyses provided additional quantitative input for the support design such as bolt length and spacing on top of the empirical design using the Q-system. With sufficient joint data, analyses before construction could also be used to optimize the orientation of the rock tunnels and caverns based on the predicted size of the unstable wedges. Figure 6 shows the calculated volume of the potentially unstable wedges as a function of the tunnel orientation. During construction, joint data from tunnel mapping were used to update the joint system and additional calculations were made to improve accuracy and check for possible additional unstable blocks. This analysis was also used for planning the excavation strategy and provision of rock support in case of expected large unstable wedges. This was critical to the safe construction of large tunnels and caverns.

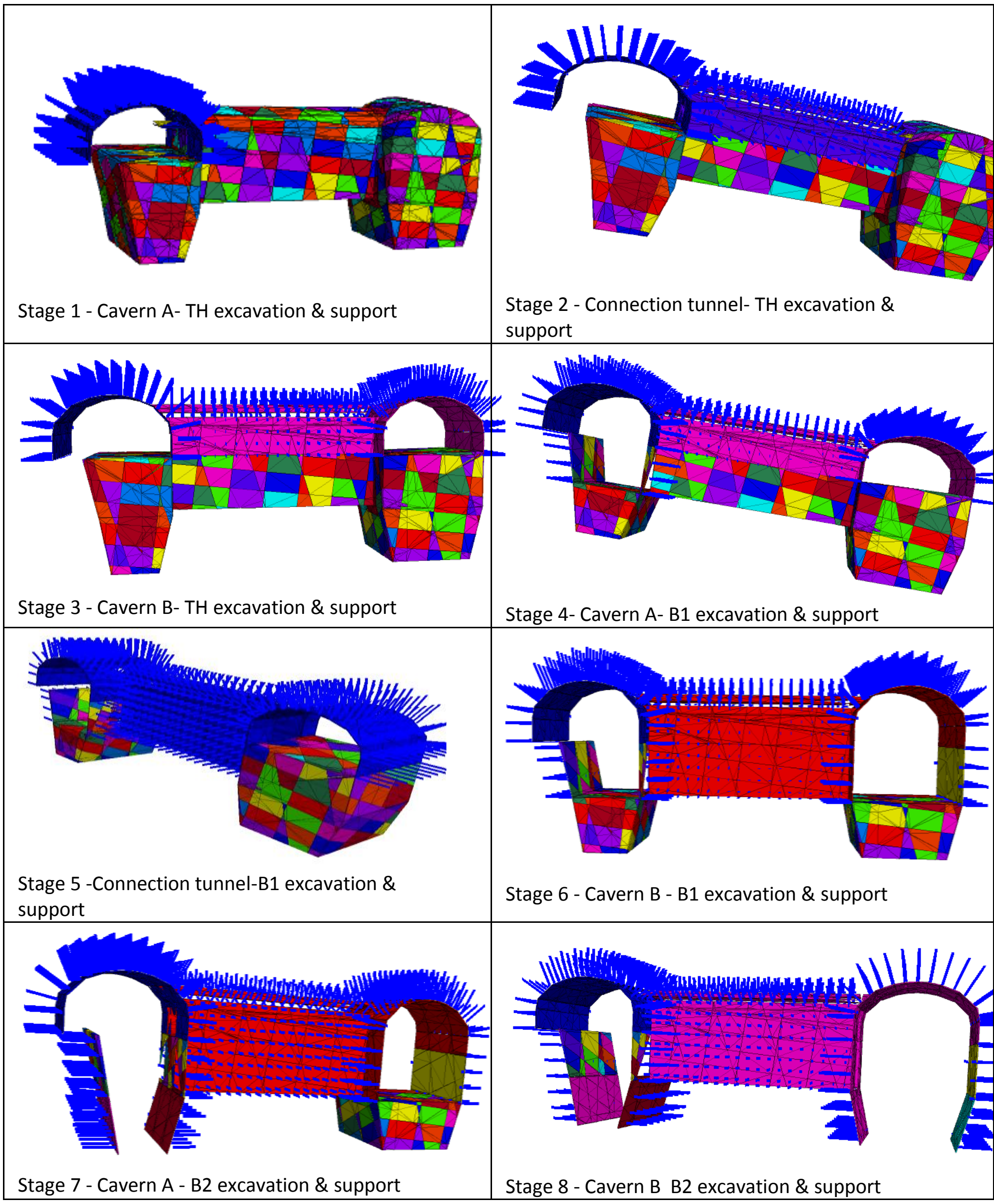


Figure 5. 3D UDEC modelling of the cavern excavation sequence and rock support of the JRC (Kar and Ng, 2012)

Table 8. Input parameters used in the 3D UDEC calculations shown in Figure 5

a) Tunnel alignment and sedimentary rock structures	
Tunnel Axis Orientation	042.5 degree (NE), plunge 0
Joint 1 (rock bedding plane)	24/054 (dip amount / dip direction)
Joint 2	84/270
Joint 3	77/182

b) General intact rock properties	
Density	27 kN/m3
Young's modulus	65,000 MPa
Poisson's ratio	0.3
Failure criteria	Mohr-Coulomb
Tensile strength	0 MPa
Peak friction angle	10.3 degree
Peak cohesion	68.6 MPa
Bulk modulus	54,166 MPa
Shear modulus	25,000 MPa

c) General rock joint properties			
Properties	Joint set 1 (bedding plane)	Joint set 2	Joint set 3
Normal stiffness	10,000		
Shear stiffness (MPa/m)	1,000		
Cohesion (MPa)	0.2		
Friction angle	30		
Slip criteria	Mohr-Coulomb		
Tensile strength	0		

4 ROCK EXCAVATION

4.1 Access to the underground

With a relatively flat terrain and deep weathering in Singapore, access to the underground would have to go through the layers of soft ground and weathered rock, unless an existing rock face such as an abandoned quarry can be found. In the case of the UAF project, the project made use of an existing quarry to gain access to the underground via a sloping tunnel. For the JRC, access to tunnels and caverns on both levels was made through two vertical shafts each of which was constructed from the surface of the reclaimed land at each side of the Banyan Basin to a depth of 132m from ground level. These shafts are connected by two tunnels, one at the 100m level and another at the 132m level. The adoption of vertical shafts as the main access to the tunnels and caverns was partly due to the flat terrain of the site, which is primarily reclaimed land (Kim et al., 2012), and partly because of the thick layers of sand fill, marine clays, and deeply weathered sedimentary rocks, which would have made tunnelling more challenging.

4.2 Blasting vibrations

Construction of the rock tunnels and caverns were carried out using the drill and blast method. There were two primary reasons for using this method: the hard rock and the uneven geometry and layout of the tunnels and caverns, which make tunnel boring machine impractical. The use of other mechanical methods such as road headers would have been possible but was considered too slow due to the large volume of excavation and slow cutting process.

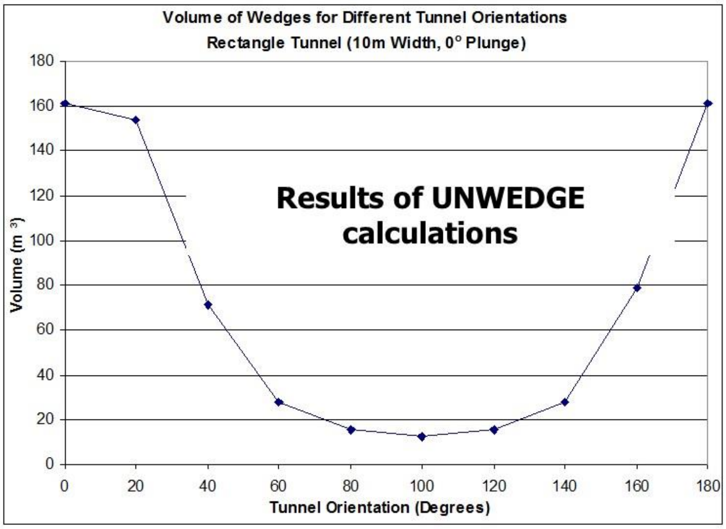


Figure 6. Example of estimated volume of largest wedges formed by three joint sets as a function of the tunnel orientation. Based on joint fata from acoustic imaging BH12: (136/57) (66/73) (280/44).

A key performance parameter for the drill and blast method is the evaluation and control of blasting vibrations. For the Mandai granite, a series of tests were conducted at a quarry to obtain data on ground shock propagation in a simulated mixed geology setup (granite bedrock overlain by residual soil). Accelerations were measured in rock free field, at the interface between rock and soil, and on soil surface. These represent various conditions at and near the construction site. Explosives charges include both fully coupled charges and decoupled charges to simulate normal blasting holes and blast holes with controlled blasting. Details of the tests can be found in Wu et al. (1998) and Zhou et al (1998 & 2001).

Based on regression analysis, the ground shock for peak particle velocity in rock and soil are shown in the following equations.

For fully coupled rock free field:

$$V = 1099\left(\frac{R}{Q^{1/3}}\right)^{-1.44} \quad (1)$$

For decoupled rock free field:

$$V = 498\left(\frac{R}{Q^{1/3}}\right)^{-1.45} \quad (2)$$

For fully coupled soil overburden:

$$V_X * \frac{R_V}{Q^{1/3}} = 212e^{-1.22\left(\frac{R}{Q^{1/3}}\right)} \tag{3}$$

$$V_Z * \frac{R_V}{Q^{1/3}} = 3876\left(\frac{R}{Q^{1/3}}\right)^{-1.88} \tag{4}$$

For decoupled soil overburden:

$$V_X * \frac{R_V}{Q^{1/3}} = 53e^{-1.22\left(\frac{R}{Q^{1/3}}\right)} \tag{5}$$

$$V_Z * \frac{R_V}{Q^{1/3}} = 969\left(\frac{R}{Q^{1/3}}\right)^{-1.88} \tag{6}$$

where R = horizontal distance (m); Q = max charge per delay (kg); V = radial peak particle velocity in free field; V_X and V_Z are horizontal and vertical velocity, respectively; and R_V = rock cover above the tunnel face (m).

Monitoring stations were used to gather data on ground vibrations during tunnel blasting. Geophones were placed on the quarry wall outside, on adjacent tunnel walls, and on water pipes placed in residual soil near the blasting face. The peak particle velocities for the tunnel wall and quarry walls are radial velocities while those for the water pipes are vertical velocities. Table 3 shows selected data from the monitoring.

It is important to note that the peak particle velocities recorded by the geophones on the quarry and tunnel walls are different from free field motion. The reflections at the free surface can increase the ppv by up to 2 times. Therefore, when comparing the monitoring data with those from the ground shock testing, the values from the monitoring results have been halved for consistent comparison (Figure 7).

From Figure 7, it can be seen that for development blasting, the fully coupled ppv equation for rock free field compares reasonably well with the monitoring data from the tunnel and quarry walls. Due to effects of soil-structure interaction, motions recorded on the water pipe, which are actually response values, are generally lower than input motions from the soil. For such motions the soil surface equation for vertical motion gives a better prediction better than that for horizontal motions. As discussed on Zhou et al (2001), most ppv criteria are actually based peak vertical motions. The main reason for this is that for most construction blasting, the point of interest are usually located near the face so that vertical motion will dominate at the surface. Therefore, looking from this point of view, again the data from the water pipe seem fall quite nicely along the best-fit soil ppv equation.

Table 9. Selected data from monitoring ground vibrations

Monitoring Location	Distance to centre of face	Max charge per delay, kg	Scaled distance, m/kg$^{1/3}$	Recorded PPV, mm/s	Adjusted PPV, mm/s
Tunnel Wall					
Tunnel-1B	13.3	90	3.0	307.0	153.5
Tunnel-2B	14	90	3.1	--	--
Tunnel-3B	17.5	90	3.9		0.0
Tunnel-4A	32.3	90	7.2	118.0	59.0
Tunnel-4B	54	90	12.0		0.0
Tunnel-1A	62.4	90	13.9	161.0	80.5
Tunnel-2A	66.6	90	14.9	30.0	15.0
Tunnel-3A	72.3	90	16.1	50.0	25.0
Water Pipe					
Water Pipe-A	190	80	44.1	1.8	0.9
Water Pipe-B	190	80	44.1	0.5	0.3
Water Pipe-A	180	80	41.8	1.3	0.7
Water Pipe-B	180	80	41.8	0.8	0.4
Water Pipe-A	150	80	34.8	1.2	0.6
Quarry Wall					
Quarrywall-B-1126	145	64	36.3	14.5	7.3
Quarrywall-B-1127	145	64	36.3	20.1	10.1
Quarrywall-A-1201	190	40	55.6	4.2	2.1
Quarrywall-B-1201	160	40	46.8	8.2	4.1
Quarrywall-B-1206	150	40	43.9	9.4	4.7

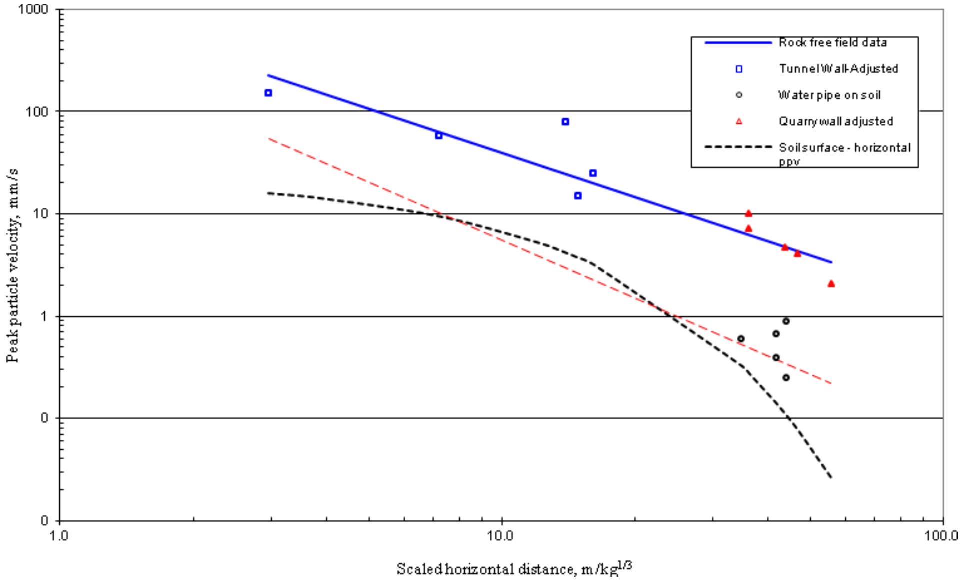


Figure 7. Comparison of peak particle velocities for various monitoring locations with data from separate ground shock testing for rock free field and soil surface.

4.3 Safe Use of blasting explosives

In rock blasting, explosives storage and handling constitute a major cost item due to stringent safety regulations in Singapore. The explosives would have to be stored in a designated commercial storage magazine. For daily use, the explosives have to be drawn from the storage magazine, and escorted to the site by certified armed security guards. Un-used explosives must be disposed of at the end of each day. For the Mandai project, the explosives storage and handling alone was estimated to be about \$5-10/m^3 if conventional explosives were to be used. In addition, the need to store explosives away from the construction site, and tedious procedures of withdrawing explosives for daily consumption would significantly affect productivity.

In order to overcome this problem, the bulk emulsion technology was introduced in the UAF project. As the bulk emulsions and gasifiers are not explosives before they are mixed during charging of the blast holes, they can be safely stored on site in separate tanks, and delivered to the blasting face by trucks. Charging of blast holes is done using a mobile charging unit, which mixes and pumps the explosives into the blast holes. This operation is highly mechanized and greatly improves productivity.

For high explosive items such as detonators and primers, a temporary storage cavern was constructed on site licensed. This combination of bulk emulsion and temporary cavern storage of explosives on site contributed greatly to increased productivity in tunnel blasting, saved millions of dollars in explosives storage and handling, and improved safety on site and eliminated the safety risks of transporting explosives on public road on a daily basis. Ventilation time is also reduced due to the reduced toxic gas from blasting.

This bulk emulsion technology and practice of on-site storage was later adopted for the JRC project. For the JRC, booster charges and detonators were stored in an engineered concrete storage structure. For the JRC, on-site storage meant even greater savings in transport cost because of the high security control on Jurong Island.

5 INSTRUMENTATION AND MONITORING OF ROCK CAVERNS IN GRANITE ROCK

The performance of the tunnels and caverns were monitored during construction by measuring the deformation, convergence, rock bolt loads, as well as 3-D stress measurements (Zhou et al., 2004). Deformation results from numerical analyses and monitoring have shown the possibility to reduce the number of rock bolts without compromising on tunnel stability due to the high horizontal stresses (Zhou, 2002). Numerical modelling and instrumentation of deformation and bolt loads both show that the relatively high horizontal stresses were favorable to tunnel stability. These results have been used to optimize the dimension and support design in later parts of the project.

Three types of instrumentation and monitoring were carried out during construction. They are: multi-point borehole extensometer (MPBX), bolt strain, and tape convergence. This section describes the instrumentation results of one cavern.

5.1 Cavern excavation

The cavern being instrumented is approximately 100m long and 30m wide with a maximum crown height of about 12.5m. The long axis of the cavern is roughly along the N-S direction. Rock support for the caverns include fully grouted rock bolts of 5-m long spaced at 1.5 m to 2.4 m, and steel-fibre reinforced shotcrete of 50-150mm, depending on the rock mass quality.

The cavern was excavated using a top heading followed by horizontal benching with a design advance of 5m for both (Fig. 8a). The top heading consists of a pilot tunnel on the right, followed by a slash of a similar section area on the left. After each pilot and side slash combination, fibre-reinforced shotcrete was applied followed by rock bolts. The top heading was carried out from one end of the cavern to another before benching. Once the heading was completed and all rock support applied, benching was carried out, again from one end of the cavern to another. The height of the bench is about 3.5 m. Due to the limited bench height, horizontal benching was chosen to allow use of the same drilling jumbos.

5.2 Combined surface MPBX and internal MPBX section

To monitor cavern deformation a number of multiple-position borehole extensometers (MPBX) were installed in several caverns.

In one cavern, a 4-point surface MPBX was installed from a vertical borehole drilled from the surface about seven weeks before excavation to measure the “absolute” deformation of the rock above the cavern crown (Fig. 8b). The vertical borehole was located at the 50-m chainage and drilled directly above the centre of the 100m long cavern with the end of the borehole at 2 meters above the cavern crown. The initial reading due to setting of the grout reached a stable -0.46 mm about eight days after installation, signalling completion of shrinkage deformation of the grout.

Another section consisting of five 4-point MPBX (referred to as the internal MPBX section) was instrumented at the same location of the long-hole immediately after top heading passed by. This section was installed about 58 days after start of the cavern excavation, or about 100 days after installation of the surface MPBX. Figure 4 shows the combined MPBX section and the relative reference points of the various anchor points. The measurement points in BH1 of the internal section points were placed at the same height of the surface MPBX for comparison purposes.

5.3 Rock deformation

Figures 9 & 10 show respectively the deformation measurements by the surface MPBX and the internal MPBX (BH1), where the excavation progress is plotted on the secondary vertical axis. In both figures, negative deformation means upward (crown) or outward movement (walls) while positive deformation means the respective downward and inward movements.

Excavation of the cavern started about 45 days after installation of the surface MPBX. Figure 9 indicates that the rock above the cavern started to deform (after self-stabilisation of extensometers due to grout setting) almost immediately after start of cavern excavation. The deformation reached a peak a few days after the top heading reached the location of the MPBX. As the excavation continued, the deformation started to decrease (moving downwards), and then reached a stable condition about a few days after the final slash.

Benching started almost immediately after completion of the top heading. From Figure 9, bench excavation had an insignificant effect on the rock deformation. At the 50-m point the deformation curve showed a little kink and then quickly recovered to the previous level and stabilized. Points 6m, 12m, and 25m show a final upward movement, while Point 2m (nearest the cavern crown) shows a slight downward final deformation of about 0.24 mm. Overall, the results show that the cavern crown moved slightly upwards after cavern excavation, suggesting that the cavern crown is in compression, a favorable condition for rock stability.

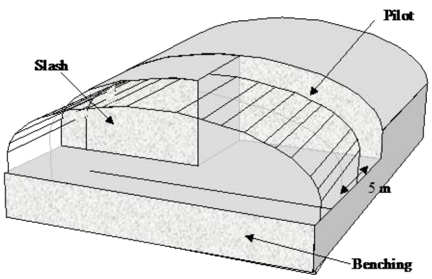
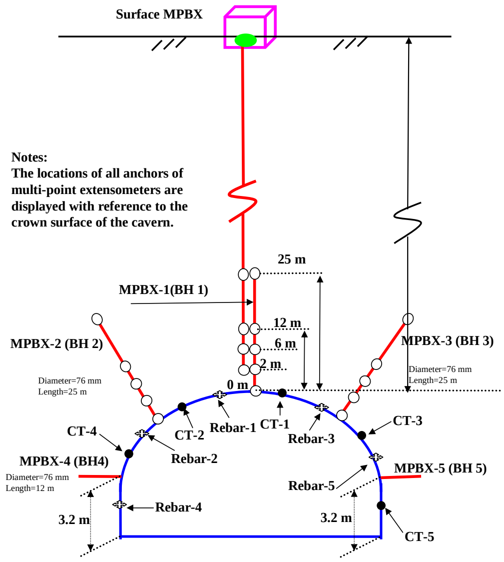


Figure 8. a) Cavern excavation sequence by top heading followed by benching



8 b) Combined MPBX section in a 30-m wide cavern

Figure 10 (Internal MPBX) shows a similar pattern of deformation as Figure 9, with the maximum deformation occurring about seven days after the excavation passed the installation location. The same “kink” in the deformation curve was also detected.

Obviously, the positive deformation shown by Points 0-m and 2-m does not mean that the cavern crown has moved downward from its original condition prior to excavation. What was measured was the recovery of the roof position downwards from the upward movement created by the top heading excavation before it reached the internal MPBX location. This observation is important, as most monitoring of tunnel deformation is based on similar conditions, i.e. deformation measurement device being installed after excavation. Obviously, in situations where the horizontal stresses are significantly higher than the vertical stress, the measured downward deformation may not necessarily mean that the roof is actually moving downward from its original position before excavation.

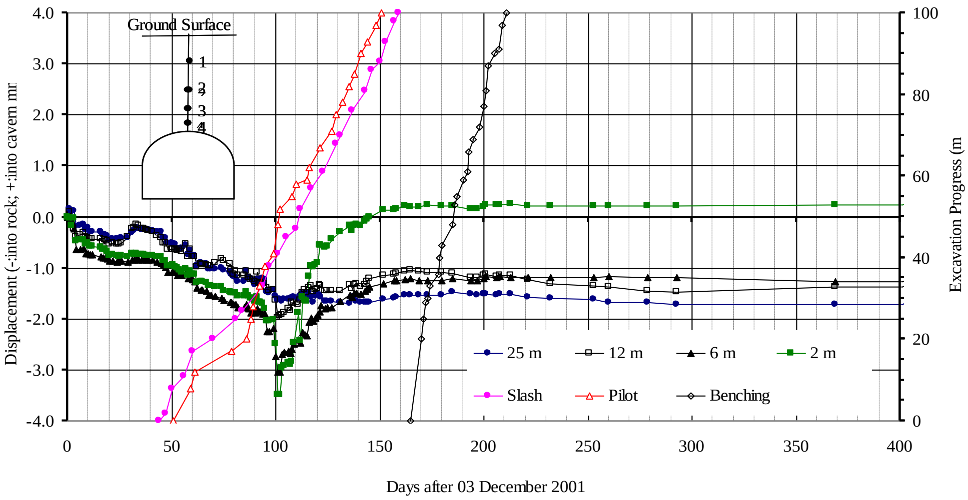


Figure 9. Measured deformation in internal MPBX

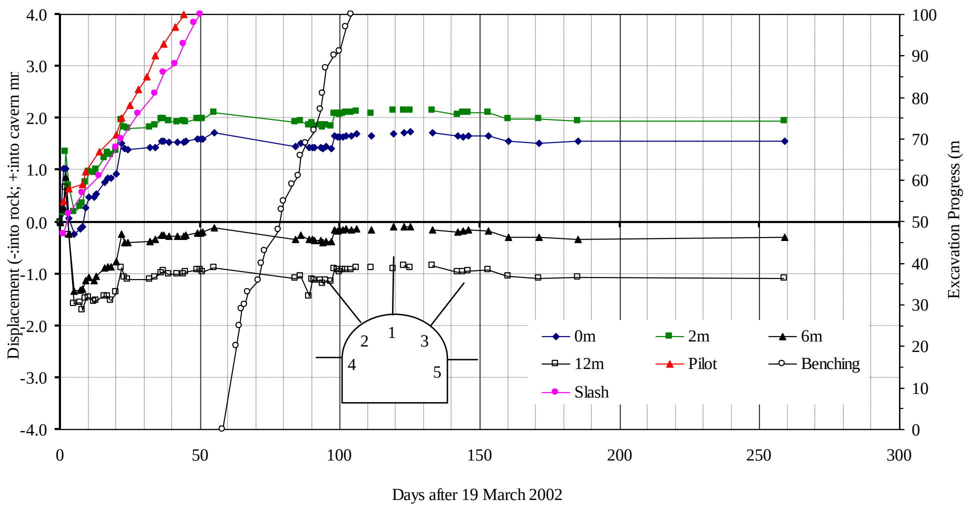


Figure 10. Measured deformation in internal MPBX

5.4 Load on Fully Grouted Rock Bolts

To check the performance of the rock bolts, a total of five sections containing a total of 22 rock bolts have been instrumented with strain gauges. The instrumented rock bolts were of the rebar type, 5m long and with four strain gauges at 1 m apart. The load measurement accuracy was about ± 1.0kN. Figure 11 shows the results of a typical bolt load calculated from measured strains.

There is no consistent pattern of where the maximum load would occur along the length of the rock bolt. This suggests that the load on the rock bolts were most likely caused by localised rock joint movements, rather than by overall roof deformation since the bolts were all fully grouted. In general, the maximum loads on the rock bolts were within the range of 20-60 KN, about 10-25% of the design strength of 250 KN. These results are consistent with a study reported by Broch et al (1996) in their extensive study of the rock support for a 61-m span rock cavern in Norway.

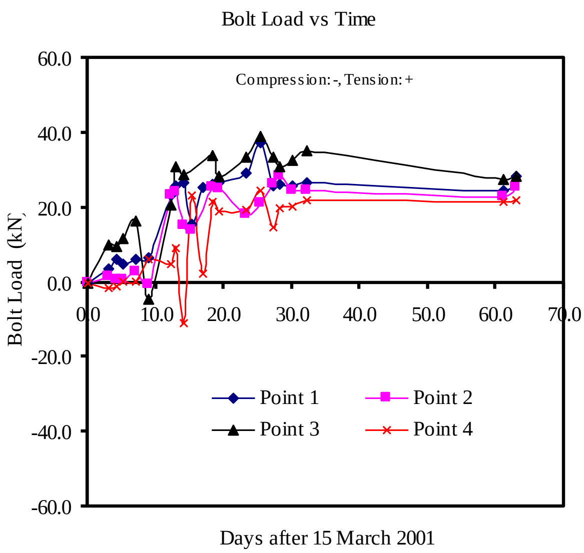


Figure 11 Measured loads on fully grouted rock bolts

5.5 Implications for cavern stability and rock reinforcement

While the magnitude of the measured total deformation was small, the trend of the deformation on the cavern was significant. When the final deformation near the cavern crown is upward, it indicates that the rock above the cavern crown is in compression, an ideal situation for roof stability as the rock is very strong in compression and much weaker in tension.

The upward movement of the cavern crown and inward movement of the walls also suggests that the cavern shape could be further optimized from a stress distribution point of view. A rule of thumb used in designing the crown arch height is to use 0.2

times the cavern span. For large caverns and tunnels, this has often resulted in wasted excavation space. An optimised cavern shape means that the excavation volume could be reduced.

All rock bolts only use up to about 25% of the design loads. This means that both the number of rock bolts and lengths could be reduced, or optimised to reinforcement areas where potential wedges could be identified.

7. CONCLUSIONS

With good engineering and a practical approach, it is possible to develop cost effective solutions in cavern space creation. The experience in the two large rock cavern projects have demonstrated the usefulness and importance of adopting a systematic rock engineering process. This process plays great importance on the contribution of engineering geology, and the use of geological and structural modelling and site investigations as part of the planning and design process. An important concept in achieving cost effective solutions in cavern construction is to help the rock mass support itself, combining temporary and permanent support without the need for concrete lining. Drained rock support, combined with pre-injection grouting to pre-determined seepage criteria, has also proven a viable alternative to water control. The empirical design approach based on rock mass classification, supported by numerical modelling, key block analysis, and instrumentation, has proven to be viable for the design and construction of rock caverns. The relatively high horizontal stress in the Singapore rocks has proven to favorable for cavern crown stability and allows for optimization of support design. Other innovative approaches include use of bulk emulsion for blasting and on-site storage of high explosives.

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