Sustainable Design Approaches for Large Span Rock Caverns in Hong Kong

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ABSTRACT

It is essential to create new land for the sustainable development of Hong Kong, and cavern development is one of the viable options to provide valuable land for multiple functions such as accommodation of various facilities, provision for service reservoirs, etc. To ensure the constructability of such large caverns, the location of these caverns would be carefully chosen based on their geological compatibility; however, obstacles remain in their design and construction processes. This paper discusses the various design challenges to overcome unexpected issues encountered during the construction of Hong Kong's largest rock cavern under the project 'Joint Cavern Development at Anderson Road Quarry Site', which four large caverns of up to 36m span were designed with multiple adits connected, forming a complex network of underground openings. Also, the design approach that is adopted to ensure a high level of confidence for construction while maintaining cost-effectiveness of the project.

1 INTRODUCTION

To support the sustainable development of Hong Kong, the Government of the Hong Kong Special Administrative Region (HKSAR) adopts a multi-pronged approach to enhance land supply. The topography of Hong Kong, characterized by a predominance of igneous rock formations, offers a conducive environment for the formation of rock caverns. Relocating suitable government facilities into caverns offers various benefits. It can release existing surface sites for other valuable uses, while relocating facilities that are incompatible with the land uses and environment nearby, and improving the urban layout and environmental quality of the area. In addition, cavern environments have been proven ideal for housing facilities requiring secure and stable operating conditions, such as testing laboratories, archives, data centers and warehouses, etc. These facilities are protected from natural or man-made disasters by the strong cavern walls and sheltered from being affected by sunlight and weather changes, hence the temperature and humidity in caverns are relatively stable. The reduction of power consumption from cooling systems and carbon emissions during operation of the facilities does not only reduce the operational costs but also fosters Hong Kong to become a green city.

2 PROJECT BACKGROUND

The project 'Joint Cavern Development at the Anderson Road Quarry Site' aims to relocate the Public Works Central Laboratory (PWCL) of the Civil Engineering and Development Department (CEDD) and establish a new Archives Centre (AC) for the Government Records Service within caverns. This initiative addresses the growing demand for public services while leveraging the inherent advantages of cavern environments, which offer enhanced security, energy efficiency, and stable conditions ideal for laboratory and archival operations. Additionally, after relocation of the existing facilities, valuable urban land could be released for other socio-economic developments after relocation of the existing facilities.

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The project comprises of construction of a cavern complex and connection adits inside the mountain body of Tai Sheung Tok at the Anderson Road Quarry Development site. Total four (4) caverns (PWCL-North, PWCL-South, AC-North and AC-South) are being constructed with maximum excavation span of 36.2m and maximum excavation height of 30.0m. The project layout plan and two of the excavated caverns are shown in Figure 1.



Figure 1. Project Layout Plan and Two of the Excavated Caverns

China State - Alchmex Joint Venture (CSAJV) is the main Contractor responsible for construction of these underground rock caverns. AtkinsRéalis is the Geotechnical Designer for under-ground excavation works. AECOM is the Project Manager for this project.

3 SITE GEOLOGY AND ROCKMASS PARAMETERS

The solid regional geology within the Project Site consists of the Early Cretaceous Lion Rock Suite, where primarily Mount Butler Granite, which is comprised of equigranular fine-grained Granite (Klb_gf), with equigranular fine to medium-grained Granite (Klb_gfm) in areas further to the west and north of the caverns is present.

All four caverns are located within the rock zone where the rockmass is predominantly Grade III or better rock with rockmass Q-value in range of 0.7 to 10.0 interpreted from boreholes. The anticipated rockmass condition is shown in Figure 2.

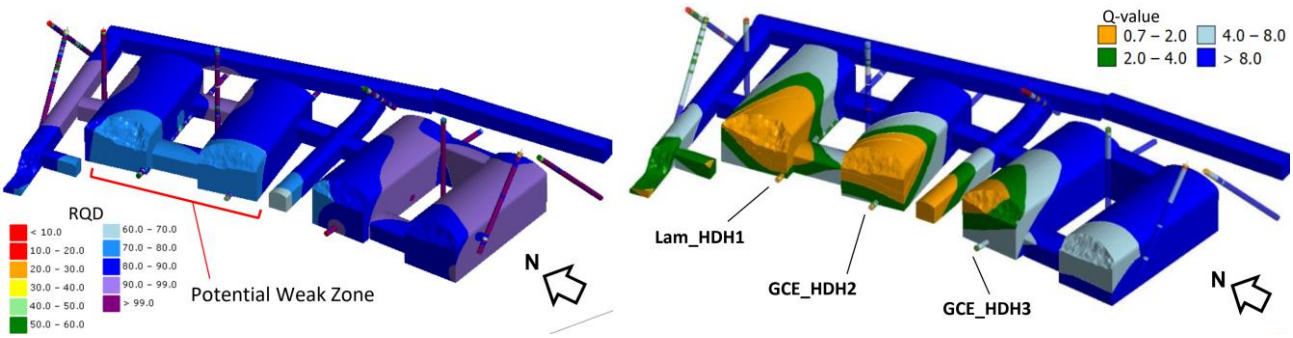


Figure 2. RQD and Q-Value Classification for Four Caverns

Rockmass parameters for geotechnical design works are derived from Ground Investigation (GI) records and laboratory tests. A total of 170 existing GI was reviewed, including their respective in-situ and laboratory testing, where available, and the findings of these data were appraised by an additional 28 GI locations, including 4 trial pits, 1 HDC, 1 inclined drillhole and 22 vertical drillholes. Project-specific field and laboratory testing included 42 lugeon and 6 televiewer tests and 188 rock and 9 soil geotechnical tests respectively. The rockmass parameters adopted for rock support design are presented in Table 1.

Table 1: Design Parameters of Rockmass

Rock Type	Bulk Unit Weight (kN/m^3)	Poisson Ratio	Intact Rock UCS (MPa)	Young's Modulus (GPa)	Tensile Strength (MPa)
Slightly Decomposed Granite (SDG)	26	0.30	140	64	13
Moderately Decomposed Granite (MDG)	26	0.30	94.6	47	12

The rockmass design parameters are derived based on the Generalized Hoek-Brown Strength Criterion:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b\frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \quad (1)$$

$$GSI = 9\ln Q' + 44 \quad (2)$$

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \quad (3)$$

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \quad (4)$$

$$E_m = E_i\left(0.02 + \frac{1-\frac{D}{2}}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right) \quad (5)$$

where σ1' and σ3' = Maximum and minimum effective principal stresses at failure (kPa), σ_{ci} = Uniaxial compressive strength of the intact rock pieces (kPa), m_b = Value of the Hoek-Brown constant of the rock mass, s and a = Constants which depend upon the rock mass characteristics, GSI = Geological Strength Index, Q' = (RQD/Jn) × (Jr/Ja), m_i = Material constant, m_b = Reduced value of material constant, s = Rock mass constant, D = Disturbance factor, E_m = Rock mass modulus (in GPa), and E_i = Intact rock modulus (in GPa).

Rock discontinuity sets are established from Borehole Televiewer Data (BHTV) and their shear strength parameters are derived from lab tests, as summarized in Table 2. These joint sets were adopted for initial design. During the construction stage, rock discontinuity sets were constantly mapped after each blast cycle and design verification was carried out accordingly.

Table 2: Major Joint Sets and Shear Strength Parameters of Rock Joints

Major Joint Set	Dip (°)	Dip Direction (°)	Joint Persistence (m)	Cohesion (kPa)	Peak Friction angle (°)	Residual Friction angle (°)
J1	57	328	15	0	45	40
J2	43	064	10	0	45	40
J3	88	261	10	0	45	40

4 KEY CHALLENGES AND THE COST-EFFECTIVE DESIGNS

During the design development stage and construction stage, several challenges are faced by the Contractor and Designer. Some of the key challenges and the cost-effective design solutions are discussed below.

4.1 Unique Scheme of Cavern Complex with Special Excavation Sequence

To ensure adequate rock cover and access to high-quality rock mass, cavern complexes are typically situated deep within hillsides and connected to surface portals via long adits or tunnel sections. These connecting structures often traverse soft or mixed ground conditions, which can lead to increased construction

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complexity, and higher costs. Additionally, extended access routes reduced long-term operational efficiency and pose challenges for emergency evacuation and rescue operations, particularly in fire scenarios.

Recognising these limitations, the project site was chosen to be a quarry site during the initial site selection exercise. The existing rock slope, formed by previous quarrying activities, offers an ideal portal location with competent rockmass conditions. This enabled a cost-effective layout that eliminates the need for permanent connecting adits or tunnels. Instead, the portal excavation is carried out at the face of the rock slopes, and an integrated building design is adopted, combining the surface portal facilities with underground cavern structures (Figure 3).



Figure 3. Integrated Building at Portal and inside Caverns

All cavern portals are located within 30m to public road, necessitating robust mitigation measures to minimise risks and nuisances associated with flyrock, dust, and noise generated during blasting excavation. Given the substantial height of the caverns (>30m), excavation was anticipated to be carried out in three benches: top heading, middle bench, and lower bench. A feasible excavation sequence must be developed to initiate works at the top heading level, approximately 17 meters above ground level.

To address public safety concerns and improve constructability, a unique construction sequence was adopted. The orientation of the entrance of construction adits is designed not to directly face towards public carriageway and residential development, in order to minimise blasting impacts. The blasting works are commenced from the rear of the cavern at the top heading level, while maintaining a minimum 5-meter-thick rock plug at the portal throughout all blasting activities. This approach effectively shields the portal from direct blast impacts. To access the rear top heading level, a network of small-diameter adits (span of 9.6m) was excavated, incorporating a longitudinal gradient of 1 in 8. This innovative sequence provided a key cost-effective solution to mitigate blasting risks and enhance constructability.

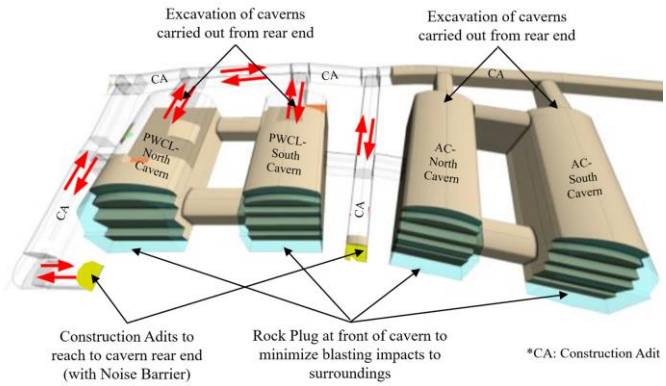


Figure 4. Cavern Excavation Sequence from Rear of Caverns, with maintaining A Rock Plug at Portal

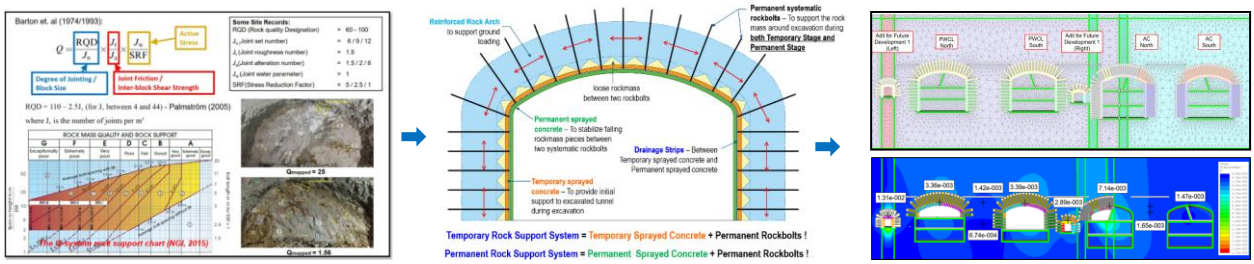
4.2 Large Span Excavation Design

The proposed caverns are massive in size, with a maximum excavation span of 36.2m and excavation height of 30.0m. One of the key challenges was to ensure a robust design while maintaining the cost-effectiveness. In Hong Kong, conventionally two stages lining would be designed to support temporary and permanent loading.

The initial stage lining comprised of rock-bolts and sprayed concrete are designed as temporary support while later stage lining using cast in-situ plain or reinforced concrete lining is designed as permanent support to take up all external loading. For such large excavations, this conventional approach requires huge number of materials for the cavern lining and a complex shutter is required to cast the permanent lining. Also, the initial stage lining installed to support the excavation is not utilized under permanent conditions. As a result, a more cost economic approach was explored to be adopted instead of the conventional design approach.

Based on the results of the ground investigation of the project site, the rock mass quality is generally good, and it is considered cost effective to adopt the rock reinforcement approach and utilize the inherent strength of the rock in the design of cavern supports. The rock reinforcement approach, which uses pattern rockbolts and shotcrete to reinforce the surrounding rock mass forming a rock arch structure to provide permanent support to the external loading instead of cast-in-situ concrete lining, thereby significantly reducing the overall lining thickness and the total volume of rock excavation. To ensure the permanent support structure fulfils the durability requirements and for 100-year design life, Double Corrosion Protection (DCP) rockbolts are adopted in the design. Spray applied waterproofing membrane is also used to achieve the watertightness requirements. In general, a three-stage design approach (Figure 5) is adopted to ensure the robustness, effectiveness and appropriateness of the designed supporting systems. The three-stage design approach is as follows:

Stage 1: Deriving initial support system from NGI Q-system Support Chart (2015);
Stage 2: Checking the suitability of the initial support system by Reinforced Rock Arch (RRA) theory, considering the thrust capacity of the reinforced rock arch against the thrust load induced by external loading. Increase the permanent support as necessary; and
Stage 3: Verifying the design by numerical analyses using Finite Element Modelling (continuum and discontinuum modelling). Increase the permanent support as necessary and confirm the permanent support requirements.



Stage 1 Stage 2 Stage 3

Figure 5. Three-stage Design Approach of Rock Reinforcement Design

Four rock support classes, as summarized in Table 3, are developed based on different Q-value ranges, representing different rockmass quality, for the above-mentioned three-stage design.

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Table 3: Permanent and Temporary Rock Support for Caverns

Support Class	Rockmass As-mapped Q Value	Unsupported Length, m	Stand-up Time, Hours	Rockbolts [1]	Temporary Sprayed Concrete (mm)[2] Roof/Wall	Permanent Sprayed Concrete (mm)[3] Roof/Wall
SC-I	Q≥8.0	6m	72	5.8m Long PB1* at 1.9m C/C spacing	80(E500)/ 55(E500)	135(E700)
SC-II	8.0>Q≥4.0	6m	72	5.8m Long PB1* at 1.7m C/C spacing	95(E700)/ 60(E500)	135(E1000)
SC-III	4.0>Q ≥2.0	6m	55	5.8m Long PB1* at 1.5m C/C spacing	110(E700)/ 75(E500)	150(E1000)
SC-IV	2.0>Q ≥0.7	3m	50	5.8m Long PB1* at 1.3m C/C spacing	140(E700)/ 110(E700)	250(E1000)

[1] PB1: Permanent Rockbolts of Tensile Capacity of 204kN (with Safety Factor of 2.0)
[2] Fiber Reinforced Temporary sprayed concrete of grade C35/45 is applied within standup time.
[3] Fiber Reinforced Permanent sprayed concrete of grade C35/45, which is applied after installing the drainage strips, at least 20m behind the excavation face.

4.3 Complex Junction Network with Multiples Junction Openings

The four caverns are connected by several adits to facilitate construction logistics. With these adits all around the caverns, a complex junction network is formed with several junction openings around the cavern. The design of those openings with more precise ground stress distribution was deemed necessary. The conventional 2D design with Jnx3 to consider for junction effect might lead to a conservative design with higher density of rockbolts and thicker sprayed concrete lining, and thus adversely affect the construction cast and time. The rock pillar stability around these multiple openings is also uncertain with conventional 2D analysis which cannot consider the ground stresses distribution realistically for such complex cavern network under planned excavation sequence.

To alleviate the concerns, a 3D model is adopted to design the rock support system around the junction openings. A multistage 3D model (Figure 6) is set up with multiple staged construction according to the actual sequence adopted on site, such as multiple heading and benching sequence, to assess the ground stresses distribution around junction openings. With the help of 3D analysis, change in ground stresses and rockbolt forces are effectively evaluated for excavation stages before and after junction openings. Also, with the help of the 3D model, rock pillar stability has been checked for all critical locations, which otherwise is difficult to check with the conventional 2D model.

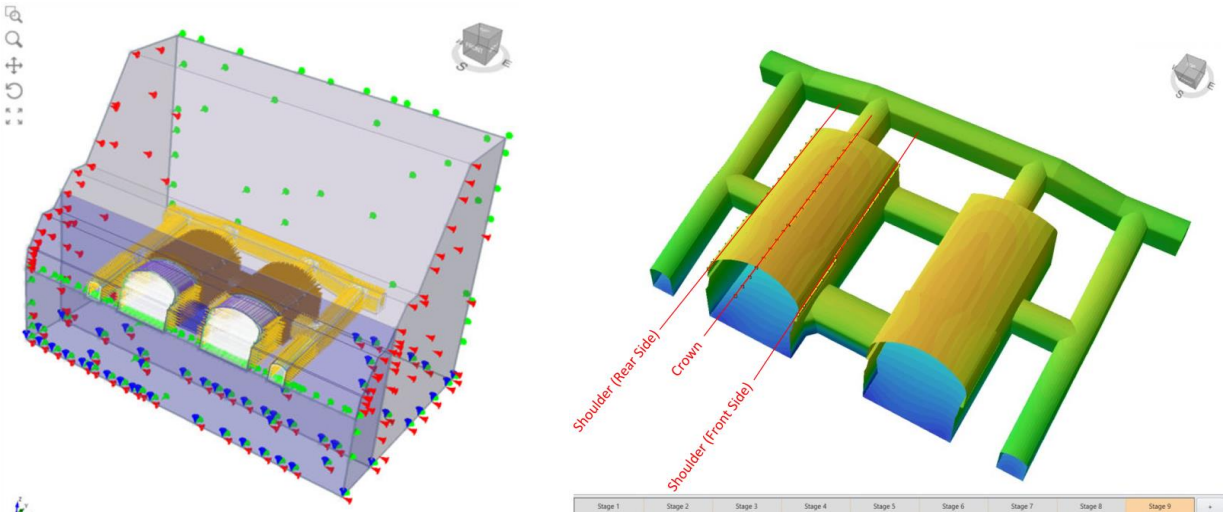


Figure 6. Staged 3D Finite Element Method (FEM) Analysis for Multiple Junction Openings

4.4 Shallow Rock Cover at Portal Area

At the cavern portals located at the face of the rock slopes, the rock cover above the cavern crown is comparatively shallow. The rock cover above the cavern crown is only 5 to 6m (less than 0.5 cavern span) imposing design concerns for instability during large cavern excavation. Also, the rockbolts of effective length cannot be installed at cavern roof due to such shallow cover, adding another challenge for cavern support design. Multi-drift excavation is designed for this shallow rock cover location so that excavation can be carried out safely with less ground deformation and the rock conditions can be mapped in advance. To mitigate risk of instability during excavation, pre-support (pipe umbrella) is provided along the cavern crown to maintain the crown stability for interim stage between excavation and installation of support. For the cavern crown, composite lining comprised 900mm thick fiber reinforced sprayed concrete and galvanized lattice girders is adopted. Whereas for the sidewalls, permanent rockbolts and fiber reinforced sprayed concrete are designed. The rock supports for caverns at shallow rock cover are summarized in Table 4.

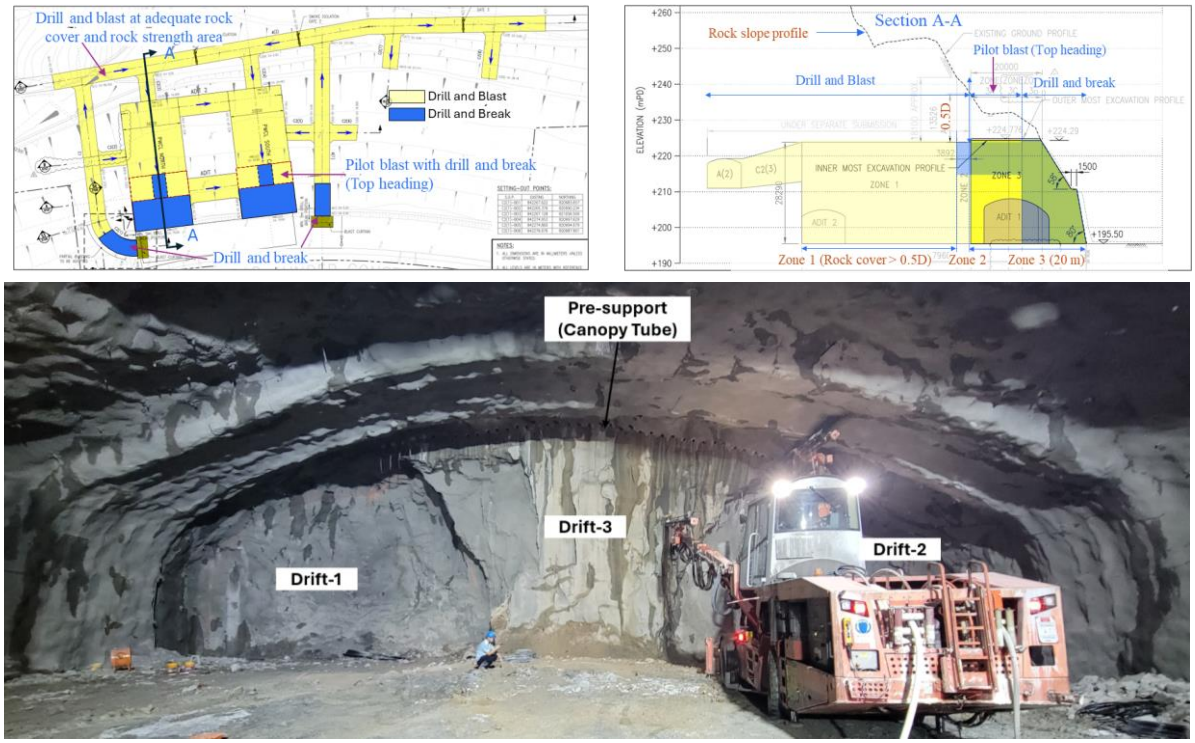


Figure 7. Multi-drift Excavation with Canopy Tubes (Pre-support) for Shallow Rock Cover Location

Table 4: Permanent and Temporary Rock Support for Caverns at Shallow Rock Cover Location

Support class	Mapped Q Value	Unsupported Length	Stand-up Time	Rock Support-Roof	Rock Support-Shoulder	Rock Support-Sidewall
Portal Area (Z3SC-IV)	Q≥0.7	3m	50 Hrs	Primary Lining [(2)]: 500mm FRSC with Lattice Girder at 1.0m C/C Spacing; + Secondary Lining [(3)]: 400mm FRSC	Primary Lining [(2)]: 500mm FRSC with Lattice Girder at 1.0m C/C Spacing + 5.8m Long PB1[(1)] at 1.5m C/C spacing; + Secondary Lining [(3)]: 400mm FRSC	Primary Lining [(2)]: 250mm FRSC + 5.8m Long PB1[(1)] at 1.5m C/C spacing; + Secondary Lining [(3)]: 400mm FRSC

[(1)] PB1: Permanent Rockbolts of Tensile Capacity of 204kN (with Safety Factor of 2.0)
[(2)] Primary lining is installed during cavern excavation, which is applied within standup time.
[(3)] Secondary lining is installed after completion of all cavern excavation.

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4.5 Weak Zone at AC-South Cavern

During the construction stage, additional ground investigation was carried out to further verify the anticipated ground conditions. 28 additional drill holes were carried out. Among 4 caverns, one of the caverns (AC-North Cavern) encountered hints of weak seams in those additional GI. A combination of highly to completely weathered granite, core loss and highly fractured rock mass are recorded between 35m and 50m depth for drill holes no. E1, F1 and G1 as shown in Figure 8.

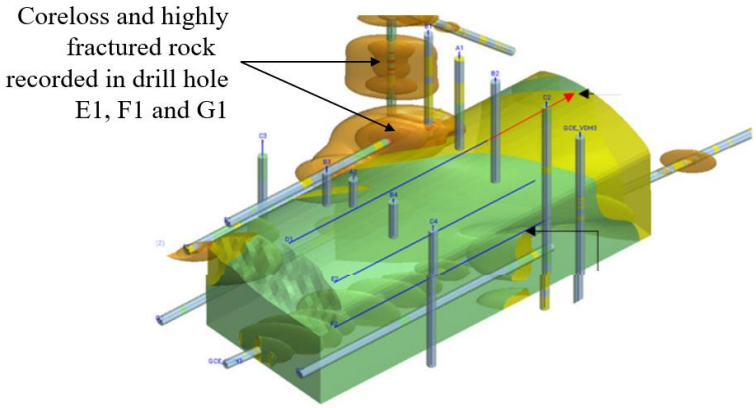


Figure 8. Project-specific Additional GI and Inferred Weak Zone for AC-North Cavern

The weak zone consists of a family of weak seams of various width. In total, the maximum width is ~8m to 10m, intermittent with competent Grade III granite. The inferred weak zone is attributed a design Q-value in range of 0.1 to 0.5. As a result, two additional rock support classes (SC-V and SC-VI), comprising shotcrete with galvanised wire meshes, are designed to cover the temporary and permanent rock support system for this weak zone. 3D modelling is adopted to simulate the 3D stresses distribution from the weak zone to the surrounding better rockmass. Prior to excavation, pre-excavation grouting was carried out for this localized weak zone to prevent the water ingress and ground lost inside the cavern. Spile bars are designed to provide as pre-support to prevent over caving during excavation in the weak zone area. Also, denser permanent bolts and thicker shotcrete lining with reinforcement bars are designed to provide sufficient strong support to the excavation.

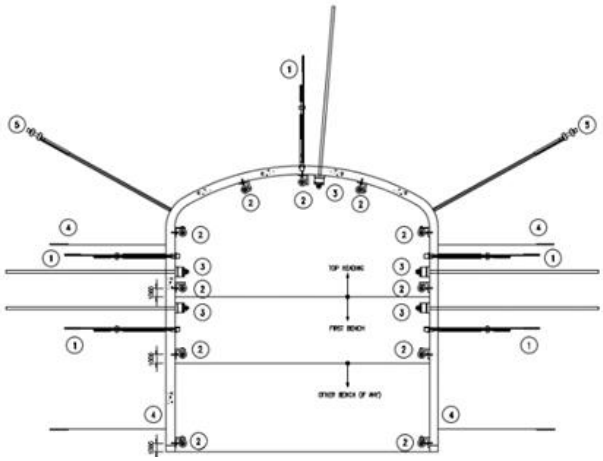
4.6 Advanced Geotechnical Monitoring

An integrated automated monitoring system is utilized to verify the design assumptions and geological/geotechnical design parameters adopted in the three-stage design approach of rock reinforcement design. The groundwater monitoring through piezometers checks that the drop in water levels from excavation stays within safe limits, protecting nearby structures. Conventional convergence monitoring arrays and extensometers are also deployed. Advanced instrumentation such as optic fibers are installed to monitor the ground deformation during cavern excavation. The cavern convergence was monitored throughout the construction period and compared with the convergence estimated from the design analysis. This helps validate numerical models and keeps displacements below acceptable levels. Furthermore, load cells on rock bolts monitor the performance of the support system, confirming effectiveness and identifying when reinforcement is needed. The GNSS (Global Navigation Satellite System) platform enables real-time, continuous monitoring of settlement and displacement. Using a network of receivers, it delivers accurate, all-weather data collection, removing the safety risks of manual surveys in dangerous areas. In addition, the Monitoring Array Platform combines real-time data from various co-located instruments, including multipoint borehole extensometers, load cells, strain gauges on rock bolts, and piezometers. This enables a correlated analysis of deformation, structural loads, and pore pressure. Figure 9 shows the location plan within PWCL and AC caverns for instrumentation and monitoring, and the typical arrangement of monitoring array is shown in Figure 10. The instrumentation within and nearby PWCL and AC caverns are summarized in Table 5, while

the estimated convergence prior to excavation and actual measured convergence during excavation are summarized in Table 6.



Figure 9. Monitoring Location Plan Within Caverns



Legend:
1. Multipoint Borehole Extensometer
2. Optical Prism Along the Excavation Profile
3. Load Cell & Strain Gauge
4. Vibrating Wire Piezometer
5. Doorstopper Strain Gauge

Figure 10. Typical Arrangement of Monitoring Array

Table 5: Summary of Instrumentation

Instrument	Quantities
Ground Settlement Points	30 nos.
Standpipes and Piezometers	3 sets
Convergence Points	63 sets
Monitoring Array	3 sets plus 1 additional set

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Table 6: Cavern Convergence Summary

Zone	Q-Value	Convergence estimated from FEM analysis	Convergence estimated from Empirical Methods	Actual convergence recorded on Site
Caverns with sufficient rock cover (> 0.5D cover, where D is span)	4-10	8mm	12mm	8mm
Caverns with shallow rock cover at Portal (< 0.5D cover, where D is span)	0.7-1.5	22mm	23mm	15mm

5 CONCLUSIONS

With innovative designs, cost-effectiveness can be secured without compromising the design robustness for complex underground excavation works. Design robustness can be achieved by implementing and verifying the design with multiple design approaches. This paper presents various design considerations for overcoming challenges for construction of large span caverns. Selection of appropriate geological design parameters and validation of those parameters with advanced instrumentation is crucial to ensure the design robustness. Additional ground investigation conducted during the construction stage provides better understanding of geological features with more accurate extent and width of geological features.

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