A Recent Case Study of Portal Cavern Design

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ABSTRACT

A new dual-two lanes tunnel of about 3.8 kilometers long was constructed in Kowloon East in Hong Kong recently. It forms part of a major strategic road network to provide an express connectivity and improve the traffic condition between Kowloon East and Kowloon West.

Two portal caverns, which are at the east end of the tunnels, are the first and largest of its kind with slender pillar constructed in highly fractured volcanic rock. A competent and optimised temporary cavern support design was required with the consideration of the pillar stability and construction logistics prior to the permanent support in place. A number of design reviews were carried out to suit the highly constrained construction sequence as the excavation works of the rock-cut slopes and the caverns were carried out concurrently. Some challenges that the project team had to deal with were installation of waterproof membrane and cast-in-situ reinforced concrete (RC) permanent lining for the crown that requires propping of steel shutter. Such challenges call for a cost saving design (CSD) with the use of sprayed waterproofing membrane and fibre reinforced sprayed concrete (FRSC) lining as the permanent support system for the portal caverns.

This paper discusses the optimisation of the temporary support design, the CSD for the permanent cavern support faced by the construction works, and the design methodology of both the temporary and permanent cavern support with the details of the application of the sprayed waterproofing membrane.

Keywords: Road tunnel, Weak rock, Cavern, Pillar, Lining cost saving design

1 Introduction

Located at the eastern end of the tunnels, the portal caverns are approximately 32 m high and 26 m wide with 11 m pillar separating the caverns. The purpose of constructing such large caverns at the portal (instead of further cut back of the rock slopes) was to minimise slope excavation works next to the seashore and hence reduce the environmental impact. As such, half of the ventilation building will be inside the cavern and the remaining half is outside.

The portal caverns were excavated predominantly in fine ash tuff belonging to the Mount Davis Formation. The tuff was mainly slightly to moderately decomposed with very closely- to closely spaced joints. The as-mapped Q'-value (where $Q' = RQD/J_n \times J_a/J_r$) varies from 1.22 to 2.44. The rock cover of the caverns ranges from approximately 3 m at the slope to 40 m at the end of the cavern.

2 Geological Setting

Based on the available ground investigation (GI) information and the published geological maps, the portal caverns are situated predominantly in Cretaceous volcanic rocks belonging to the Mount Davis Formation (Figure 1). Site specific GI indicates Grade II/III rock with closely spaced discontinuities. The apertures of the discontinuities are generally relatively tight with localised variations. Micro-fractures or incipient joints were observed in the core photos.

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A three-dimensional (3D) geological ground model was constructed based on the available GI information (**Error! Reference source not found.**). Two sub-vertical NE-SW and ENE-WSW trending g eological features comprising highly fractured rocks and localised Grade IV/V materials with limited extent (approximately 3 m width) were inferred at the site area. It was inferred that the rock mass quality, Q-value, in the vicinity of the caverns was expected to be better than 0.1 (where $Q = RQD/J_n \times J_a/J_r \times J_w/SRF$) with the consideration of the low rock cover and the existence of geological features.

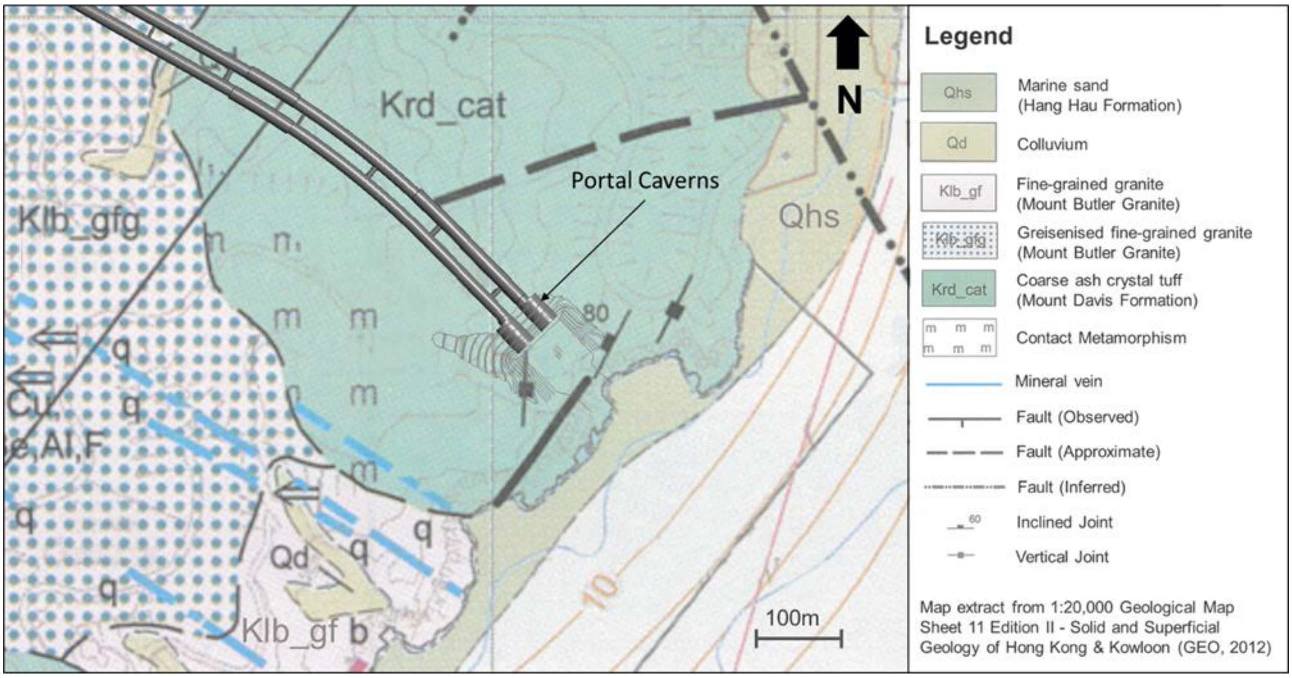

Figure 1: *Geological map of the site area*

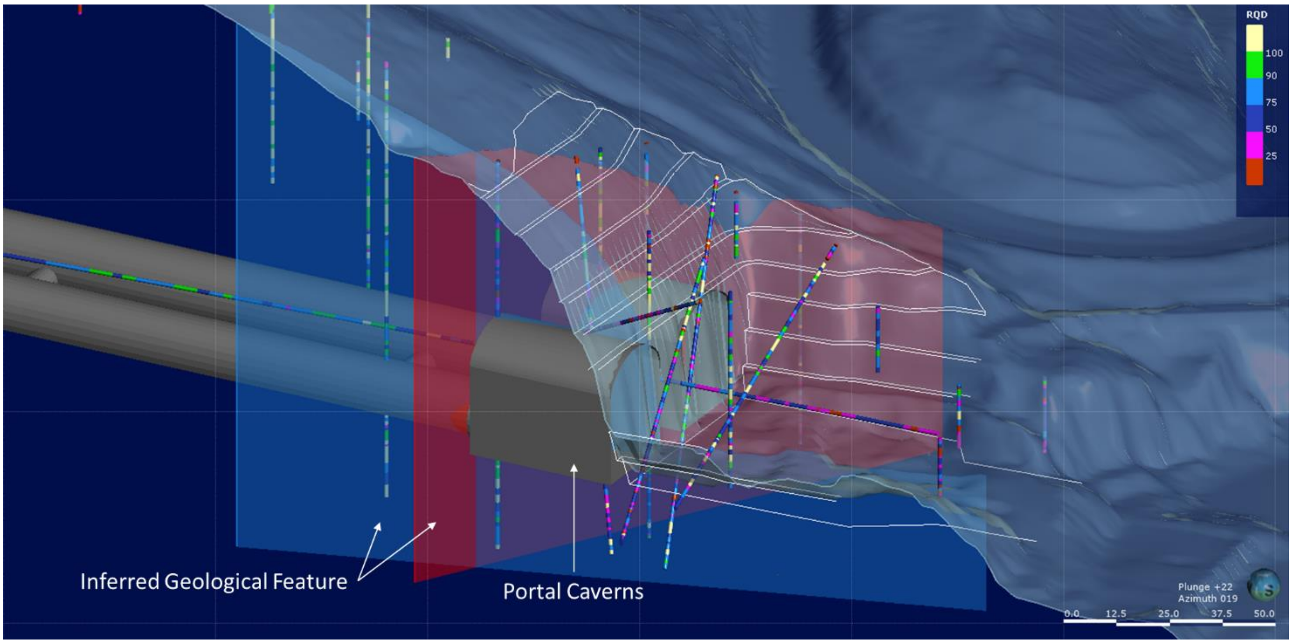

Figure 2: *3D geological model of the site area. Two geological features are shown in red and blue.*

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3 Cavern Temporary Support Design

The span and height of the portal caverns are approximately 25.8 m and 30.2 m respectively (including the 1.5 m thick cast-in-situ RC lining as per original design). The pillar width between the two caverns is approximately 11 m.

Aurecon was employed by the Contractor and responsible for optimising the original cavern temporary support design. The original temporary support for the rock caverns generally comprises systematic bolts and fibre reinforced shotcrete. However, this original design required the pillar to be supported by pre-tensioned stitching of rock dowels penetrating the pillar and locked by face plates with nuts on both ends. It also required both caverns to be excavated concurrently (

Figure **3**) so that the stitching bolts could be pre-tensioned from both caverns. These design requirements resulted in significant limitation to the overall construction program. For example, the caverns were required to excavated concurrently from the heading to the benching. Therefore, the key targets of this temporary support design were to optimise the pillar support and allow a greater flexibility of the construction sequence as preferred by the Contractor (

Figure **3**).

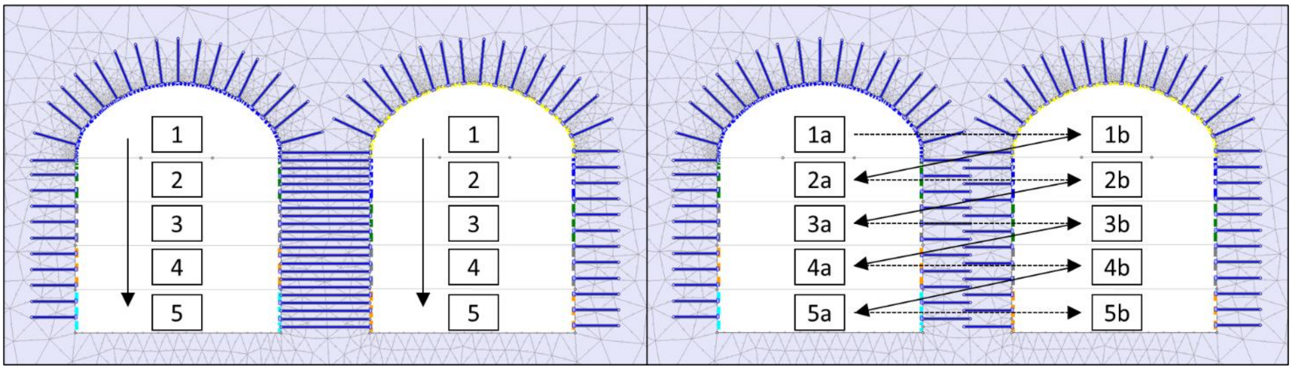

Figure 3: *(left) Original concurrent and (right) Contractor's preferred flexible construction sequence of the portal caverns*

3.1 Design Methodology

The cavern stability and the proposed temporary supports were assessed using continuum finite element analysis conducted by a two-dimensional (2D) numerical modelling program, *Phase2* (now *RS2*). Three support classes for $Q \geq 0.4$ (Support Class 1), $0.13 \leq Q < 0.4$ (Support Class 2) and $0.02 \leq Q < 0.13$ (Support Class 3) were proposed based on the predicted ground conditions. The Q-value which is used for classifying different support classes and tunnel face mapping is converted to Geological Strength Index (*GSI*) to derive the geotechnical design parameters of rock mass using the Generalized Hoek-Brown (GHB) Failure (Hoek, 1994; Hoek *et al.*, 1995; Hoek *et al.*, 2002; Hoek and Brown, 2019) based on the following equations and in the numerical model.

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_i \frac{\sigma_3}{\sigma_{ci}} + s\right)^a$$

where σ_1 and σ_3 are the major and minor principal stresses respectively, and m_i, s and a are the rock mass material constants given by

$$m_b = m_i \exp[(GSI - 100) / (28 - 14D)]$$

$$s = \exp[(GSI - 100) / (9 - 3D)]$$

$$a = 1/2 + 1/6\,(e^{-GSI/15} - e^{-20/3})$$

where *D* is a factor which depends upon the degree of disturbance to which the rock mass has been subjected to blast damage and stress relaxation.

A summary of the adopted geotechnical design parameters for the most possible expected ground conditions of 0.13 ≤ Q < 0.4 (Support Class 2) is presented in Table 1.

Table 1: *Adopted geotechnical design parameters for Support Class 2 for the portal caverns*

Parameters	Values	Remarks
Q-value, *Q*	0.13	$Q = RQD/J_n \times J_a/J_r \times J_w/SRF$
Joint water reduction factor, J_w	1	Dry or minor inflow
Stress reduction factor, *SRF*	5	Low rock cover condition
Q'-value, *Q'*	0.64	$Q' = RQD/J_n \times J_a/J_r$
Geological strength index, *GSI*	40	$GSI = 9 \ln(Q') + 44$
Intact rock constant, m_i	13	Recommended value for tuff
Disturbance factor, *D*	0	Mechanical excavation
Unit weight, γ (kN/m³)	27	From project-specific lab tests
Intact rock unconfined compressive strength (*UCS*), σ_{ci} (MPa)	150	From project-specific lab tests
Intact rock Young's modulus, E_i (MPa)	79,000	From project-specific lab tests
Poisson's ratio, ν'	0.3	From project-specific lab tests
m_b	1.52515	Calculated from GHB Failure Criterion
s	0.001273	Calculated from GHB Failure Criterion
a	0.511368	Calculated from GHB Failure Criterion
Rock mass *UCS*, σ_c (MPa)	4.96	Calculated from GHB Failure Criterion assuming $\sigma_3 = 0$
Rock mass modulus, E_{rm} (MPa)	12,612.5	According to Hoek & Diederichs (2006)

To model the 3D effect of the progressive cavern excavation and support the installation in 2D plain strain numerical analysis, longitudinal displacement profiles (LDP) based on the method by Vlachopoulos & Diederichs (2009) and Hoek *et al.* (2008) were established to understand the convergence behaviour of the cavern induced by cavern advance length. Ground reaction curves (GRC) were constructed to understand the required support pressure and convergence behaviour of the cavern under gradual increase of stress relaxation in cavern (i.e. a gradual decrease of in-situ pressure or internal pressure in cavern as the excavation advances) and different ground conditions using numerical analysis. Combining the LDP and GRC, the relationship between the degree of stress relaxation in cavern and the cavern advance length could then be obtained and applied in the 2D numerical analysis.

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3.2 Results

A number of numerical analyses were carried out for different pillar support and construction sequence for different ground conditions. Based on the numerical analysis results, pillar stabilization measure using 6.0 m long 25 mm diameter Grade 500 rock dowels (without pre-tension) in a staggered pattern with 1 m overlapping at the end of the rock dowels installed from the two portal caverns was adopted. This support measure also fulfilled the site constraint and preferred excavation sequence by the contractor. The results of the adopted numerical analysis of Support Class 2 are shown in Figure **4** and Figure 5. The results show that the maximum total deformation is approximately 17 mm at the cavern crown and the maximum shear strain of the pillar is approximately 0.3% and the cavern deformation is within the anticipated limit.

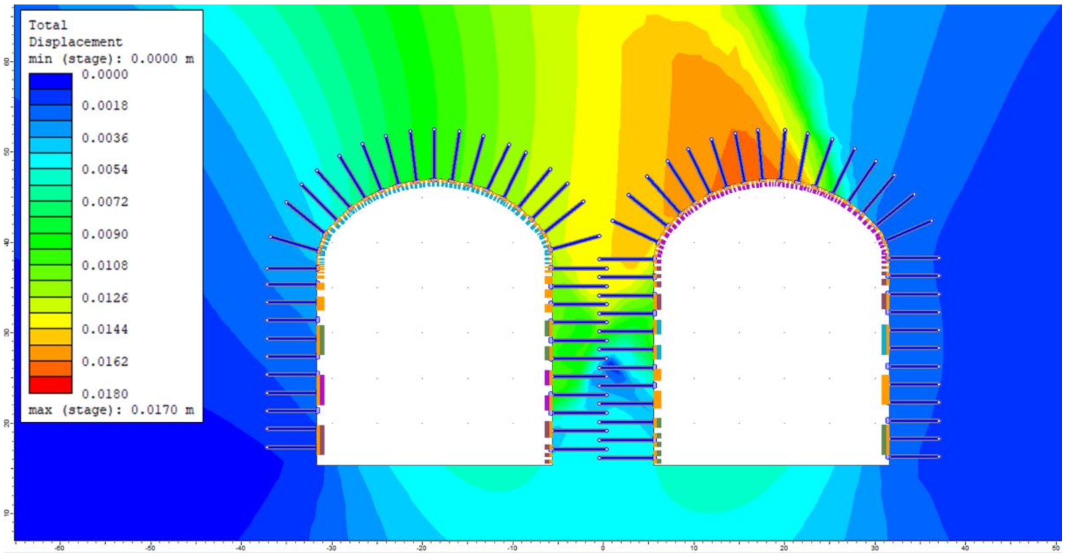

Figure 4: *Total deformation of the adopted numerical model of Support Class 2*

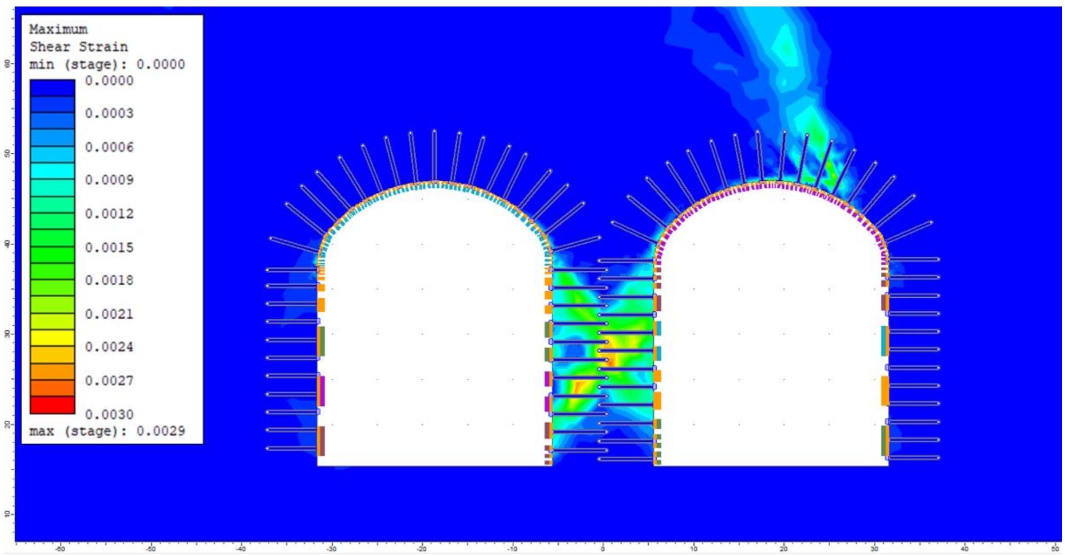

Figure 5: *Maximum shear strain of the adopted numerical model of Support Class 2*

The induced strain of the rock mass surrounding the opening was also assessed as suggested by Hoek (1998) and Sakurai (1983). The strain is defined by the ratio of convergence to cavern diameter. Sakurai

(1983) suggested that any tunnels with strain levels exceeding approximately 1.0% could be associated with tunnel stability problems and difficulties in providing adequate support. It was further supported by Hoek (1998) based on the plot showing the field observations during construction of three tunnels in Taiwan by Chern *et al.* (1998) against the rock mass uniaxial strength (σ_c) (Figure 6). The maximum strain of the portal caverns measured from the adopted numerical model of Support Class 2 is only approximately 0.1% while the rock mass uniaxial compressive strength calculated according to the Generalized Hoek-Brown Failure Criterion is 4.96 MPa. Therefore, as shown in Figure 6, the excavation of the portal caverns with the optimised pillar support and the Contractor's preferred flexible construction sequence is stable.

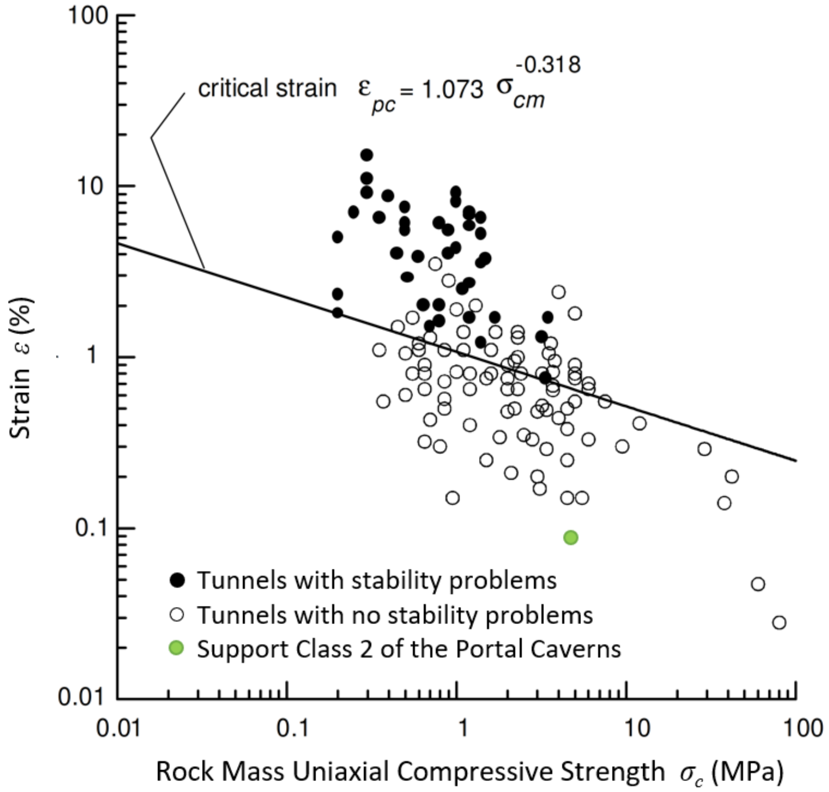

Figure 6: *Percentage Strain for different rock mass strengths (after Hoek (1998))*

4 Cavern Permanent Support Design

Apart from the temporary support design, Aurecon was also responsible for CSD of the permanent lining of the portal caverns. The conforming design of the caverns comprises 1.5 m thick cast-in-situ RC lining with sheet waterproofing membrane between the temporary and permanent lining for the wall and crown. However, the installations of the cast-in-situ RC lining and sheet waterproofing membrane for caverns with height up to 30 m are extremely difficult, especially at the crowns. RC lining at such height would involve a tremendous amount of scaffolding works from the crown all the way down to the invert, thus occupying the entire working space of caverns, which would seriously affect other construction works within the caverns such as the construction of the ventilation buildings. Hence it was proposed to optimise the cavern crowns from cast-in-situ RC lining and sheet waterproofing membrane to FRSC and sprayed waterproofing membrane.

4.1 Design Methodology

Numerical, non-linear analyses using finite element programme Strand7 were undertaken for the structural analysis of the permanent linings. The Strand7 programme utilises a 3D plate model (Figure

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7) with compression only support attribute whilst the surrounding soil-structure interaction medium is represented by a series of springs. The tensile capacity of these ground springs is ignored and therefore no reaction is given to the lining when the springs are in tension under the action of the loadings.

4.1.1 As-built Ground Condition

The permanent support design was commenced after the excavation of the top heading of the caverns. Therefore, the design was based on the actual ground conditions observed during the excavation of top heading. The caverns were divided into two sections with an adopted design *Q*-value of 0.22 and 0.3 (with the application of SRF of 5) and different rock cover.

4.1.2 Design Load

Different load combinations including dead load, earth load, wedge load, groundwater load, surcharge load, E&M load, earthquake load and fire load were considered in the numerical analysis.

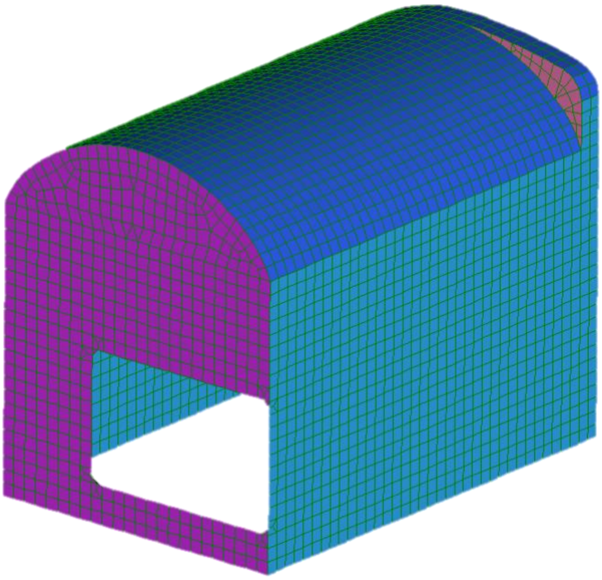

Figure 7: *Strand7 model geometry*

The earth load support pressure was estimated according to Grimstad & Barton (1993). This estimation often gives a conservative result. Therefore, in order to optimise the design, additional numerical analysis using Phase2 models was carried out. The model adopted the same design *Q*-values with corresponding temporary support and the preferred construction sequence, and the average contact pressures between the ground and the permanent lining extracted from the Phase 2 models were then used as input earth load in Strand7 structural model. A comparison of the earth load estimates according to Grimstad & Barton (1993) and those extracted from the Phase2 models are summarised in Table 2. The results show that the earth load extracted from Phase 2 models could significantly help optimise the support design while it was also considered that this load was more applicable and realistic since the numerical models considered the more realistic as-built ground and support information.

Table 2: *Comparison of the earth load estimated according to (Grimstad & Barton, 1993) and extracted from the Phase 2 models*

Design Q-value	Support Pressure (P_{roof}) according to Grimstad & Barton (1993) (kPa)	Average Contact Pressure extracted from Phase2 Model (kPa)

0.3	299	109
0.22	330	40

Note 1: Different rock covers of the two sections with the adopted Q-values of 0.3 and 0.22 were considered in the Phase2 models. The section with $Q = 0.22$ is closer to portal slope, hence rock cover and average contact pressure is lower.
Note 2: $P_{roof} = 0.2\ J_n^{1/2}\ Q^{-1/3} / 3 / J_r$ (unit in MPa)

Regarding the fire induced load, the design for the sidewalls and end walls, which would be cast-in-situ RC structure, was deemed-to-satisfy in accordance with the Hong Kong Code of Practice for Structural Use of Concrete. The nominal cover provided shall satisfy the fire limit state (FLS) design requirements. The deemed-to-satisfy approach of the Hong Kong Code of Practice Structural use of Concrete is not applicable to FRSC because there is no steel bar reinforcement in the FRSC so cover to reinforcement is not applicable. Hence, the design of the FRSC for the crown of the caverns under fire load assumed that a certain thickness of the lining would be structurally ineffective from fire damage due to the loss of stiffness and excessive spalling under high temperature. These structurally ineffective sections, which is still attached to the remaining lining, are being treated as superimposed dead load on the remaining structurally sound lining that is not damaged by fire. This design approach followed the 500°C isotherm method in accordance with BS EN 1992-1-2:2004.

The FRSC lining was designed for 4-hour fire resistance rating. The 4-hour ISO curve was chosen for the FLS design. The loss of strength of the FRSC lining at elevated temperatures was estimated using data for siliceous aggregates in BS EN 1992-1-2:2004. For the fire design case, the capacity reduction factor curves of the EC2 were used, and the damaged and structurally ineffective section was calculated to have an equivalent of 65 mm thickness. Another 10 mm thick of the lining was rendered ineffective on account of concrete spalling. The adequacy of the remaining lining after section reduction was analysed with the appropriate load and material factors according to the Hong Kong Code of Practice for Structural Use of Concrete.

To reduce the risk of concrete spalling for all structural concrete during fire, monofilament polypropylene fibres not less than 1.0 kg/m^3 shall be included in the concrete mix regardless of any thermal barrier to be installed. The fibres shall be 6 – 12 mm long and 18 – 32 µm in diameter, and shall have a melting point less than 180°C.

4.1.3 Ground Spring Stiffness

In the Strand7 model, the springs are located at the extrados of the shell elements to simulate elastic behaviour of lining-rock interaction. The springs represent the soil/rock medium and are modelled as “compression only” face support to the shell elements used to model the lining.

Tangential springs are applied in the crown only. These springs represent the resistance to the sliding action between the ground and the lining. These are modelled as beam elements acting as springs tangential to the lining profile. The magnitude of spring stiffness in the tangential direction is assumed to be 20% of the radial spring stiffness. This assumption is conservative and assumed to be the lower bound value suggested in “Design Recommendations for Concrete Tunnel Linings” (Paul *et al.*, 1983). The radial spring stiffness was calculated based on the equation suggested by Duddeck and Erdmann (1985); the spring model and the spring stiffness for straight wall were calculated based on Bowles (2001).

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4.2 Results

Based on the numerical analysis results, the conforming design using 1,500 mm thick cast-in-situ RC lining for cavern crown was successfully optimised to FRSC with thickness varying from 650 mm to 1,500 mm with local steel reinforcement using wire mesh near the portal and end wall (Figure 8).

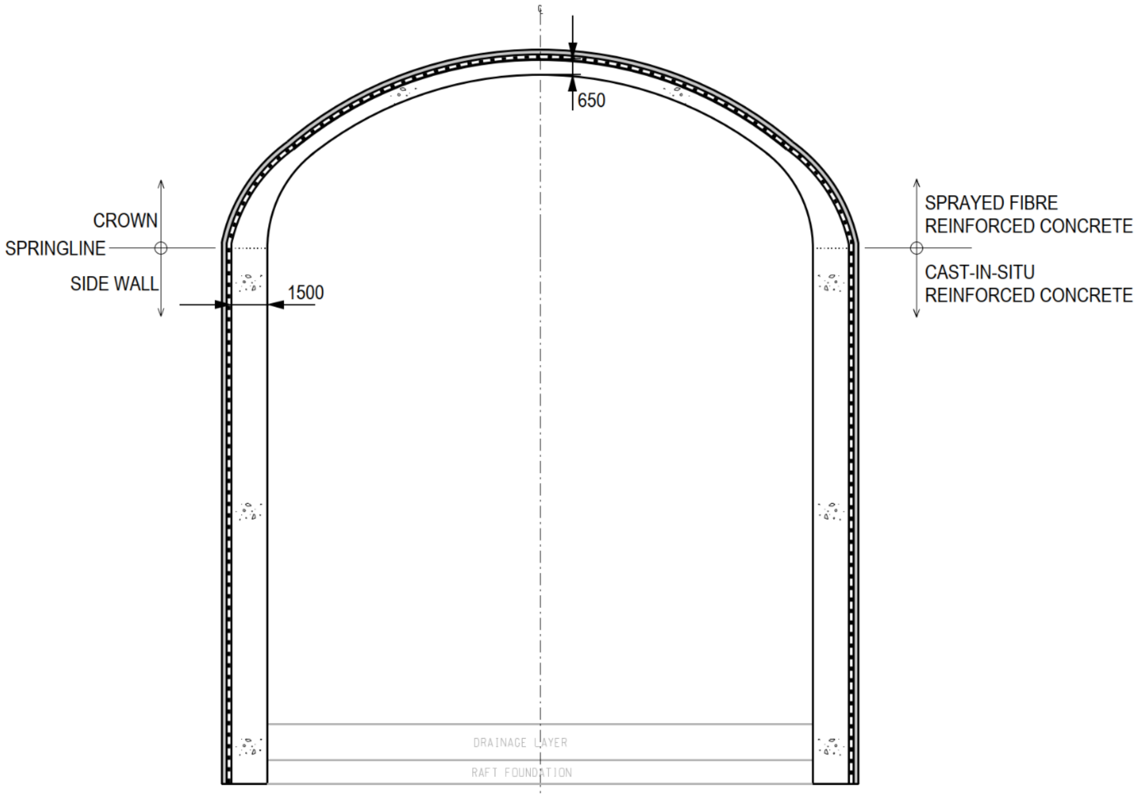

Figure 8: *General arrangement of optimised permanent lining for the portal caverns*

4.3 Waterproofing

Since the typical sheet waterproofing membrane applied between the temporary systematic dowel and shotcrete lining and permanent cast-in-situ concrete lining could not be used with permanent FRSC lining, it was proposed to adopt the sprayed waterproofing membrane for the cavern crown. The key requirements listed below were considered for successful spray-applied membrane installation in accordance with Geoguide 4:

- method suitable for excavation with limited water ingress;
- adequate substrate surface quality and preparation;
- adequate selection and maintenance of spraying equipment to promote adherence of membrane to the surface with minimum rebound and maximum adhesion and coverage,
- proper training and accreditation of applicators, and
- application trials, and close and systematic quality control of an in-situ produced membrane, to ensure correct thickness and coverage, and curing of the membrane under the tunnel environmental conditions.

Quality control was required throughout the membrane application process to ensure full compliance with the Particular Specification and manufacturer's instructions. A list of quality control measures was proposed as per recommendations in the ITAtech (2013) and ASTM standards (Table 3).

Table 3: *Proposal of quality control for application of sprayed waterproofing membrane*

Parameter	Test method	Frequency	Pass criteria
Coverage / continuity	Visual inspection	Visual inspection to be carried out continuously while the membrane is applied and following application	100% coverage
Thickness	Wet film thickness by depth gauge	10 tests per 100 m^2	Min. measured thickness of each course of the membrane and the entire thickness of the finished membrane shall be ≥ 2 mm or as per manufacturer's recommendation, whichever thickness is greater.
Thickness	Application quantity	Applied thickness assessed by measuring the quantity of spray membrane used for the area over which it has been applied; 1 measurement per 100 m^2.	Average thickness over the applied area identified from the measurement of spray quantity; Thickness shall be greater than or equal to the identified average thickness.
Thickness	Cut-out inspection for 50 mm x 50 mm patches	Optional method; Patches taken randomly from all sections of the tunnel profile	Min. patch thickness shall be ≥ 2 mm or as per manufacturer's recommendation, whichever thickness is greater.
Composite	Bond tests	1 test per 200 m^2; Locations where deficient adhesion is suspected by Site Engineers or a min. of 3 tests	Min. bond strength > 0.5 MPa

5 Conclusion

The portal caverns are located in highly fractured tuff with rock mass quality *Q*-value in the range of 0.2 to 0.5. During the excavation works of the portal slope and caverns, the movement of the slope and caverns were monitored closely and found to be within the predicted observation levels. It showed a successful example of the construction of caverns with narrow rock pillar in Hong Kong, even in tuff with very low rock mass quality. This cavern design also demonstrated a successful adoption of permanent FRSC lining and sprayed waterproofing membrane for rock caverns which are much safer to apply while providing equally sufficient structural contribution. This successfully designed and built portal caverns can be considered as the good practice and design reference for all upcoming cavern projects in Hong Kong.

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References

Bowles, J. E. (2001). *Foundation Analysis and Design* (5th ed.). McGraw Hill.

Chern, J. C., Yu, C. W., & Shaiu, F. Y. (1998). Tunnelling in Squeezing Ground and Support Estimation. *Proc. Reg. Symp. Sedimentary Rock Engineering.* Taipei.

Duddeck, H., & Erdmann, J. (1985). Structural Design Models for Tunnels in Soft Soil. *Underground Space*, *9: 5-6*, pp. 246-259.

Grimstad, E., & Barton, N. (1993). Updating of the Q-System for NMT. *International Symposium on Sprayed Concrete*, 46-66. Fagernes.

Hoek, E. (1994). Strength of rock and rock masses. *ISRM News Journal*, *2*(2), 4-16.

Hoek, E. (1998). Tunnel Support in Weak Rock. *Keynote Address, Symposium of Sedimentary Rock Engineering.* Taipei, Taiwan.

Hoek, E., & Brown, E. T. (2019). The Hoek-Brown Failure Criterion and GSI - 2018 edition. *Journal of Rock Mechanics and Geotechnical Engineering, 11*(3), 445-463. Retrieved from https://doi.org/10.1016/j.jrmge.2018.08.001

Hoek, E., & Diederichs, M. S. (2006). Empirical Estimation of Rock Mass Modulus. *Internal Journal of Rock Mechanics and Mining Sciences, 43*(2), 203-215.

Hoek, E., Carranza-Torres, C., & Corkum, B. (2002). Hoek-Brown Failure Criterion - 2002 Edition. *NARMS-TAC Conference*, 267-273. Toronto.

Hoek, E., Carranza-Torres, C., Diederichs, M., & Corkum, B. (2008). The 2008 Kersten Lecture: Integration of Geotechnical and Structural Design in Tunneling. *56th Annual Geotechnical Engineering Conference.* Minneapolis: University of Minnesota.

Hoek, E., Kaiser, P. K., & Bawden, W. F. (1995). *Support of underground excavations in hard rock.* Rotterdam: A.A. Balkema.

Paul, S. L., Hendron, A. L., Cording, E. J., Sgouros, G. E., & Saga, P. K. (1983). *Design Recommendations for Concrete Tunnel Linings Volume II: Summary of Research and Proposed Recommendations.* University of Illinios at Urbana-Champaign, Department of Civil Engineering. Cambridge, Massachusetts: U.S. Department of Transportation - Research and Special Programs Administration.

Sakurai, S. (1983). Displacement Measurements Associated with the Design of Underground Openings. *International Symposium on Field Measurement in Geomechanics*, 1163-1178. Zurich.

Vlachopoulos, N., & Diederichs, M. S. (2009). Improved Longitudinal Displacement Profiles for Convergence-Confinement Analysis of Deep Tunnels. *Rock Mechanics and Rock Engineering, 42*(2), 131-146.

Waterproofing, I. A.-L. (2013). *ITAtech Design Guidance for Spray Applied Waterproofing Membranes.* ITA, ITAtech Activity Group - Lining and Waterproofing. The International Tunnelling and Underground Space Association.