DESIGN AND CONSTRUCTION OF EXCAVATION WORKS FOR KLANG VALLEY MASS RAPID TRANSIT UNDERGROUND STATION AT COCHRANE, KUALA LUMPUR, MALAYSIA

CONCEPTION ET CONSTRUCTION DES TRAVAUX D'EXCAVATION POUR LA VALLÉE DU KLANG MASS RAPID TRANSIT STATION DE MÉTRO À COCHRANE, KUALA LUMPUR, MALAISIE

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ABSTRACT - Kuala Lumpur limestone formation exhibits karstic features with irregular bedrock profiles and variable weathering condition. The Klang Valley Mass Rapid Transit project is the first Mass Rapid Transit project in Malaysia. The Cochrane station is one of the underground stations with maximum excavation depth of 32m below ground and it is located in Kuala Lumpur limestone formation. This station also serves as launching shaft for the tunnel boring machine from both ends of the station. A cost effective earth retaining system deemed suitable for the geological formation was adopted using secant pile wall supported by temporary ground anchors. High ground water table is also a concern. Rock grouting was carried out to prevent water ingress into excavation pit as well as excessive ground settlement and occurrences of sinkholes surrounding the excavation area due to groundwater drawdown. Vertical rock excavation adjacent to the secant pile wall involving rock slope strengthening works, surface protection, controlled blasting and vibration control was successfully designed and implemented. This paper presents the design of the secant pile wall together with vertical rock excavation to the final depth of the station in karstic limestone formation. The unique experience (design and construction) gained from this project will be useful reference for similar excavation works, especially in mature karstic limestone.

1. Introduction

Geotechnical design is both an art and science as it deals with uncertainties associated with variable geological ground conditions. Kuala Lumpur limestone formation is karstic limestone with variable weathering condition. If complexities of the karstic limestone bedrock are overlooked during design and construction, it will pose great uncertainties and difficulties during excavation works. Therefore, excavation works in limestone formation required major geotechnical design input particularly on safety during construction and during operation of the underground structures.

The Klang Valley Mass Rapid Transit (MRT) from Sg.Buloh to Kajang is one of the major infrastructure projects launched in 2011. It is the first MRT project in Malaysia. The project involved a total of 9.8km long tunnel from Semantan to Maluri with 7 underground stations and associated structures such as portals, ventilation shafts, escape shafts and crossovers to be constructed over the Klang Valley and Kuala Lumpur city areas. Cochrane station is one of the underground stations located in the city area with maximum excavation depth of 32m below ground. This station also serves as launching shaft for the tunnel boring machine from both ends of the station. Figure 1 shows the location of the construction site. Figure 2 shows the construction site layout plan of Cochrane underground station.

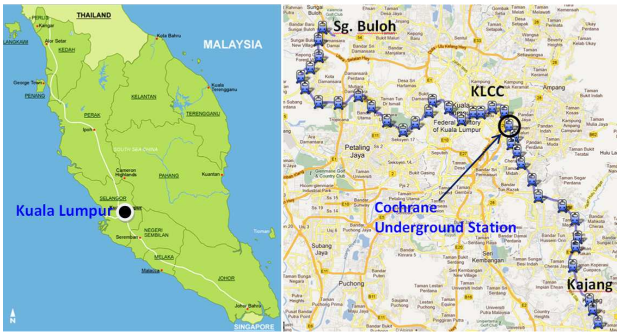


Figure 1. Location of the construction site

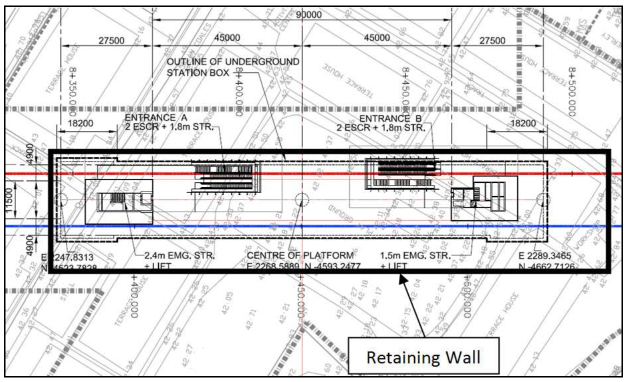


Figure 2. Construction site layout plan

2. Geological condition

Figure 3 shows the Geological Map of Selangor, (ref: sheet 94 Kuala Lumpur 1976 and 1993, published by the Mineral and Geoscience Department, Malaysia) superimposed with the tunnel alignment. The tunnel alignment starts from the Semantan Portal to Bukit Bintang Station and is underlain by Kenny Hill formation, while from Pasar Rakyat Station until the end at Maluri Portal is underlain by Kuala Lumpur Limestone. Cochrane station is located within the Kuala Lumpur Limestone formation.

Kuala Lumpur Limestone is well known for its highly erratic karstic features (Tan 2005). Due to the inherent karstic features of limestone bedrock, the depth of the limestone bedrock is highly irregular. The overburden soils above Kuala Lumpur Limestone are mainly silty sand. The thickness of overburden soils varies significantly due to the irregular topography of the limestone bedrock.

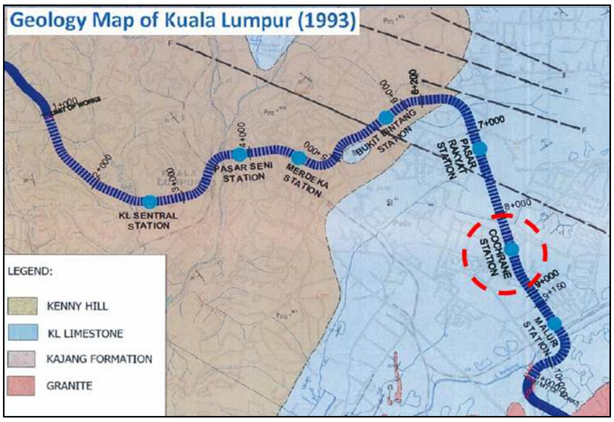


Figure 3. Geological map of Kuala Lumpur

3. Subsurface investigation

Subsurface investigation was carried out to obtain necessary subsoil information and design parameters. Thirty-one boreholes were carried out in stages at the Cochrane underground station as shown in Figure 4. Generally, the boreholes are located at the station footprint and the retaining wall alignment.

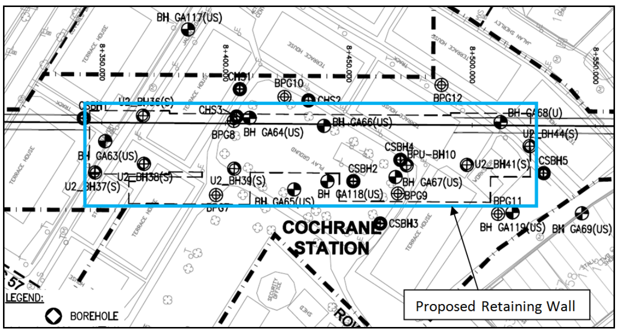


Figure 4. Subsurface investigation layout plan

The investigation depth is 10m below final excavation level or 10m continuous (cavity free) coring into limestone whichever is deeper. Selected boreholes were terminated at 1.6 times the excavation depth i.e. 20m below final excavation depth. Standard Penetration Tests (SPT) were carried out in the boreholes at 1.5m vertical intervals. Disturbed and undisturbed soil samples were collected for visual inspection and laboratory testing. Pressuremeter tests and field permeability tests were also carried out in boreholes to obtain elastic modulus and permeability respectively for the subsoil. The groundwater table was about 1m below ground. The interpreted geotechnical parameters are tabulated in Table 1.

For limestone bedrock, rock core samples were collected for rock quality assessment such as weathering condition and fracture state with rock-quality designation (RQD) values. Lugeon tests were carried out to obtain water permeability of bedrock and the hydraulic conductivity resulting from fractures. Point load tests in vertical and horizontal direction and unconfined compression strength tests (UCS) were carried out to correlate between UCS values against point load index ($I_{s(50)}$). The interpreted correlation factor is UCS = 11($I_{s(50)}$) and UCS = 18($I_{s(50)}$) for horizontal and vertical direction respectively where UCS is in MPa.

Table 1. Interpreted geotechnical parameters

	Overburden	Bedrock
Material type	Silty sand	Limestone
Average depth	0m – 5m	5m below
Unit weight	18 kN/m^3	24 kN/m^3
SPT N	2 - 4	-
RQD	-	0 – 100%
Average UCS	-	50 MPa
Effective shear strength	c'= 1 kPa ϕ'= 29º	c'= 400 kPa ϕ'= 32º
Elastic Modulus, E' (kPa)	4000 - 12000	1.0E6
Hydraulic conductivity, k	1.0E-5 m/s	0 – 31 Lugeon

4. Design for excavation works

Excavation works for Cochrane station consist of overburden soil excavation and rock excavation to required depth for TBM launching preparation and also permanent structure construction. A rectangular cofferdam measuring about 37m x 176m was constructed as shown in Figure 2 to facilitate soil excavation until bedrock level. Continuous vertical rock slope excavation to final excavation level was carried out with just 1m offset from the retaining wall alignment. Figure 5(a), shows the overall view of the excavation works. Figure 5(b) shows the TBM launching face upon reaching the final excavation depth.

(a) (b)

Figure 5. Overall view of excavation works

4.1. Temporary earth retaining system

The selection of retaining wall has considered the workability and suitability of subsoil and rock conditions. Secant pile wall was selected as the earth retaining wall supported by temporary ground anchors. The advantages of the selected wall type are (i) water-tightness to prevent groundwater draw-down at the retained side; and (ii) the ability to vary the pile lengths to suit the irregular limestone bedrock profiles. Secant piles of 880mm and 1000mm in diameter were designed with an overlap of 130mm and 200mm respectively representing 15-20% of pile diameter. The extents of overlapping of the secant piles are governed by pile installation verticality, pile deviation and pile depth (CIRIA C580, 2003). The hard/firm secant pile wall consists of primary (female) piles casted first with concrete strength class C16/20 without reinforcement and followed by secondary (male) pile with concrete strength class C32/40 with reinforcement. Figure 6 shows typical arrangement of the secant pile wall. Schematic of excavation works is shown in Figure 7.

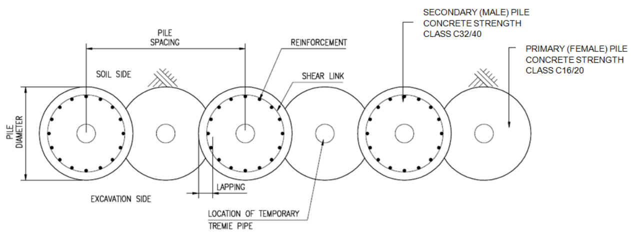
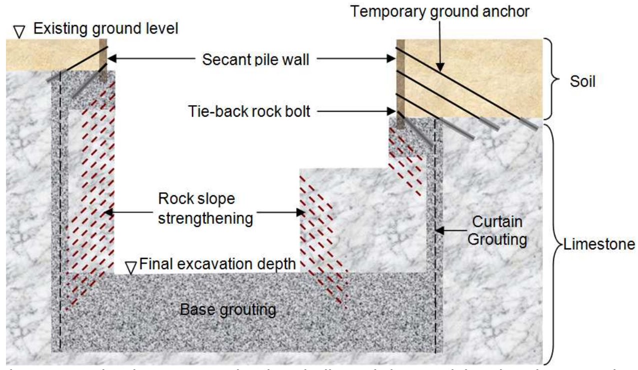


Figure 6. Typical arrangement of secant pile wall

The analysis of the retaining wall was carried out using PLAXIS, a finite element code. Wall displacement, bending moment and shear force were obtained from the analysis for structural design. A load factor of 1.4 for bending moment and shear force were applied for pile reinforcement design. The quantity of reinforcement is about 0.5% to 4% of pile cross-section area depending on the rock head level. Finite element modeling input parameters and criteria are presented in Table 2.



(Note: Rock slope strengthening indicated is provisional only. Actual locations and length of rock slope strengthening are determined after geological mapping works and kinematic analysis).

Figure 7. Schematic of excavation works

Table 2. Modeling input parameters and criteria

Description	Modeling input
Model type	Plain strain analysis
Soil model	Hardening Soil Model
Soil shear strength	Effective stress parameters
Soil material type	Drained
Soil loading stiffness	2000 x SPT'N (1)
Soil unloading stiffness	6000 x SPT'N (2)
Soil/wall Interface factor	0.8
Wall element	Plate element
Wall bending stiffness	0.7 x EI (3)
Wall compression stiffness	0.7 x EA (4)
Anchor pre-stress load	60 – 80% of anchor working load
Construction surcharge	20 kPa
Groundwater condition	Phreatic line

Notes: (1) (2) Tan & Chow (2008); (3) (4) CIRIA 2003

All secant piles were founded on bedrock with minimum rock socket of 1.5-3.0m. The termination criteria of rock socket are based on coring in competent bedrock with point load index strength, $I_{s(50)}$ > 4 MPa (equivalent to UCS of 44 MPa). It is important to ensure that the retaining wall is socketed into competent bedrock as the vertical rock excavation is just 1m away from the retaining wall alignment. A row of tie-back rock bolts were installed above the bedrock level to enhance toe stability. Toe stability check was carried out in accordance with BS8002:1994 with some modification which replaces passive resistance by tie-back force to achieve minimum safety factor of 1.2. In addition, vertical stability was checked with resultant vertical load from ground anchor pre-stress against the rock socket length.

Excavation was carried out in stages facilitated by installing temporary ground anchors. Design and testing of ground anchor is in accordance with BS8081:1989. U-turn ground anchor was used for removable requirement after construction. The anchor consists of a few pairs of strand with different unit lengths. Proofing tests were carried out prior to the working anchor installation for design verification. Based on the proofing test result, the recommended reduction factor due to bending of strand at U-turn point is 0.65. Summary of the anchor properties are shown in Table 3.

Table 3. Ground anchor properties

Description	Properties
Working loads (kN)	212; 424; 636; 848
No. of strand	2; 4; 6; 8
Strand diameter	15.24mm
Breaking load	260.7 kN
Factor of safety	1.6
Strand U-turn radius	47.5mm
Reduction factor	0.65
Drill hole diameter	175mm
Allowable bond stress	400 kPa (limestone)
Free length	Varies (until bedrock)
Bond length (m)	3; 3; 4.5; 6

4.2. Rock slope strengthening works

The rock excavation was carried out using conventional pre-split blasting followed by bulk blasting with suitable delays to minimize the impacts of blasting works. The blasting works were carried out in 2 to 3m benches. After blasting, geological mapping was carried out by qualified geologist to collect field data on the exposed rock face including details of discontinuities, rock face weathering condition, etc. The field data is used for kinematic stereonet analysis using software (Rock Pack III) to determine the probable mode of rock slope failures. The probable failure mode was further analysed using software RocPlane (planar stability), Swedge (wedge stability) and RockPack II (toppling stability) to establish the factor of safety and determine suitable rock slope strengthening works.

4.3. Grouting works

Grouting techniques rely much on local experiences. Grouting works is mainly carried out for limestone to reduce the rate of groundwater inflow into excavation and reduce pathways of water flow into excavation area. Rock fissure grouting was carried out along the perimeter of excavation area to form a curtain grouting as shown in Figure 7. Fissure grouting involves a single packer in ascending or descending stages in order to inject grout suspension into existing pathways, fissures, cavities and discontinuities within the rock formation. Rock fissure grouting is also adopted for base grouting at larger grout hole spacing. If any cavities are detected during drilling / grouting, compaction grouting with cement mortar will be used as cavity treatment.

5. Achievement

Excavation works started in early 2012 and reached the final excavation level in January 2013. Proper geotechnical input and continuous support from the design engineers during construction have enabled vertical rock excavation to be carried out safely and without delay. This design scheme has resulted in considerable time and cost saving compared to non-vertical excavation which will incur additional cost and also present challenges in terms of additional land acquisition.

With proper geotechnical input, costly failure and delay associated with underground works in limestone formation such as excessive groundwater lowering, occurrences of sinkholes, excessive ground settlement, etc. can be prevented. It is important to have continuous feedback from the construction team to anticipate problems and such model of cooperation between the construction team and the geotechnical engineers has proven to be successful as the excavation works at Cochrane station were completed successfully and within the contract period.

6. Conclusions

Secant pile wall supported by temporary ground anchors and rock strengthening were successfully used for the underground station excavation works. The secant pile wall system together with grouting works prevented excessive groundwater lowering and excessive ground movement. Overall, the system performs satisfactorily and the excavation works were successfully completed within the contract period.

7. Acknowledgement

The Author would like to thank G&P Geotechnics design team members and project team of MMC-Gamuda KVMRT for various discussions on overcoming challenges associated with limestone formation. The support and sponsorship from the Institution of Engineers, Malaysia (IEM) to attend the conference is also gratefully acknowledged.

8. References

BS8002:1994. *British Standard Code of Practice for Earth Retaining Wall.*

BS8081:1989. *British Standard Code of Practice for Ground Anchorages.*

CIRIA C580. (2003). *Enbedded retaining walls – guidance for economic design*, London.

Raju V. R. & Yee Y W (2006). Grouting in limestone for SMART tunnel project in Kuala Lumpur. *International Conference and Exhibition on Tunnelling and Trenchless Technology,* 7-9 March 2006, Subang, Selangor, Malaysia.

Tan S. M. (2005). Karstic Features of Kuala Lumpur Limestone*. Bulletin of the Institution of Enginner Malaysia, June 2005, 6 -11.*

Tan Y. C. & Chow C.M. (2008). Design of retaining wall and support systems for deep basement construction – a Malaysian experience. *Seminar on "Deep Excavation and Retaining Walls",* , 24 March 2008, Petaling Jaya, Malaysia.