Underground stations excavation of up to 45m deep for mass rapid transit in limestone formation, Malaysia

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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 (KVMRT) SBK Line project is the first Mass Rapid Transit project in Malaysia. There are three underground stations namely Tun Razak Exchange (TRX) station, Cochrane Station and Maluri Station located in Kuala Lumpur limestone formation. TRX Station is the deepest station with maximum excavation depth of 45m below ground and also one of the underground interchange stations for future line. Cochrane Station with maximum excavation depth of 32m below ground also serves as launching shaft for the tunnel boring machine from both ends of the station. Maluri Station with maximum excavation depth of 20m below ground includes an underground train crossover as operational requirement. A technically appropriate and cost effective temporary earth retaining system suitable for the challenging geological formation using secant pile wall supported by temporary ground anchors or temporary strutting was adopted. High ground water table is also a challenge for the deep excavation. Rock grouting was carried out to prevent water ingress through rock fissures and cavities into the excavation pit as well as to prevent excessive ground settlement and occurrences of sinkholes surrounding the excavation area due to groundwater drawdown. Vertical rock excavation adjacent to the retaining wall involving rock slope strengthening works, surface protection, controlled blasting and vibration control was successfully designed and implemented. Temporary traffic decking on top of underground station was designed in order to maintain the traffic flow during the station excavation works. This paper presents the design of the temporary earth retaining system 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 formation.

Keywords: KVMRT, SBK Line, Underground station, Deep excavation, Temporary Earth Retaining System, Kuala Lumpur limestone.

1 INTRODUCTION

Geotechnical design is both an art and science as it deals with uncertainties associated with variable 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 requires major geotechnical design input particularly on safety during construction and during operation of the underground structures.

The Klang Valley Mass Rapid Transit (KVMRT) from Sg.Buloh to Kajang (SBK Line) is one of the major infrastructure projects launched in 2011 by the Government of Malaysia and managed by MRT Corporation Sdn Bhd. It is the first MRT project in

Malaysia. The project comprises of a total of 9.8km long twin tunnels 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. Tun Razak Exchange (TRX) Station, Cochrane Station and Maluri Station are underground stations located in the city area with excavation depth up to 45m deep in limestone formation. TRX Station is the deepest station with maximum excavation depth of 45m below ground and also one of the underground interchange station for future line. Cochrane Station also serves as launching shaft for the tunnel boring machine from both ends of the station. Maluri Station will be combined with an underground train crossover and fully covered temporary road decking on top during excavation works. Figure 1 shows the location of the

construction site.

Fig. 1. Location of the construction site.

2 GEOLOGY

Figure 2 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 TRX Station until the end at Maluri Portal is underlain by Kuala Lumpur Limestone. Kuala Lumpur Limestone is well known for its highly erratic karstic features. 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.

Fig. 2. Geological map of Kuala Lumpur.

3 SUBSOIL CONDITIONS

Subsurface investigation was carried out to obtain necessary subsoil information and design parameters. 63 boreholes were carried out at TRX station, 31 boreholes were carried out at Cochrane station and 40 boreholes were carried out at Maluri station. Generally, the boreholes are surrounding the station footprint and the retaining wall alignment.

The investigation depth criteria is 10m below final

excavation level and 10m continuous (cavity free) coring into solid limestone. Standard Penetration Tests (SPT) were carried out in the boreholes for the overburden subsoil 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 is generally about 1m below ground.

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 the 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 ($Is_{(50)}$). The interpreted average correlation factor is $UCS = 11(Is_{(50)})$ and $UCS = 18(Is_{(50)})$ for horizontal and vertical direction respectively where UCS is in MPa.

The overburden subsoil above limestone generally comprises of loose silty sand to sand materials with SPTN value less than 4. Average unit weight and permeability of subsoil are 18 kN/m^3 and $1 \times 10^{-5} \text{ m/s}$ respectively. Interpreted effective shear strength is $c' = 1 \text{ kPa}$ and $\phi' = 29^\circ$. Bedrock profiles of limestone formation are highly variable which range from 3m to 30m below ground. Cavities, pinnacles and valleys are detected during subsurface investigation works. Figure 3 presents some typical features of limestone formation.

Fig. 3. Typical features of limestone formation (ref???)

4 DESIGN FOR EXCAVATION WORKS

4.1 Advance works

Advance works were planned and implemented before commencement of excavation works. Affected existing utilities were piloted and protected or relocated if necessary. Groundwater recharge wells were installed as precautionary measures to prevent excessive groundwater drawdown. Dilapidation survey and baseline monitoring results for existing buildings or properties were carried out and recorded for further assessment when needed. For the critical existing

buildings and structures, damage assessment due to excavation works were carried out to identify the potential damage classification and risk assessment to the project. Suitable strengthening and underpinning works were designed and completed before excavation works.

4.2 Temporary earth retaining system

The selection of retaining wall system 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; (ii) the ability to vary the pile lengths to suit the irregular limestone bedrock profiles; and (iii) installed primary pile serves as reference for reinforcement determination based on more accurate bedrock profiles. 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 4 shows typical arrangement of the secant pile wall.

Fig. 4. Typical arrangement of secant pile wall.

The secant piles sizes used for this project are 880mm, 1000mm, 1180mm, and 1500mm. The secant pile were generally designed with an overlap of 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). After reviewing the piles as-built performance, the recommended overlapping values of secant pile wall are shown in Table 1 where overlapping of up to 34% were specified to ensure water-tightness of the wall

Table 1. Overlapping of secant pile wall.

Pile diameter	Length<8m	Length<15m	Length<25m
880mm	130mm	170mm	-
1000mm	150mm	200mm	340mm
1180mm	170mm	230mm	360mm
1500mm	225mm	260mm	380mm

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 ranges from 0.5% to 4% of pile cross-

sectional area depending on the analysis based on different rock head level. 20kPa construction surcharge and 0.5m unplanned excavation were considered in ultimate limit state design. Service limit state analysis were carried out to ensure the ground deformation caused by excavation will not exceed acceptable threshold limits of existing buildings and structures.

All secant piles were founded on competent bedrock with minimum rock socket of 1.5m to 4.0m. The termination criteria for rock socket are based on coring in competent bedrock with verification of point load index strength, $I_s(50) > 4$ MPa (equivalent to average 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 1.25m away from the retaining wall alignment. Support system will be installed in stages until reaching the bedrock level. A row of tie-back rock bolts were installed above the bedrock level to enhance wall 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 anchors were used due to removable requirement after construction. The anchor consists of a few pairs of strand with different unit lengths. Adopted strand diameter is 15.24mm with U-turn radius of 47.5mm. Proofing tests were carried out prior to the working anchor installation for design verification. Based on the proofing test results, the recommended reduction factor due to bending of strand at U-turn point is 0.65. Working loads of anchor range from 212kN to 1060kN with 2 to 10 nos. of strands. Typical designed pre-stress load is 60-80% of working load capacity. Generally the anchor will be locked off at 110% of designed pre-stress load. All anchors are subjected to acceptance test up to 125% of working load before lock-off. It is important to clearly define in construction drawing the anchor working load, pre-stress load and lock-off load to prevent misunderstanding and confusion during construction works

The design of temporary steel strutting elements for this project are in accordance with limit state design to BS 5950 and recommendations of CIRIA Special Publication 95. Design criteria considered in strutting design are earth pressure and groundwater, material dead load, 1.5 kN/m live load, eccentric load, temperature effect (changes of 10°C), accidental impact load (50kN in vertical direction; 10kN in horizontal direction), and one-strut failure. Recommended partial load factors for strutting design are shown in Table 2.

Table 2. Partial load factors.

Load case	EL	DL	LL	TL	IL
Working condition	1.4	1.4	1.6	1.2	NA
Accidental impact	1.05	1.05	0.5	NA	1.05
One-strut failure	1.05	1.05	0.5	NA	NA

Note:

EL – Earth pressure and groundwater

DL – Dead load

LL – Live load

TL – Temperature effect

IL – Accidental impact load

NA – Not applicable

Obstruction free retaining wall system was also designed for TBM launching and retrieval. 21m diameter circular shaft formed by 1m diameter secant pile wall was designed for two launching shafts at Jalan Inai near TRX station. Hoop stress concept is applied to determine required effective contact area of secant piles wall and ring beam support. At Maluri retrieval shaft, ground improvement method using deep soil mixing (DSM) was constructed as alternative retaining wall system to secant pile wall. DSM was designed as gravity block and founded on average rock head of 8m. The extent of the DSM block away from excavation face is about 0.9 times of retained height with dowel reinforcement into rock to enhance the sliding stability due to high groundwater table.

4.3 Groundwater control

Groundwater control is one of the important criteria to be considered in excavation works. Groundwater drawdown may lead to excessive ground settlement and occurrences of sinkholes surrounding the excavation. Potential risk of excessive groundwater ingress into excavation pit shall be evaluated especially in limestone formation. Natural features of solution channel with cavities and highly fractured limestone connected to excavation pit may cause a disastrous flooding inside the excavation pit. Therefore, grouting in limestone was carried out as risk mitigation measure for groundwater control. Schematic of the excavation works is shown in Figure 5.

Grouting techniques rely much on local experiences and contractor workmanship. 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 up to 10m below final excavation level. 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. Additional grouting may be required after reviewing the grout intake from primary grouting. Rock fissure grouting is also adopted for base grouting at larger grout hole spacing. If any cavities are detected during drilling or grouting works, compaction grouting

with cement mortar will be used as cavity treatment. Recommended holding pressures for fissure grouting in limestone are shown in Table 3.

Table 3. Holding pressure for fissure grouting.

Depth (m)	Holding pressure (Bar)
0 to 10	2 to 4
10 to 20	6 to 8
20 to 30	10 to 12
30 to 40	14 to 16
40 to 50	18 to 20
>50	>22

Note: Termination criteria shall be satisfied with flow rate less than 2 liters per minute or grout volume reaches 10m³ for every grouting zone in 5m depth.

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

Fig. 5. Schematic of excavation works.

4.4 Excavation in limestone

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 commercial software (Rock Pack III) to determine the probable mode of rock slope failures. The probable failure mode was further analysed using another commercial 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.

4.5 Temporary road decking

The design of temporary steel road decking structures is in accordance with BS5950 and the traffic loads are based on BS5400 and BD37/01 as per Malaysia's Public Works Department (JKR) guideline. Load cases considered are HA, HA+30HB and 45HB traffic loadings. In summary, the road decking is designed for a general area load of 25kPa. 2m x 1m concrete deck panels were adopted for public road to

provide equivalent road surface requirements.

4.6 Instrumentation and monitoring

The instrumentation and monitoring works are important to serve as an early detection scheme for potential problems which may arise during the construction works. The instrumentation is not only applicable for designed elements within construction site but area outside the site boundary also needs to be monitored for existing buildings and structures and environmental requirements. Typical instruments for designed element are inclinometer for wall movement, ground settlement marker for ground movement, load cell for support force monitoring, strain gauge for steel strain measurement, standpipe for groundwater monitoring, piezometer for pore pressure measurement, vibrometer for vibration monitoring, etc. Some instruments for existing buildings and structures are ground displacement marker for horizontal and vertical ground movement, building tilt meter and settlement marker, standpipe, etc. In order to ensure the construction works complied with environmental requirements, generated vibration and noise were monitored in accordance with guidelines by the Department of Environment (DOE).

Monitoring triggering scheme was implemented at different notification levels (Alert, Action and Alarm). The contractor is responsible to coordinate, inform and implement necessary action when the monitoring results achieve every triggering level. Alert level is to allow the contractor or designer to revisit their design or method of construction when monitoring results showed that the actual performance is close to the design assumptions and contingency plan shall be prepared. When the monitoring results reached Action level, action plan shall be implemented immediately and monitoring frequency increased for close monitoring. Alarm level is to give an early warning notification when the designed element is close to ultimate limit state or failure condition. At this stage, necessary remedial works and risk mitigation shall be carried out to ensure the safety of construction works.

5 CONSTRUCTION

Excavation work started in year 2012 at Cochrane station for 4 nos. of TBM launching towards north and south directions. Bedrock profile at this station is generally at shallow depth of about 5m, with localised deep rock head found at northern side of the station. Secant pile wall are mainly supported by temporary ground anchors to provide obstruction free area when lifting down TBM structure to the required platform. Micropiles foundation was designed for heavy lifting crane with capacity up to 600 tons. When the excavation reached final level of 32m below ground, base slab were casted to provide a platform for TBM launching preparation. Figure 6 shows the second TBM

launching condition at Cochrane station.

Excavation works at TRX station started when Cochrane station excavation works are still in progress. This is the biggest and deepest station and is planned as the interchange station for future line of the project. Excavation depth is 45m below ground and station footprint is about 170m long and 35m wide. Bedrock profile at this station is generally at shallow depth of about 10m with deep rock head of up to 24m found at the center and northern part of the station. Secant pile wall are mainly supported by temporary ground anchors to provide obstruction free area when lifting up TBM structure after retrieval from Cochrane station. Temporary strutting was adopted at north ventilation building excavation due to limit of construction boundary. Another TBM were launched at independent launching shaft at Jalan Inai towards Bukit Bintang direction while a portion between TRX station and launching shaft will be mined tunnel of about 25m long. Figure 7 shows the excavation works at TRX station.

Fig. 6. Cochrane Station (Launching of second TBM).

Fig. 7. TRX Station (excavation in progress).

Maluri station and crossover are located underneath one of the major public road in town (Jalan Cheras). Excavation works for this station started late compared

to TRX and Cochrane stations due to major utilities diversion (e.g. 132kV cables) and traffic diversion in four stages for installation of secant pile wall. Deckposts (UC section) for temporary road decking were installed concurrently with secant pile installation. About 300m long and 21m wide road decking covered up the top of the station and crossover area during excavation works beneath. The excavation works were carried out under the road decking until final level of 20m below ground. One of the construction difficulties is pile installation under existing electrical transmission lines with safe allowable working head room of 13m. A modified low head machine was used for secant pile installation. In this condition, limit of drilling size to small diameter is required to fulfill the capacity of the modified machine. Deckpost installation required high capacity with deep rock drilling which is beyond the machine capacity and as such, deckpost formed by 4 nos. micropiles in a group was proposed as the alternative method underneath the existing electrical transmission line.

Fig. 8. Maluri Station (base slab casting with life traffic on top).

Fig. 8. Maluri Portal (excavation in progress).

As-built performance showed that major deviation of micropile installation in rock occurred and additional strengthening was done during excavation to enhance deckpost capacity. Figure 8 shows the base slab casting at Maluri station and Figure 9 shows the excavation

works with strutting support at Maluri Portal.

6 CONCLUSIONS

Proper geotechnical input and continuous support from the design engineers during construction have enabled the excavation works in challenging ground conditions supported by secant pile retaining wall with vertical rock excavation to be carried out safely. 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 from experienced engineers, 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 progressed.

Suitable temporary earth retaining system 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.

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