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CHALLENGES IN DESIGN AND CONSTRUCTION OF DEEP EXCAVATION FOR KVMRT IN KUALA LUMPUR LIMESTONE FORMATION

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Graphical abstract

TRX Station (excavation in progress).

Maluri Portal (excavation in progress)

Abstract

The Klang Valley Mass Rapid Transit (KVMRT) Sungai Buloh - Kajang Line project is the first Mass Rapid Transit (MRT) project in Malaysia. The KVMRT Project when completed will cover a distance of 51km and comprises of 31 passenger stations. This paper covers the challenges in design and construction of deep excavation works for three underground stations, namely Tun Razak Exchange (TRX) station, Cochrane Station and Maluri Station, as well as one portal (South Portal) all located in Kuala Lumpur limestone formation. The Kuala Lumpur Limestone formation exhibits notorious karstic features with irregular bedrock profiles, variable weathering condition, cavities and slime zones. This paper presents the design principles of 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 a useful reference for similar excavation works, especially in karstic limestone formation.

Keywords: Mass Rapid Transit (MRT); deep excavation; limestone

Abstrak

Projek Klang Valley Mass Rapid Transit (KVMRT) Sungai Buloh – Kajang merupakan projek pengangkutan rel bandar (MRT) pertama di Malaysia. Apabila keseluruhan projek siap, jajaran MRT ini akan merangkumi jarak sepanjang 51km dan 31 stesen penumpang. Kertas ini akan membentangkan cabaran merekabentuk dan pembinaan kerja-kerja pengorekan dalam untuk tiga stesen bawah tanah, iaitu Stesen Tun Razak Exchange (TRX), Stesen Cochrane dan Stesen Maluri, dan juga sebuah portal (South Portal) yang terletak di kawasan dengan Kuala Lumpur limestone (batu kapur). Kuala Lumpur limestone mempunyai ciri-ciri karstic dengan profil batu yang tidak seragam, keadaan luluhawa yang berubah-ubah, rongga dan juga zon lendir. Kertas ini juga membentangkan prinsip-prinsip rekabentuk untuk tembok penahan sementara dengan pengorekan batu menegak sehingga paras akhir stesen di kawasan karstic limestone. Pengalaman unik (rekabentuk dan pembinaan) yang diperolehi dari projek ini amatlah berharga sebagai rujukan masa hadapan untuk kerja-kerja serupa yang melibatkan pengorekan dalam, terutamanya di kawasan karstic limestone.

Kata kunci: Mass Rapid Transit (MRT); pengorekan dalam, limestone (batu kapur)

1.0 INTRODUCTION

Due to scarcity of land, especially in urban areas, the need for basements to optimize the use of land has resulted in increasing depth of basements being constructed. In this paper, the approximate division between shallow and deep excavation is based on 6m which is guided by the definition used by CIRIA [1] on trenching practice and Puller, 1996 [2]. The design of retaining walls and support systems for deep basement construction requires careful analysis, design and monitoring of performance. This is because the risk associated with the works is high and high profile failures involving deep excavation (e.g. Nicoll Highway, Singapore and Shanghai Metro, China) have highlighted the need for proper design and construction control. A recent study by Moh & Hwang, 2007 [3] has listed 43 failures since 2001 related to MRT works of which 8 failures were related to retaining walls and strutting works and some of the failures have resulted in death, collapsed buildings and economic losses in millions. Some of the recommendations by Moh & Hwang, 2007 [3] include having a proper risk management program associated with underground works and a sound understanding of geotechnical fundamentals to complement the use of computer codes. Proper implementation of risk management programmes and the use of computer codes require sound understanding of the design and construction considerations of underground works in order for the risk management to be effective and computer codes used properly. As such, this paper intends to highlight some of the important aspects of Malaysian experience on design of retaining walls and support systems for deep basement construction to ensure a safe and economical design.

A recent successful case history involving deep excavation works for three (3) underground stations and one (1) portal in Kuala Lumpur limestone for Malaysia's first MRT project, i.e. Sungai Buloh-Kajang Line will also be presented.

2.0 DESIGN CONSIDERATIONS

In this paper, a brief discussion on the planning of subsurface investigation and testing and selection of retaining walls and support systems will be presented followed by a more detailed discussion of the design of retaining walls and support systems for deep basement excavation. The design of retaining walls and support systems for deep basement excavation will cover the following aspects:

a) Overall stability
b) Basal heave failure
c) Hydraulic failure
d) Axial stability
e) Finite element analysis
f) Ground movement associated with excavation

The details are presented by Tan & Chow, 2008 (4). This paper update some of recent development for the design and construction of deep excavation in Malaysia. The analysis and design flowchart for deep excavation works are summarised in Figure 1

3.0 SMALL STRAIN STIFFNESS OF SOILS AND ITS NUMERICAL CONSEQUENCES

It is well understood that soil stiffness decays non-linearly with strain (Figure 2). The maximum strain at which soils exhibit almost fully recoverable behaviour is found to be very small. The very small-strain stiffness associated with this strain range, i.e. shear strains $\gamma_s \leq 1 \times 10^{-6}$, is believed to be a fundamental property of all types of geotechnical materials including clays, silts, sands, gravels, and rocks [6] under static and dynamic loading [7] and for drained and undrained loading conditions [8].

For practical purposes, small-strain stiffness is probably most reliably obtained using geophysical techniques which measure shear wave velocity (Figure 3). Out of the various field and laboratory methods, cross-hole surveying is probably the most reliable method, but also the most expensive. A cheaper alternative would be downhole seismic survey or seismic piezocone /dilatometer and as such, it is recommended to use a combination of the two methods for in-situ measurement of shear wave velocity.

The input parameters for the small-strain stiffness model in PLAXIS are as follows:

a) G_0 – maximum small strain-strain shear modulus
b) $\gamma_{0.7}$ – denotes the shear strain, at which the shear modulus G is decayed to 70 percent of its initial value G_0

The above two parameters would be able to define the entire stiffness degradation curve, for example using the Hardin-Drnevich relationship [18]. The values of G_0 can be obtained from measurement of shear wave velocity from the following relationship:

$$G_0 = \rho v_s^2$$

where, ρ is mass density of soil and v_s is shear wave velocity of soil.

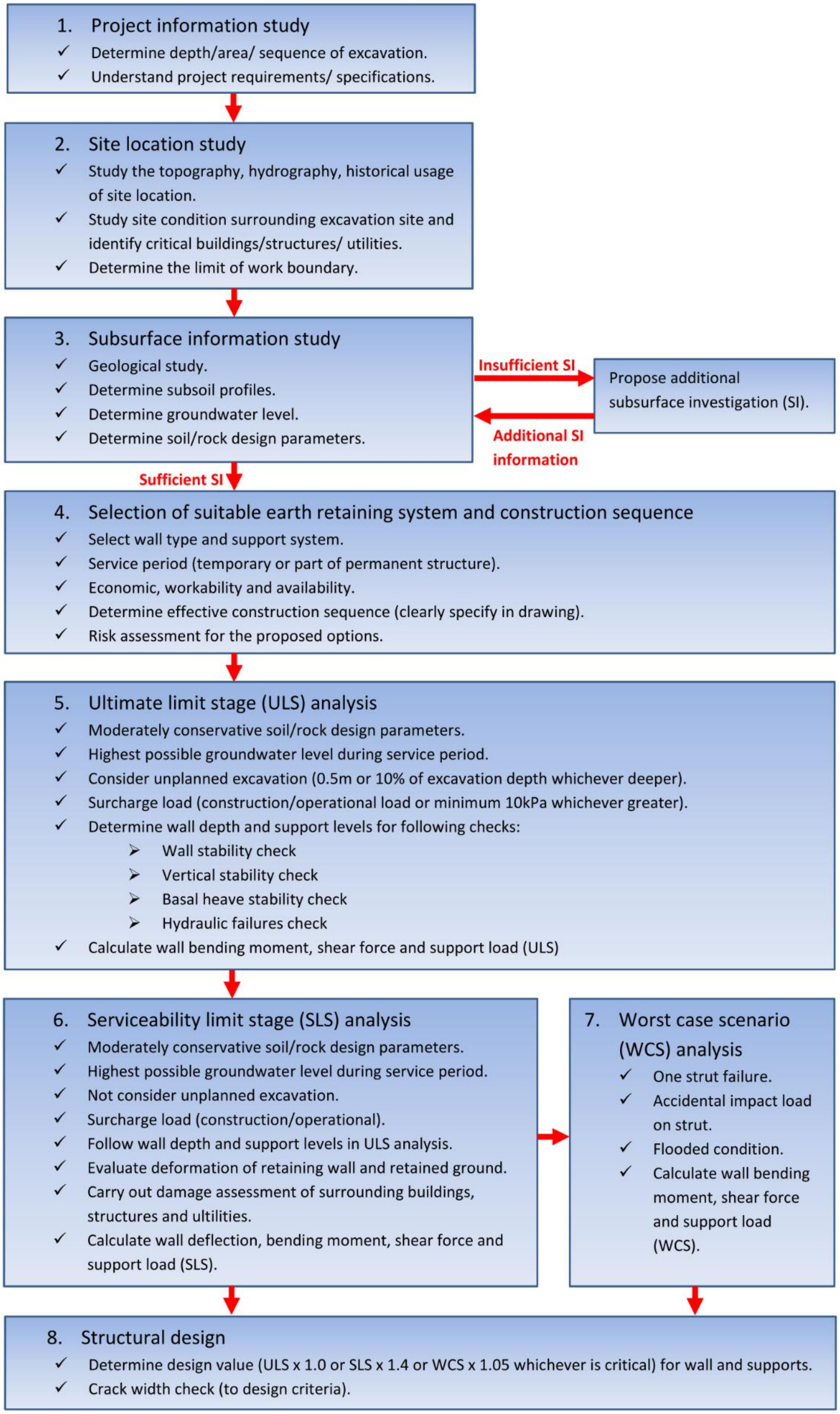


Figure 1 Flowchart for analysis and design of deep excavation works

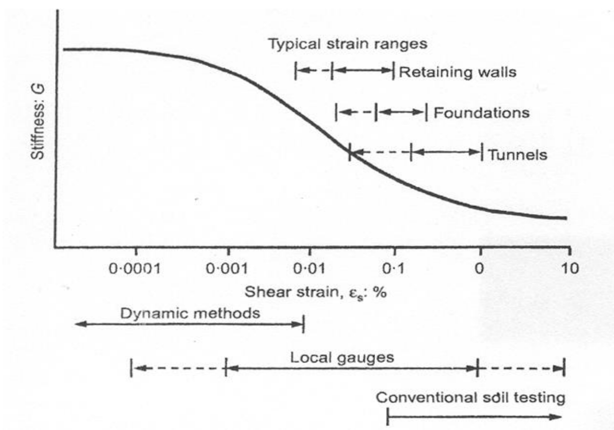


Figure 2 Characteristic stiffness-strain behaviour of soil with typical strain ranges for laboratory tests and structures [5].

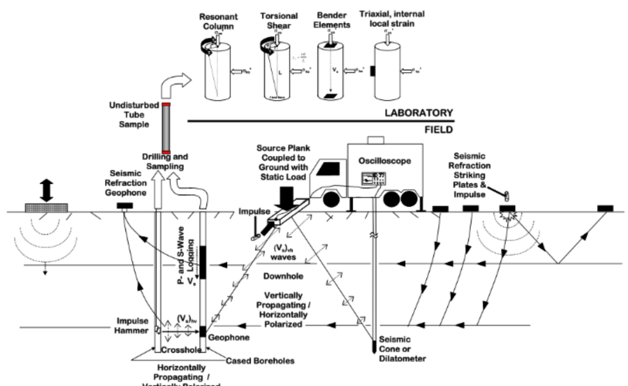


Figure 3 Field and laboratory methods to evaluate shear wave velocity [9].

Table 1 Typical values of maximum small-strain shear modulus [9].

Soil Type	Maximum small-strain shear modulus, G_0 (kPa)
Soft clays	2,750 to 13,750
Firm clays	6,900 to 34,500
Silty sands	27,600 to 138,000
Dense sands and gravels	69,000 to 345,000

In addition to using shear wave measurement, the maximum small strain-stiffness can also be estimated using empirical correlations. Table 1 presents the typical range for G_0 for several generic soil types. The maximum small-strain shear modulus can be correlated to the SPT N_{60} value and to the CPT q_c value as follows [9]:

$$G_0 = 15{,}560\,(N_{60})^{0.68}$$
$$G_0 = 1{,}634(q_c)^{0.25}(\sigma'_{vo})^{0.375}$$

Please note that the units for the above equations are in kPa.

The shear strain at which the shear modulus G is decayed to $0.7G_0$ can be calculated form the following equation [10]:

$$\gamma_{0.7} = \frac{0.385}{4G_0}\left(2c(1+\cos 2\emptyset) + \sigma'(1+Ko)\sin 2\emptyset\right)$$

The values obtained above should also be checked against values given by Stokoe et al., 2004 [11] who proposed a linear increase of $\gamma_{0.7}$ from $\gamma_{0.7} \approx 1 \times 10^{-4}$ for PI = 0 up to $\gamma_{0.7} \approx 6 \times 10^{-4}$ for PI = 100.

To demonstrate the effect of small-strain stiffness in deep excavation works, a simple comparison is made on the analysis results of a deep excavation works modelled using PLAXIS, where one is analysed using conventional hardening soil model while another model adopted HS-Small model which incorporates the small-strain stiffness. The typical PLAXIS model of the deep excavation works with retained height of 16m using semi top-down method is shown in shown in Figure 4 while comparison of the wall deflection and bending moment of the two different models are summarised in Figure 5 and Figure 6 respectively.

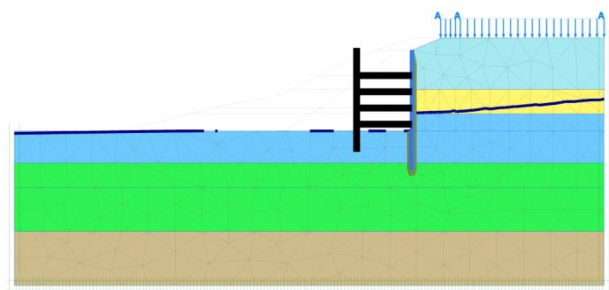


Figure 4 PLAXIS model of deep excavation using semi top-down method.

From Figure 5, it can be observed that the deflection predicted using HS-Small model is smaller compared to conventional Hardening Soil model with maximum deflection of 37mm compared to 44mm predicted using Hardening Soil model. This represents a 16% reduction in predicted maximum deflection. Overall, the deflection predicted using HS-Small model is about 27% smaller (on average) compared to Hardening Soil model.

Figure 6 shows bending moment induced on the retaining wall at the final stage of excavation where the maximum bending moment predicted using HS-Small model is smaller with magnitude of 517kNm/m compared to a value of 612kNm/m using Hardening Soil model. This represents a reduction of approximately 16% in predicted bending moment. Overall, the bending moment induced on the retaining wall is about 30% smaller (on average) compared to Hardening Soil model.

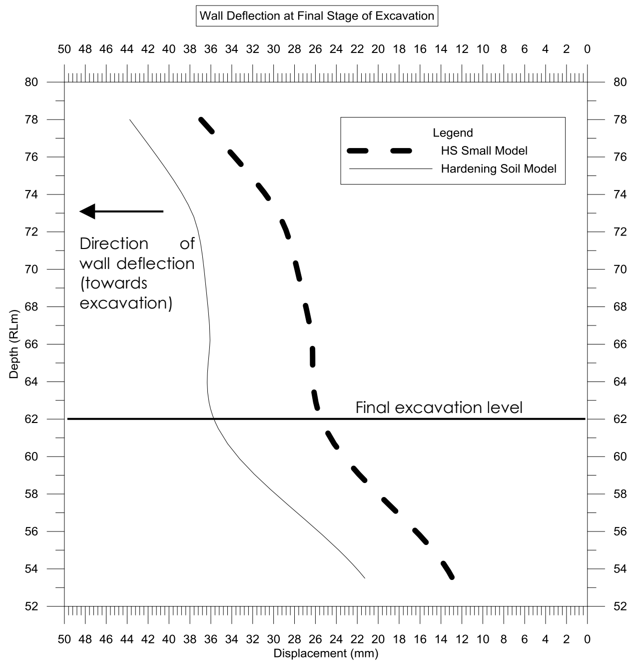


Figure 5 Comparison of wall deflection at final stage of excavation between Hardening Soil Model and HS-Small Model.

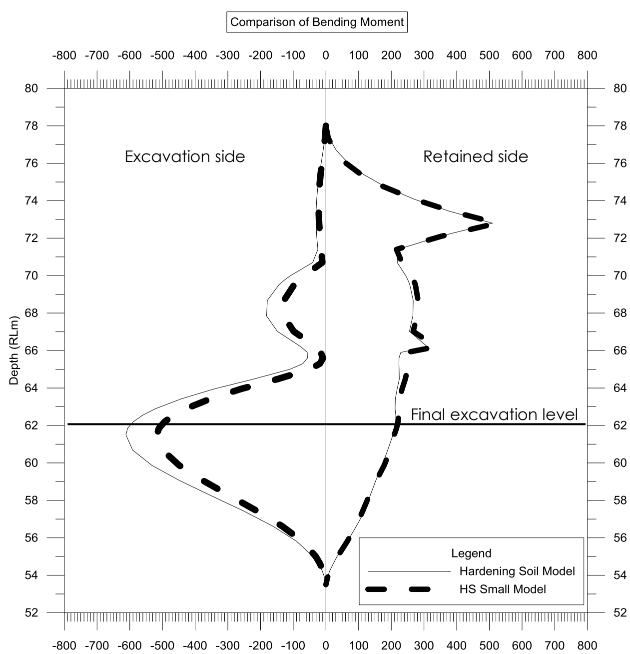


Figure 6 Comparison of bending moment at final stage of excavation between Hardening Soil Model and HS-Small Model.

In summary, it can be seen that HS-Small model which incorporates small strain stiffness offers potential savings in the design of deep excavation works and represents a step forward in the understanding of soil-structure interaction.

4.0 CIRCULAR SHAFT

In addition to conventional retaining wall and support system used in Malaysia as discussed in detailed by Tan & Chow, 2008 (4), the use of circular shaft is an attractive option as it provides an unobstructed excavation area/working space which results in faster overall construction for the deep excavation works. This system is very efficient especially for works involving large basement or circular shafts such as ventilation shaft, escape shaft, Tunnel Boring Machine (TBM) launching/retrieval shaft. For such an application, the circular lining for deep excavation works is formed using suitable embedded retaining wall.

The design of circular shaft is based on the hoop stress concept. Earth pressures surrounding the circular shaft will induce compression hoop stress on the circular lining. As the earth pressures increase with depth, the induced hoop stress will also increase and the hoop stress shall not exceed the allowable compressive stress of the concrete as per equation below:

$$\frac{Critical\ hoop\ force\ in\ wall}{Effective\ thickness\ of\ wall} \geq Allowable\ compressive\ stress\ of\ concrete$$

where

Critical hoop force (kN per meter)
= (Maximum lateral pressure) x (0.5 of circular shaft outer diameter)

Effective thickness (m)
= (structurally connected area of retaining wall) – (pile deviation and verticality at critical depth during installation)

Allowable compressive stress of concrete (kPa)
= 0.25 of concrete design strength

Even though theoretically, ring beam/circular waling is not required as all the induced stress is in hoop compression, ring beam/circular waling is recommended especially for critical excavation works/large diameter shafts due to risk of poor connection between the retaining wall elements. For design of circular diaphragm wall, reference can be made to [2] while for circular sheet piled cofferdam, reference can be made to CIRIA SP95 [12].

A recent successful application of the circular shaft designed by the Authors using secant pile wall is for the KVMRT (Klang Valley Mass Rapid Transit) project in Jalan Inai, Kuala Lumpur. Two circular shafts with 19m inner diameter was formed using hard/firm secant pile to create an unobstructed opening to lower down the TBM for launching. The diameter of the secant pile is 1.18m with 240mm overlapping into the primary pile.

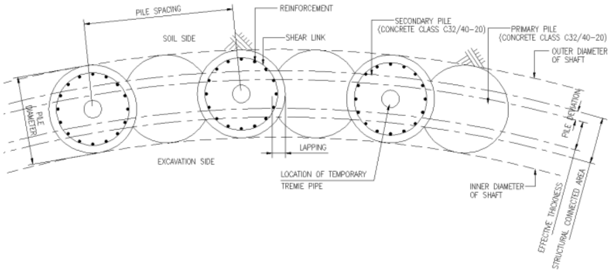


Figure 7 Secant pile arrangement to form circular shaft.

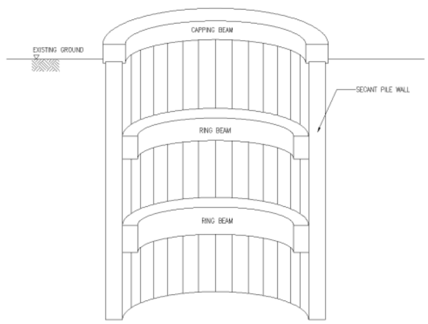


Figure 8 Sectional view of circular shaft with ring beams.

Figure 9 Circular shaft during excavation.

Figure 10 Overall view of the two completed TBM launching shafts (picture sourced from internet).

Figure 7 shows the secant pile wall arrangement with the overlapping to form a watertight circular shaft while Figure 8 shows sectional view of the shaft with the ring beams. The effective thickness of the wall for design purposes takes into consideration potential deviation on plan and also verticality. Guide wall for secant pile installation is required to ensure pile deviation are within allowable tolerance.

Figure 9 is picture taken during excavation works while picture in Figure 10 is taken after completion of the two circular shafts and ready for TBM launching.

5.0 CASE HISTORY OF DEEP EXCAVATION FOR MASS RAPID TRANSIT IN LIMESTONE FORMATION

5.1 Introduction

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 seven (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 (known as Pasar Rakyat Station during design development), 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 11 shows the location of the construction site.

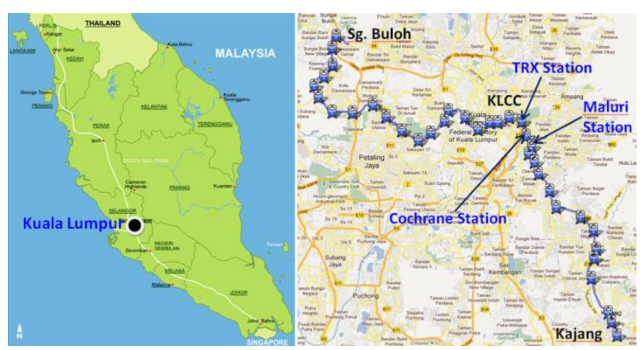


Figure 11 Location and alignment of Klang Valley Mass Rapid Transit (KVMRT) Sg. Buloh to Kajang (SBK) line.

5.2 Geology and Subsoil Conditions

Figure 12 shows the Geological Map of Kuala Lumpur (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.

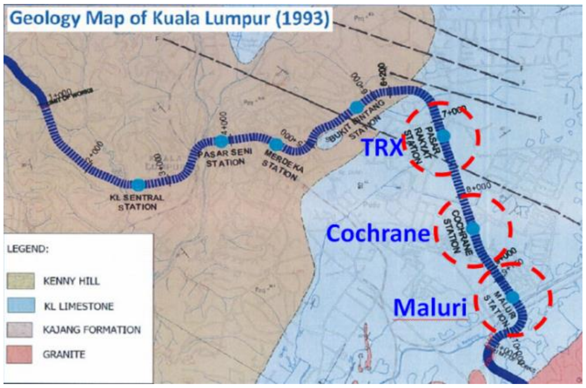


Figure 12 Geological Map of Kuala Lumpur superimposed with tunnel alignment.

The overburden subsoil above limestone generally comprises of loose silty sand to sand materials with SPT'N' value less than 4. Average unit weight and permeability of subsoil are 18 kN/m^3 and 1x10^{-5} m/s respectively. Interpreted effective shear strength is c'= 1kPa and ϕ'= 29°. 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 13 presents some typical features of limestone formation.

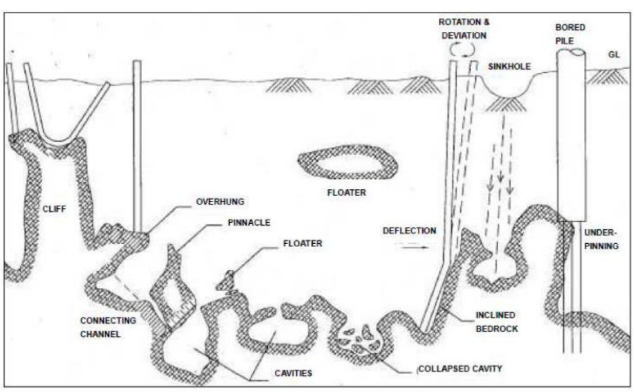


Figure 13 Typical features of limestone formation [13].

5.3 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 14 shows typical arrangement of the secant pile wall.

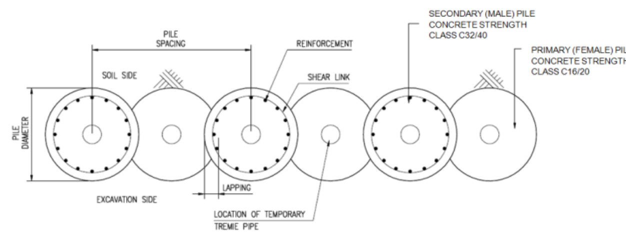


Figure 14 Typical arrangement of secant pile wall.

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 extent of overlapping of the secant piles are governed by pile installation verticality, pile deviation and pile depth [14]. After reviewing the piles as-built performance, the recommended overlapping values of secant pile wall are shown in Table 2 where overlapping of up to 34% were specified to ensure water-tightness of the wall.

Table 2 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. Serviceability 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, Is(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.

Table 3 Partial load factors.

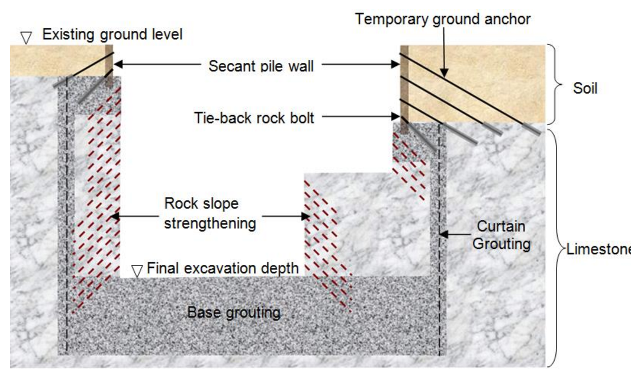
Load case	EL	DL	LL	TL	IL
Working condition	1.4	1.4	1.6	1.2	NA
Accidental impact	1.05	1.05	0.5	NA	1.05
One-strut failure	1.05	1.05	0.5	NA	NA

Note:
EL – Earth pressure and groundwater
DL – Dead load
LL – Live load
TL – Tempreture effect
IL – Accidental impact load
NA – Not applicable

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 [12]. 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 3.

5.4 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 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 15.



(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).

Figure 15 Schematic of excavation works.

Grouting techniques rely much on local experiences and contractor workmanship. Grouting works are mainly carried out in 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 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 4.

Table 4 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 10m^3 for every grouting zone in 5m depth.

5.5 Instrumentation and Monitoring

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 comply with environmental requirements, generated vibration and noise were monitored in accordance with guidelines by the Department of Environment (DOE), Malaysia.

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.

5.6 Construction

Excavation works started in year 2012 at Cochrane station for four (4) numbers 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. When the excavation reached final level of 32m below ground, base slab were casted to provide a platform for TBM launching preparation. Figure 16 shows the second TBM launching condition at Cochrane station.

Figure 16 Cochrane Station (launching of second TBM).

Figure 17 TRX Station (excavation in progress).

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 17 shows the excavation works at TRX station.

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 rig with deep rock drilling which is beyond the machine capacity and as such, deckpost formed by four (4) numbers of micropiles in a group was proposed as an alternative for support underneath the existing electrical transmission line.

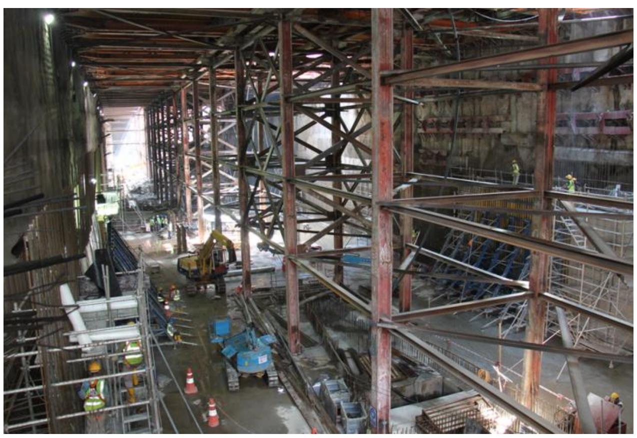


Figure 18 Maluri Station (base slab casting with live traffic on top).

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 18 shows the base slab casting at Maluri station and Figure 19 shows the excavation works with strutting support at Maluri Portal.

Figure 19 Maluri Portal (excavation in progress).

6.0 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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