Experiences and innovations for foundations, tunnels and underground developments in Malaysia

TAN Yean-Chin[1], CHOW C. M.[2] & KOO K. S.[3]

[1,2 & 3] G&P Professionals Sdn Bhd, 39-5, Jalan Tasik Selatan 3, Bandar Tasik Selatan 57000 Kuala Lumpur, Malaysia.
www.gnpgroup.com.my

ABSTRACT

Malaysia experienced rapid growth since it enters the 21st century as the nation aspires to achieve developed nation status by 2020. With the rapid growth, massive investments in infrastructure works such as highways, railways and urban transits together with property developments have given geotechnical engineers opportunities to provide innovative solutions for challenging engineering problems in difficult ground conditions such as soft ground overlying limestone formation or the heterogeneous metasedimentary formation known locally as Kenny Hill formation.

This paper describes and shares some of the recent experiences and innovations from Mass Rapid Transit (MRT) development in Kuala Lumpur. The Klang Valley Mass Rapid Transit from Sg. Buloh to Kajang (KVMRT-SBK Line) is one of the major infrastructure projects launched in 2011 and completed in 2016. It is the first MRT project in Malaysia. The underground section of 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 constructed in the city areas. The second line of the MRT from Sungai Buloh to Serdang to Putrajaya (KVMRT-SSP Line) commenced in 2016 and targets to be completed by 2021. When completed, this line will have 10 underground stations and 13.5km long twin tunnels (from Jalan Ipoh to Desa Water Park) alongside with several shafts for emergency escape and ventilation purposes.

Part of the design and construction risks and hazards for foundations, tunnels and underground developments are related to existing ground conditions and geological formation. Generally, geological formation of Kuala Lumpur consists of Kuala Lumpur (KL) Limestone, Granite, Kenny Hill Formation, Kajang Formation and Hawthornden Schist. KL 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 KL Limestone are mainly loose silty sand with drastic variation in thickness.

The nature of highly variable limestone properties is known to create quality control issues during bored pile installation. Conventional rock coring measurement is mainly dependent on piling operator experience and visual inspection from cored out rock material in order to determine the final rock socket length during bored pile construction. Another approach is confirmation based on rock probing results but the approach is inadequate as the numbers of rock probes are usually not adequate and because of the highly variable limestone properties, uncertainties still remained with regards to accuracy of the rock probe results. A novel quantitative method proposed by the Authors was implemented since early 2000s as improvement to site verification for rock coring termination criteria based on correlated point load index of $_{Is(50)}$ will be discussed in this paper.

To facilitate underground development in urban areas where land scarcity is a major issue, basement or underground space is required and the depth of basement and underground space is increasing with higher development density. As such, appropriate temporary or permanent earth retaining structure system (ERSS) needs to be considered based on time and cost effectiveness, project requirements, ground conditions as well as impact to existing buildings and structures. Suitable ERSS adopted for vertical rock excavation in limestone adjacent to retaining wall involving rock slope strengthening works, surface protection, controlled blasting and vibration control was successfully designed and implemented by the Authors.

The challenges of tunnelling in limestone are risk and hazard from limestone natural features. Irregular limestone bedrock profiles are expected during tunnelling works with unpredictable cavities, fractured/weathered rocks, underground water solution channels, mixed face and slimezone. Ground investigations, ground improvement works and tunnel face support need to be carefully evaluated to minimize potential tunnelling risk to the above ground structure, safety and construction progress. Other design and construction risks for tunnel and underground development are potential damage to existing buildings and structures. A series of impact assessment shall be carried

out beginning from identifying affected units, damage classification, detailed evaluation, necessary protection measures and continuous monitoring. This paper summarised some of the challenges faced and innovative methods proposed and used successfully to overcome these challenges.

Keywords: Foundation, Tunnel, Underground development, Limestone, Metasedimentary

1 INTRODUCTION

Malaysia experienced rapid growth since it enters the 21st century as the nation aspires to achieve developed nation status by 2020. With the rapid growth, massive investments in infrastructure works such as highways, railways and urban transits together with property developments have given geotechnical engineers opportunities to provide innovative solutions for challenging engineering problems in difficult ground conditions.

Since 2010 several mega projects have been constructed and some are still on-going. The projects are KVMRT–SBK Line (Line 1 from Sg. Buloh to Kajang), KVMRT-SSP Line (Line 2 from Sg. Buloh to Serdang to Putrajaya), LRT extension, LRT Line 3 (Damansara to Klang), Kuala Lumpur International Airport 2 (KLIA2), Skyscraper PNB 118 (644 meters high), Tun Razak Exchanges (TRX) development with a skyscraper under construction, i.e. The Exchange 106 (452 meters high), Southern Electrify Double Tracks (Seremban to Johor Bharu), Melaka Gateway, Refinery and Petrochemical Integrated Development (RAPID) in Pengerang Johor, West Coast Expressway (Taiping to Banting), Kuala Terengganu Tower Drawbridge, etc.

In Kuala Lumpur (capital city of Malaysia), complex geological formations such as alluvium overlying limestone formation (KL Limestone) or the heterogeneous metasedimentary formation known locally as Kenny Hill formation are commonly encountered at city centre. Figure 1 shows geology of Kuala Lumpur with limestone and Kenny hill at city centre and some Granite or Hawthornden Schist formation around the city. Recent underground development of MRT Line 1 and Line 2 are mainly located in KL Limestone and Kenny Hill formation that pose geotechnical challenges to engineer especially in KL Limestone formation. A total of 3.6km tunnel and 3 underground stations in Line 1, around 5km tunnel and 3 underground stations in Line 2 are constructed in limestone formation. The deepest shaft excavation in limestone is 58m below ground at intervention shaft (IVS2) of Line 2.

Many future developments and infrastructures in Kuala Lumpur will be planned to be adjacent or nearby to existing tunnels and underground structures along the completed MRT Lines. This will require extensive geotechnical input to ensure safety and serviceability of the existing tunnels and stations. Therefore, geotechnical assessment is compulsory and enforced under Malaysian Railways Act and Regulations to ensure public safety and design compliance. Other than city development, intercity transportation projects such as KL-SG High-Speed Rail (Kuala Lumpur to Singapore), Intercity Railway System (Kuala Lumpur to East Coast region called ECRL), Penang Master Transport Plan, etc. are being planned. It is expected that more and more projects in Malaysia will require innovative geotechnical input especially in difficult geology and ground conditions such as soft ground, hilly terrain, urban areas, etc.

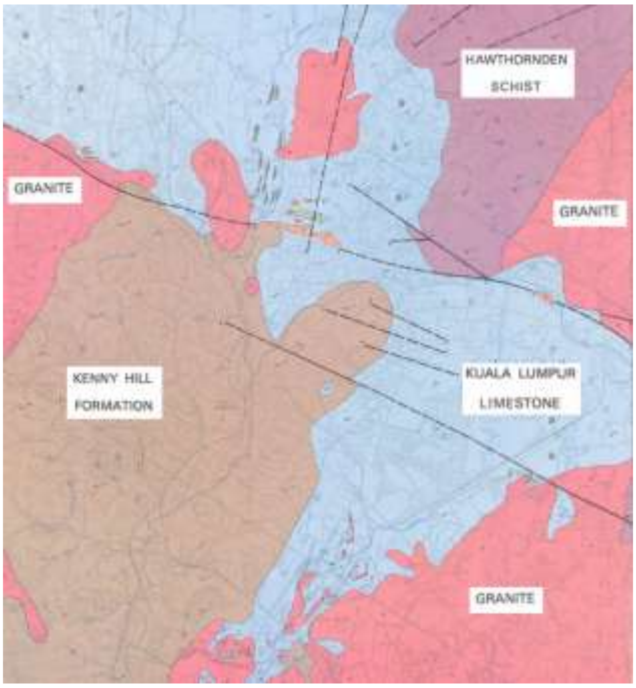

Fig. 1. Geology of Kuala Lumpur

2 UNDERGROUND DEVELOPMENT

2.1 Deep Excavation in Limestone

KL Limestone is infamour 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 soil above Kuala Lumpur Limestone are mainly loose silty sand. The thickness of overburden soil varies significantly due to the irregular topography of the limestone bedrock. Some experiences of deep excavation in limestone for the completed KVMRT Line 1 were described in Tan et al. (2015) and Koo (2013). Figure 2 presents some typical features of KL Limestone formation showing the complexity of natural features that may be encountered during geotechnical works. Proper ground investigations are required for design before commencement of physical works. Interaction and construction information updates are important to assess the actual variations at site compared to design assumptions. Design changes and calibrations due to site variations during construction period shall be closely monitored until completion.

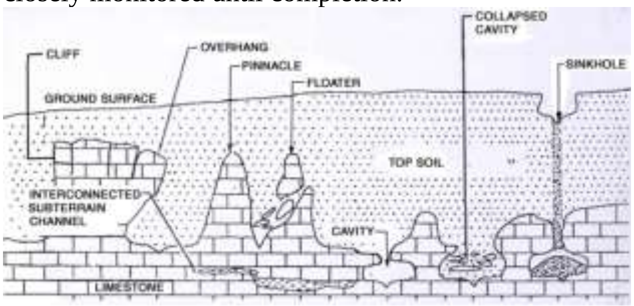

Fig. 2. Typical features of limestone formation

Overburden soil above limestone bedrock in Kuala Lumpur region generally consist of alluvium deposit, shallow Kenny Hill formation at boundary of two different geological formations or backfilled materials from tin mining activities in the middle of the 19th century, etc. Secant bored piles (SBP) wall are commonly used as temporary wall in KVMRT projects. Design approach for retaining wall and support system were described in Tan & Chow (2008) and Tan et al. (2016). Other than conventional retaining wall, ground improvement methods such as deep soil mixing (DSM) method to form a gravity wall was adopted for shallow bedrock not more than 10m deep. For example, Chan Show Lin Station of KVMRT2, the retaining wall system adopted at the perimeter are 80% DSM wall while SBP wall was used in remaining areas with deep rock valley. Design concept and adopted parameters for DSM were described in Tan et al. (2019A).

2.2 Groundwater Control in Limestone

Groundwater control is one of the important criteria to be considered in excavation works. Groundwater drawdown may lead to excessive ground settlement and potential occurrences of sinkholes surrounding the excavation. Potential risk of excessive groundwater ingress into excavation pit shall also be evaluated especially in limestone formation. Natural features of solution channel with cavities and highly fractured limestone connected to excavation pit may cause large inflow of water carrying fines in the soil into excavation pit. Therefore, grouting in limestone was carried out as a risk mitigation measures for groundwater control. Schematic of the grouting and excavation works are shown in Figure 3.

Grouting techniques rely much on local experiences and contractor’s workmanship. Grouting works are mainly carried out in limestone bedrock to reduce the rate of groundwater inflow and to reduce available pathways of water flow into the 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 grouting in stages using single packer in ascending or descending direction in order to inject cement grout suspension into existing pathways, fissures, cavities and discontinuities within the rock formation. Fissure grouting are carried out in stages (primary, secondary, tertiary grouting, etc.) where additional grouting holes may be required after reviewing the grout intake from primary grouting. Rock fissure grouting is also adopted for base grouting where needed at larger grout hole spacing. If any cavities are detected during drilling or grouting works, compaction grouting using cement mortar to fill the cavity encountered.

During construction, the allowable magnitude of groundwater drawdown depends on many factors such as the thickness of the overburden soil, stiffness of the soil, type of foundation of structures that could be affected by the settlement of the ground, etc. Damage and risk assessments shall be carried out to determine the allowable magnitude of groundwater drawdown in monitoring scheme. Recharge wells can be implemented to prevent groundwater drawdown near to critical structures.

Fig. 3. Schematic of limestone grouting and excavation works.

2.3 Vertical Rock Excavation in Limestone

In KVMRT Line 1, three underground stations, one shaft and one southern portal were completed with vertical rock excavation. In Line 2, three underground stations and three shafts are on-going with vertical rock excavation. The deepest vertical rock excavation in KL Limestone in Line 1 was 45m deep TRX station which is also one of the deepest MRT Station in Southeast Asia. In Line 2 the deepest vertical rock excavation is 58m deep for circular shaft of IVS2.

Before commencement of rock excavation works, sufficient subsurface investigations were carried out to have a better understanding of bedrock profile, rock quality as well as design parameters for rock stability assessment. A series of boreholes as primary investigation method were assigned along the excavation perimeter and at the proposed rock excavation alignment. Recommended borehole spacing is 20m to 30m with minimum borehole depth of 10m below final excavation level. Additional boreholes can be added if any abnormal or drastic changes in rock levels are found in between two boreholes. All the rock core samples shall be properly stored in core boxes with clear identification of rock core information for future reference. Engineering geologist with experience in rock slope design is required for rock core inspection, borehole supervision and logging in accordance to Malaysian standard (e.g. weathering condition, fracturing condition, discontinuity, joint condition, cavity and infilled materials). Strength parameters can be obtained from laboratory testing on selected core samples.

Borehole tele-viewer tests were carried out for selected boreholes after reviewing the rock cores to measure potential dip direction and angle of rock joints found in drilled boreholes. These results will give a general observations of joints persistence and to be further verified through rock joints mapping by experienced geologist on the exposed rock surface during excavation. Other than boreholes, geophysical survey method such as micro-gravity and resistivity were adopted for wide area survey. The survey area usually covers the excavation area and also extended to a distance of minimum one-time excavation depth behind the rock face. Preliminary interpreted bedrock profiles with rock condition information will be established based on investigation results for construction and rock stabilization design reference.

Prior to the excavation works, retaining walls to support the soil above the bedrock and grouting works will be completed first. Records from installation of retaining wall will provide a more accurate bedrock profile along the wall alignment. Curtain grouting of limestone bedrock surrounding the excavation area will also provide more details on rock conditions such as bedrock level, cavities found, fractured rocks with high grout intake and intact rock with low grout intake. Risk classifications proposed based on subsurface investigation data and construction information from pile installation and grouting records are shown in Table 1.

Rock excavation with controlled blasting method was carried out in stages (not exceeding 3m in vertical height). The exposed rock surface are mapped and if necessary, rock strengthening measures were carried out. Mapping works were carried out by qualified engineering geologist with experience in rock slope assessment and strengthening design. The assessment procedures consist of rock mapping, identify potential failure, rock slope strengthening analysis and design. Rock mapping information consist of rock joints (dip direction/dip angle), joint conditions (tight/infill), infill materials (clay/sand), groundwater conditions (dry/wet/ingress), cavity (size/infill material), weathered and fractured condition (high/medium/low), etc. Mapping report will clearly identify mapping area (e.g. chainage and reduced level) with a high-resolution photo together with its own identification number (e.g. reference chainage and depth information) for future reference and checking. Proper records and identification are very important as after completion of the strengthening works, the rock surface will be shotcreted and thus, covering up the rock surface making visual inspection later not feasible.

Table 1. Risk Classification of Rock for Vertical Excavation.

Risk category	Descriptions
Low	Intact rock (e.g. RQD > 75), No cavity and no major discontinuity found, Tight joint with persistence dip angle, No anomalies found in geophysical survey result, Low grout intake from grouting record.
Medium	Fractured rock (e.g. 75 > RQD > 25), No cavity and no major discontinuity found, Tight joint with few persistence dip angles, Minor anomalies found in geophysical survey, Medium grout intake from grouting record.
High	Highly fractured rock (e.g. RQD < 25), Small cavity and minor discontinuity found, Infilled joint with inconsistent dip angles, Weak zone found in geophysical survey, High grout intake from grouting record.
Very High (to study feasibility of rock excavation)	Highly fractured/weathered rock (RQD = 0), Big cavity and intermediate weak layer found, Infilled joint with weak materials, Weak zone found in geophysical survey, high grout intake in multiple grout holes.

Note: RQD - Rock Quality Designation

Any localised or global potential rock failure mode with insufficient safety factor are strengthened to enhance the rock excavation stability before proceeding to next excavation depth. Generally, rock bolts in various length coupled with shotcrete surface were used for rock slope strengthening works as shown in Figure 4.

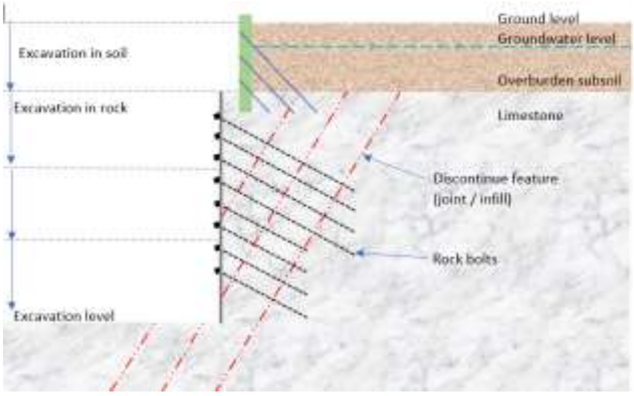

Fig. 4. Schematic of rock excavation strengthening works

Design approach for rock excavation was established based on two different conditions in Table 2 with respective required factor of safety (FOS) in Table 3.

Table 2. Design Approach for Rock Excavation.

Category	Descriptions	Joint parameters
Design Approach A	Intact rock surface. Persistent tight joints. Dry surface condition.	Hoek and Brown criteria
Design Approach B	Fractured rock surface, Infilled joints, Wet surface condition.	Refer to infilled material properties or back analysed parameters

Table 3. Required FOS.

Category	Required FOS	Groundwater level
Design Approach A	1.4 for dry analysis	No water
	1.05 for wet analysis (worst case)	Monitored highest level
Design Approach B	1.4 for wet analysis	Interpreted design water level
	1.05 for wet analysis (worst case)	Monitored highest level

Fig. 5. Overall View of Conlay Station

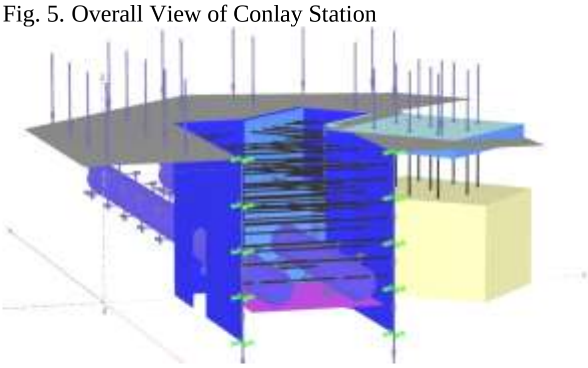

Fig. 6. 3D FEM Modelling for Impact Assessment

2.4 Excavation Adjacent to Existing Buildings and Structures

One of the challenges of excavation works in urban area is the impact to nearby existing buildings and structures. Majority of the MRT underground stations are located near to existing buildings that required detailed impact assessments to ensure the safety and serviceability of these buildings. In addition, the design of the retaining wall and support system also have to cater for additional loading and to ensure not affecting the safety and serviceability of the existing structures. Figure 5 shows the overall view of Conlay underground station with nearby condominium located very near to excavation area. Detail 3D Finite Element Method (FEM) modelling were carried out for damage assessment and prevention. 3D-FEM model is shown in Figure 6.

2.5 Excavation Adjacent to Completed MRT Structures

With urban development, there will be more construction of buildings and basement adjacent to completed underground structures or crossing existing tunnel. This will require geotechnical input to work within the constraints and limitations set either by existing laws and regulations or conditions of the adjacent buildings. To ensure public safety and railway operation, Malaysia Railway Act 1991 and the Railways Regulations 1998 were enforced for all future development within the Railway Protection Zone (RPZ). Permit to work within RPZ must be obtained from relevant Authorities in compliance with the required rules and regulations.

Tan et al. (2019B) shares the experience on design and construction of earth retaining system for basement structure at AEON Maluri which is located adjacent to Maluri MRT station. The site was originally an open carpark and was being proposed to be developed into two levels basement structure with 12m retained height using contiguous bored pile wall as earth retaining system. The proposed basement is to connect Maluri MRT Station and AEON Maluri shopping mall for direct public access.

The Maluri MRT station was completed and in operation since year 2017 before commencement of proposed AEON Maluri basement structure and therefore, the basement construction works have to comply to the requirements set for construction in Railway Protection Zone (RPZ). Permit to work within the RPZ has to be obtained from SPAD (Malaysia Land Transport Authority) with technical evaluation from Mass Rapid Transit Corporation (MRTC). Figure 7 shows the overview of excavation works for the project.

Fig. 7. Overview of Excavation Works

Some of the restrictions under second reserve of Railway Protection Zone (RPZ) are as below: -

a) Pile debonding is required for foundation within Second Reserve to limit load transfer to MRT structure.
b) No blasting shall be permitted.
c) Vibration during construction shall be controlled to not more than peak particle velocity (ppv) of 15mm/s
d) Limit of distortion in the track and plinth not more than 1:2000 or total movement not more than 15mm (in any plane)

3 TUNNEL DESIGN AND CONSTRUCTION

3.1 Support of the cavity and settlement during tunnel advancement using TBM

Proper assessment of face support pressure is important to ensure safety of tunnel advancement using TBM and also to mitigate excessive settlement and formation of sinkholes (Figure 8) especially for tunnelling works in limestone formation.

Fig. 8. Typical sinkhole observed on ground surface due to underground construction works in limestone formation.

For KVMRT Line 1 and 2, a specially developed Variable Density (VD) TBM were used for tunnelling works in karstic limestone formation. The VD TBM can operate in four (4) different modes:

a) EPB closed mode with dry or liquid conveyance
b) EPB closed mode with bentonite support and dry or liquid conveyance
c) Mixshield mode with bentonite slurry of low density (LDSM mode)
d) Mixshield mode with bentonite slurry of high density (HDSM mode)

The assessment of face support pressure due to earth pressure is carried out based on German practice as summarised in DAUB(2016) and is essentially based on Horn's failure mechanism (Figure 9). The allowable operational pressures at the tunnel crown based on German safety concept for tunnel face stability are summarised in Figure 10. The approach summarised in DAUB (2016) has proven to be adequate for tunnelling in limestone formation with karstic features even at locations with mixed tunnel face where the tunnelling works for KVMRT Line 1 were successfully completed with no major incidences. However, proper coordination of advance soil investigation along the proposed tunnel alignment together with ground treatment works to treat mixed face conditions are essential to ensure safe and smooth progress of the tunnelling works.

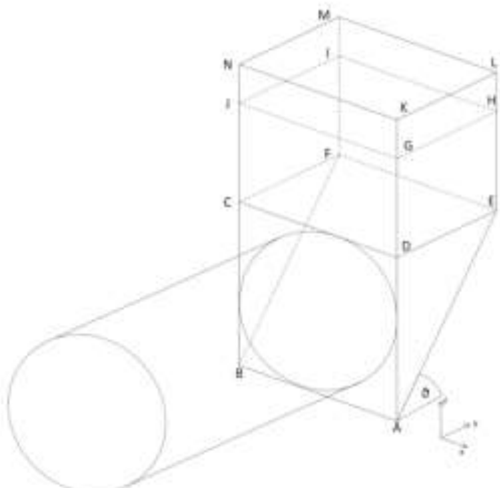

Fig. 9. Horn's failure mechanism with the wedge (ABCDEF) and prism (CDEFKLMN) considering groundwater level (GHIJ) (Daub, 2016).

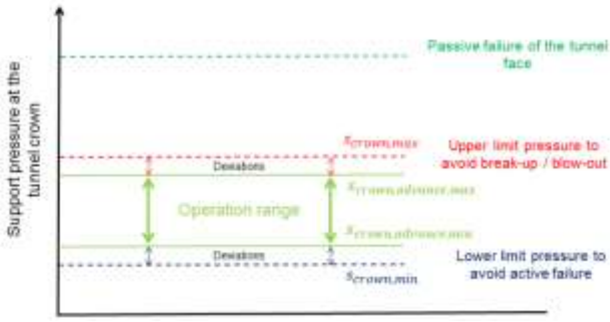

Fig. 10. Allowable operational pressures at the tunnel crown (Daub, 2016).

3.2 Design of segmental rings for tunnels

The segmental lining consists of a bolted gasketted precast concrete segmental lining. It consists of 7 segments and a key (Figure 11). The universal ring is tapered to achieve a minimum curve radius of 155m to accommodate alignment and level correction during construction.

The stresses on the segmental lining are assessed based on the methods of Muir Wood with DJ Curtis discussion (1976) and also checked with numerical analysis using PLAXIS software. In soft ground, the segments are designed based on Duddeck & Erdmann (1982). In addition to determining the stresses on the segmental lining due to earth pressures, stresses induced due to construction process especially on the joints are important. Critical stresses due to construction includes high partial surface pressures developed under the jack shoes and the resulting split tensile forces in radial and circumferential direction (Figure 12). References can be made to DAUB (2013) on methodology to assess internal forces and stresses from the construction process. It is important to note that the design of the segmental lining may be governed by the stresses on the joints especially for joints where no splitting reinforcement are provided.

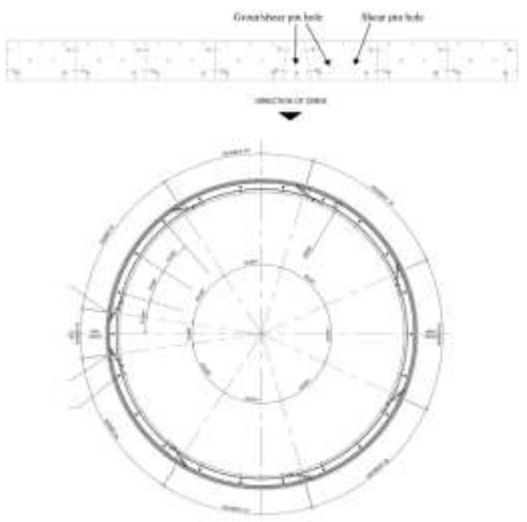

Fig. 11. Layout of segmental lining.

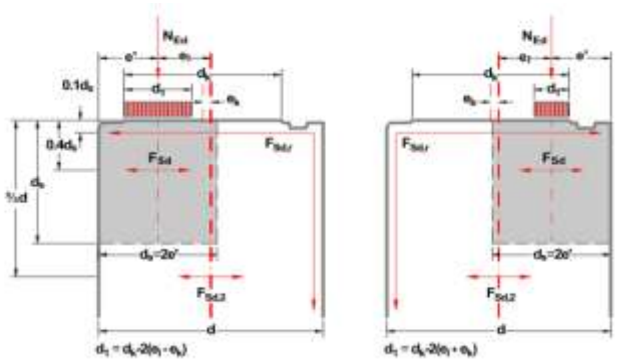

Fig. 12. Segment joint with load eccentricity and split tensile stress (Daub, 2013).

3.3 Existing Piles Removal for TBM

While the TBMs are capable of addressing variety of soils and rock conditions, however, tunnelling through reinforced concrete piles is not advisable as the ductile rebars are likely to bend instead of being crushed or cored through, thus potentially jamming the TBM cutterhead. Due to restrictions of land use or land acquisitions in addition to any other technical or financial constraints, there will be situations where tunnel alignment has to cross beneath high-rise buildings or structures with pile foundations. In these cases where buildings will remain in service throughout the tunnelling operation, underpinning and pile removal of existing structures are necessary for TBM passage. Two case studies are documented by Tan et al. (2019C).

Case 1 – Pile removal below 5-storey building

The first case involves the underpinning of a 5-storey building located above the proposed tunnel alignment with some of its piles located within the tunnel horizon. Piles removal was designed to be carried out while the building was occupied. Two feasible options to remove existing piles were considered. Option 1 was to remove the pile manually by creating an access via hand-dug caisson shaft with horizontal mined adit, while Option 2 was to core the piles in inclined direction from ground surface. For Option 2, in order to ensure stability of the drilled holes during coring work, ground improvement by means of jet grouted block was first carried out surrounding the edge of the inclined drilled holes. Once the grouted block has gained sufficient strength, coring is to be carried out at the planned direction, inclination and distance. The void left by the coring will be backfilled with cement grout. This option has been adopted as it offers a shorter pile removal duration compared to Option 1. It should be noted that unlike Option 1, the pile removal from Option 2 may not be as complete or thorough even within the tunnel horizon. However, this is considered manageable for the tunnel operation as long as majority of the pile material (especially the pile reinforcement) can be removed; preventing jamming of TBM's system.

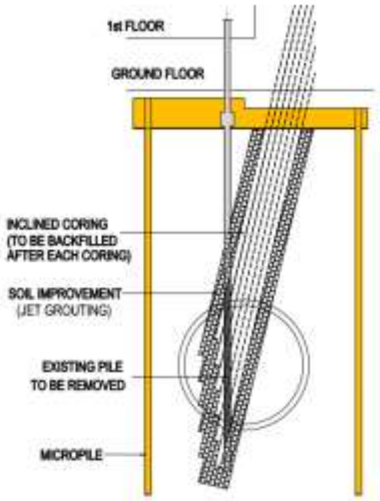

Fig. 13. Typical section of pile removal from ground surface using coring rig.

3.4 Deepest Intervention Shaft in Malaysia

The 18m diameter intervention shaft IVS2 in KVMRT Line 2 is up to 58m deep which is the deepest urban excavation in Malaysia. A suitable and economical temporary earth retaining structure using deep soil mixing (DSM) block was used to provide excavation support in overburden alluvial soil, followed with vertical rock excavation up to a total of about 58m deep below the existing ground in KL Limestone formation. Ground treatment scheme in limestone is established to minimize drawdown of groundwater during deep excavation to prevent excessive ground settlement that could cause distress to adjacent buildings and structures. A mined adit is to be constructed to connect the shaft at mid-depth with a south-bound bored tunnel drive. Meanwhile, the north-bound tunnel drive passing through the bottom of the shaft is constructed by tunnel boring machine cutting through 'soft-eye' and pre-constructed concrete-filled block within the shaft compartment. Tan et al. (2019D) covers the challenges in design and construction of the deep excavation works for this intervention shaft, ground improvement scheme for the bored tunnelling drive near the shaft, temporary works for the mined adit and unconventional construction sequence of the bored through tunnel/adit. Figure 14 shows schematic diagram of IVS2.

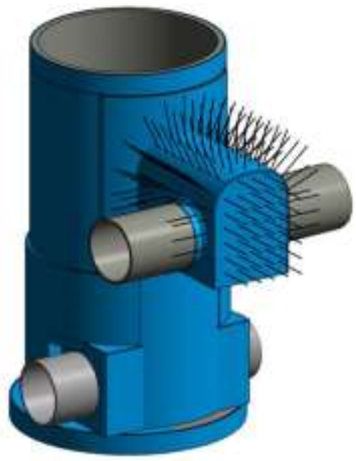

Fig. 14. Schematic diagram of IVS2

4 FOUNDATION

4.1 Foundation Assessment Impact to Future MRT Tunnels

Chow and Teh (2019) presented a high-rise development of up to 60-storey high to be developed in limestone formation with two future MRT tunnels cutting across the proposed development area. The foundation system for the building comprises of bored piles socketed into shallow limestone bedrock with most of the pile terminated above the tunnel axis level. Due to the close proximity of the proposed tunnels, 3D finite element analyses (3D-FEM) were carried out to address the impact of load transfer from pile foundation to the MRT tunnels. The main objective of the analysis is to ensure compliance to requirements of Railway Protection Zone (RPZ). 3D-FEM analysis was employed to capture complex soil-structure interaction involving MRT tunnels at varying depths and orientations and piled foundation supporting loadings with non-regular pattern that could not be properly simulated in 2D-FEM modelling. Advance finite element analyses have demonstrated the feasibility of the proposed development in compliance to the strict requirements of the Railways Regulations 1998. The analyses result also demonstrated the importance of obtaining reliable rock stiffness parameters to assess impact of the proposed development to the future tunnels in limestone formation.

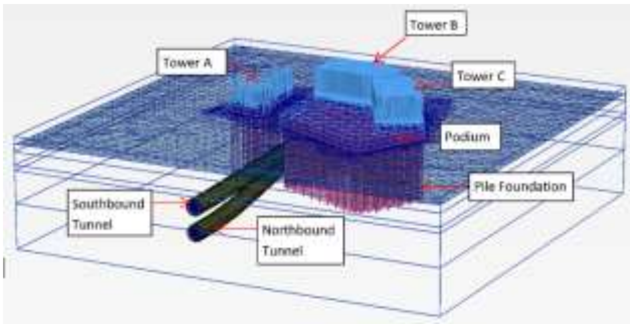

Fig. 15. Plaxis 3D model of proposed foundation loading including MRT Tunnels

4.2 Foundation Assessment Adjacent to Completed MRT Structures

The proposed development at Jalan Tun Razak, Kuala Lumpur comprising of 3 tower blocks (i.e. office, hotel, and condominium) with 8 levels of podium car park and 1 level of basement, is located beside Tun Razak Exchange (TRX) underground station of KVMRT Line 1.

The proposed site is underlain by Kuala Lumpur Limestone formation. There are rock floaters and infill cavities owing to the karstic features of limestone. These cavities were treated by way of pressured cement-sand mortar to compact and fill in the cavity. The overburden soil layers mainly consist of sandy CLAY and silty SAND. The SPT-N values of overburden soil are generally less than 15. The depth of limestone bedrock varies from 5m to 8m below ground level.

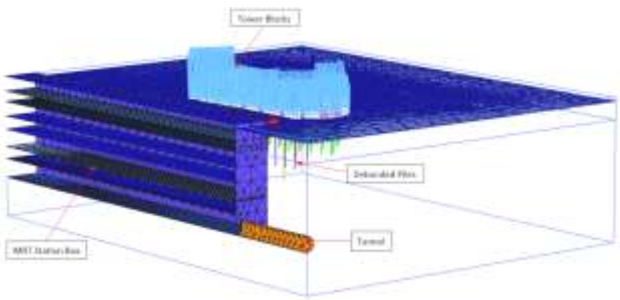

Figure 16: 3D FEM Model of TRX station and loading from proposed development

3-D Finite Element Analyses (FEM) were carried out to simulate the construction of 1 level basement, pile foundation and the application of varying building loads from different blocks to assess the stress-induced and deformation onto the existing TRX underground station adjacent to the proposed development. For the pile foundation constructed in Second Reserve area, they were modelled to be de-bonded within the zone of influence of underground structures with zero shaft friction to comply with the Railway Act 1991. However, application for waiver to requirements of Railway Act 1991 is required as the piles to be constructed in the Second Reserve will be debonded to a 60 degrees line (instead of 45 degrees) from the soffit of the station box in order to explore a more economical design.

The results of 3-D FEM analyses have successfully proven that the deformation of the existing underground tunnels and TRX station is within the permissible limit set by Railways Act 1991. The analyses also show that the induced forces (i.e. bending moment) on the existing underground TRX structures will not exceed the structural capacity designed. Therefore, pile foundation in Second Reserve area will be constructed with de-bonding based on a steeper 60 degrees line instead of the more conservative 45 degrees line. This outcome will reduce the de-bonded length of piles and directly reduce the depth of piles significantly. In summary, 3-D FEM analyses are capable of properly simulating the stresses induced by loadings transferred to the piles of the proposed development onto the existing TRX underground station and tunnels instead of overly conservative results obtained from 2-D FEM analyses.

4.3 Improvement to site verification for rock coring termination criteria based on correlated point load index of $I_{s(50)}$

The nature of highly variable limestone properties have also created quality control issues during bored pile installation. Conventional rock quality measurement for termination of pile and for payment purposes are mainly dependent on piling operator experiences like change of tools from soil boring to rock coring equipment and visual inspection from cored out rock material in order to determine the final rock socket length during bored pile construction. Another approach is confirmation based on rock probing results but the approach is inadequate as the numbers of rock probes are usually not adequate and because of the highly variable limestone properties, uncertainties still remained with regards to accuracy of the rock probe results.

Tan & Chow (2013) introduced a novel approach for verification of rock socket in bored piles construction based on correlated point load index of $I_{s(50)}$ which has been successfully implemented in limestone formation and also other types of geological formation in Malaysia as criteria for site control and payment purposes. Prior to introduction of this approach, there were always disputes and even court cases on what is payable or defined in contract as rock socket during construction which is much higher cost compared to boring in soil.

The recommended definition of rock coring shall fulfill all three (3) criteria below:

a) Change of tools to rock coring tools, and
b) The rock materials shall be verified by carrying out point load test on at least three (3) rock samples to achieve minimum index strength, $I_{s(50)}$ of certain values based on site specific rock strength (e.g. 3MPa) subject to site confirmation on typical rock lump sizes based on the size correction factor, $F = (De/50)^{0.45}$ where De is the equivalent core diameter in mm ($I_{s(50)} = F*I_s$), and[1]
c) Recovered rock coring materials of more than 50% subject to site calibration upon start work unless otherwise agreed by the engineer.[2]

Note:

(1) The value of $I_{s(50)}$ varies for different rock formation

and site conditions. Therefore, the Geotechnical Engineer designing the bored piles have to determine the proper value to be used as it is also related to rock socket capacity adopted for the bored piles.
(2) The criteria of recovered rock coring materials can be relaxed by the Consultant based on site conditions and rock types as it is dependent on machine capabilities.

Any coring/boring in rock like materials that do not fulfil the definition of rock coring mentioned above shall be considered as partial rock coring where the rock socket and payment can be adjusted based on the equivalent rock length as tabulated in Table 4.

Table 4. Sample of equivalent rock socket length for every 1m of partial rock coring not fulfilling $I_{s(50)}$ of 3MPa.

Average Values $I_{s(50)}$ per layer (Mpa)	Equivalent Rock Socket Length to $I_{s(50)}$ = 3MPa (m)
< 0.5	0
0.5 to < 1.0	0.2
1.0 to < 2.0	0.4
2.0 to < 3.0	0.6
≥ 3.0	1.0

For example, for a 2m rock coring with $I_{s(50)}$ = 2MPa, the equivalent rock socket length to $I_{s(50)}$ = 3MPa is 1.2m where the rock coring payment is solely for this 1.2m rock socket length only and the remaining 0.8m length to be paid as soil boring.

Minimum two (2) sets of point load tests shall be conducted for each pile and one (1) set of point load test for every 1m rock coring. Each set of point load test shall consist of minimum three (3) rock fragments to be selected by Engineer's representative from the most representative rock samples recovered for rock core at every 1m depth. The provision of point load test equipment at site for the above-mentioned testing shall be by the Contractor and the equipment is simple and can be easily operated at site. In a recent project described by Chow et al. (2019), the method has proven to be effective also for bored pile works in limestone-granite interface zone. Based on results of instrumented test pile on 900mm and 1800mm bored piles, the mobilised shaft friction ranging from 900-1350kPa and end bearing ranging from 1900-6750kPa for the granite rock matched well with the assumed design parameters after adjusting rock socket during construction based on site-obtained point load strength, $I_{s(50)}$ values as shown in Table 5.

Table 5. Equivalent Rock Socket Length.

Test Pile Results					Calc. Equiv. Socket Length (m)
Test Pile	Layer	Is (50), MPa	$f_{s(ult)}$, MPa	Actual Equiv. Socket Length (m)	
TP900	4-5	1.418	862	0.57	0.47
	5-6	3.517	932	0.62	1.0
		Average for TP900 =		*0.6*	*0.7*
TP1800	6-7	6.430	1360	0.91	1.0
	7-8	8.474	1321	0.88	1.0
	8-9	4.415	1358	0.91	1.0
		Average for TP1800 =		*0.9*	*1.0*

Note: Assumed design ultimate shaft friction in rock (kN/m^2) = 1500kN/m^2 with $I_{s(50)}$ = 3MPa.

It can be seen from Table 5 that the average proposed equivalent rock socket length of TP900 is 0.7m which is similar to the actual tested equivalent rock socket length of 0.6m for TP900. The actual equivalent rock socket length of TP1800 is relatively close to the proposed equivalent rock socket length based on site-obtained $I_{s(50)}$ values.

5 CONCLUSIONS

The construction of major infrastructure works such as urban rail transportation and high-rise development in the city of Kuala Lumpur have given geotechnical engineers in Malaysia opportunities to provide innovative solutions for challenging engineering problems in difficult ground conditions such as soft ground overlying limestone formation or the heterogeneous metasedimentary formation known locally as Kenny Hill formation.With the successful completion of MRT Line 1 and various high-rise development works, the experiences and solutions adopted would be useful reference for future works with innovations such as vertical rock excavation in limestone formation of up to 58m deep with close proximity of adjacent structures, tunnel design and construction including pre-tunnelling measures such as ground improvement and removal of obstructions such as existing piles, etc., high-rise development adjacent to existing and future MRT structures complying to the strict requirements of Railways Act 1991 and improvement to site verification for rock coring of bored piling works.

ACKNOWLEDGEMENTS

The works cited in this paper including the case studies were carried out together with colleagues from G&P Geotechnics Sdn Bhd. and their contributions are gratefully acknowledged.

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