Proceedings of the 7th International Young Geotechnical Engineers Conference – Scott (Ed.)
© 2022 Australian Geomechanics Society, Sydney Australia, ISBN 978-0-9946261-5-8

Excavation in limestone formation of KVMRT Conlay underground station, Malaysia

Excavation dans la formation calcaire de la station souterraine de KVMRT Conlay, Malaisie

Chee Fong Wah
G&P Geotechnics Sdn Bhd, Malaysia, fwchee@gnpgroup.com.my

ABSTRACT: This paper describes the deep excavation design concept for the Klang Valley Mass Rapid Transit (KVMRT) Second line (Putrajaya line) underground station, namely Conlay Station (CLYS), located within the vicinity of Kuala Lumpur City Centre underlain by Kuala Lumpur limestone formation with natural karstic features and irregular bedrock profiles. Karstic Kuala Lumpur Limestone condition poses challenges especially for the design and construction of excavation works to a maximum depth of 31.5m below existing ground level. To fulfil the construction requirements such as ground movement, preventing surrounding groundwater drawdown and economical design, selection of suitable temporary Earth Retaining Structure System to accommodate the aforesaid purposes are vital. Secant bored pile (SBP) wall supported with temporary ground anchor and strutting system had been selected for overburden subsoil excavation and groundwater control scheme. The experience (planning, design and construction) obtained from this project will be a helpful reference for similar excavation works, especially in karstic limestone formation.

RÉSUMÉ : Cet article décrit le concept d'excavation profonde pour la station souterraine de la deuxième ligne du Klang Valley Mass Rapid Transit (KVMRT) (ligne Putrajaya), à savoir la station Conlay (CLYS), située à proximité du centreville de Kuala Lumpur et reposant sur la formation calcaire de Kuala Lumpur avec des caractéristiques karstiques naturelles et des profils rocheux irréguliers. L'état karstique du calcaire de Kuala Lumpur pose des problèmes, notamment pour la conception et la construction des travaux d'excavation jusqu'à une profondeur maximale de 31,5 m sous le niveau du sol existant. Pour répondre aux exigences de construction telles que le mouvement du sol, la prévention de l'abaissement de la nappe phréatique environnante et la conception économique, la sélection d'un système de structure de retenue des terres temporaire approprié pour répondre aux objectifs susmentionnés est essentielle. Un mur de pieux forés sécants (SBP) soutenu par un système temporaire d'ancrage au sol et d'étayage a été choisi pour l'excavation du sous-sol de couverture et le contrôle des eaux souterraines. L'expérience (planification, conception et construction) acquise dans le cadre de ce projet sera une référence utile pour des travaux d'excavation similaires, notamment dans les formations calcaires karstiques.

KEYWORDS: Deep excavation, Kuala Lumpur Limestone, secant bored pile, KVMRT, Malaysia

1 INTRODUCTION

This paper shares the experience in the deep excavation design for the Klang Valley Mass Rapid Transit (KVMRT) Second line (Putrajaya line) underground station, namely Conlay Station (CLYS) located within the vicinity of Kuala Lumpur City Centre underlain by Kuala Lumpur limestone formation in Kuala Lumpur, Malaysia. The Kuala Lumpur limestone formation is notorious for its karstic features such as pinnacles and cavities which pose tremendous challenges to the engineers (Gue 1999, Neoh 1997, Yeap 1985 and Ting 1974). For such complex bedrock conditions, the tendency to overlook on the effects of highly variable bedrock level during the design stage could cause significant difficulties during the excavation works. As such, detailed and precise geotechnical design input is required.

The Putrajaya line is the second line of the KVMRT project to be developed. The project is projected to serve a corridor with a population of around 2 million people stretching from Damansara, a new township development in northwest Kuala Lumpur and its southern suburbs, to Putrajaya, Malaysia's federal administrative centre. The proposed alignment length is 57.7km, consisting of 44.2km of elevated tracks and 13.5km of underground tunnels. 36 operational stations with 27 elevated and 10 underground. Conlay Station is one of the underground stations located within the vicinity of Kuala Lumpur City Centre, with a maximum excavation depth of 31.5m below existing ground level. Figures 1 & 2 show the location of the construction site while Figure 3 shows the construction site layout plan of the station.

Figure 1. Map of Malaysia

2 GEOLOGICAL CONDITION

Figure 4 shows the Geological Map (ref: sheet 94 Kuala Lumpur 1976 and 1993, published by the Mineral and Geoscience Department, Malaysia) overlaid with the proposed tunnel alignment. The tunnel alignment begins from the north portal at Jalan Ipoh and exits from the south portal at Desa Waterpark. From the geological map, it can be observed that the tunnel alignment passing through different geological formations, i.e., granite formation from Sentul west station to Titiwangsa Station, Kenny Hill formation from Titiwangsa Station to Bandar Malaysia Station with the interference of limestone formation from General Hospital Station to Kampung Baru Station and also limestone formation from Conlay Station until tunnel exiting the portal at Desa Waterpark.

The nature of Kuala Lumpur limestone with karstic features included the pinnacle zone, overhangs, floaters, cavities, solution channels and collapsed weak soil just above bedrock, are potential hazards for deep excavation work.

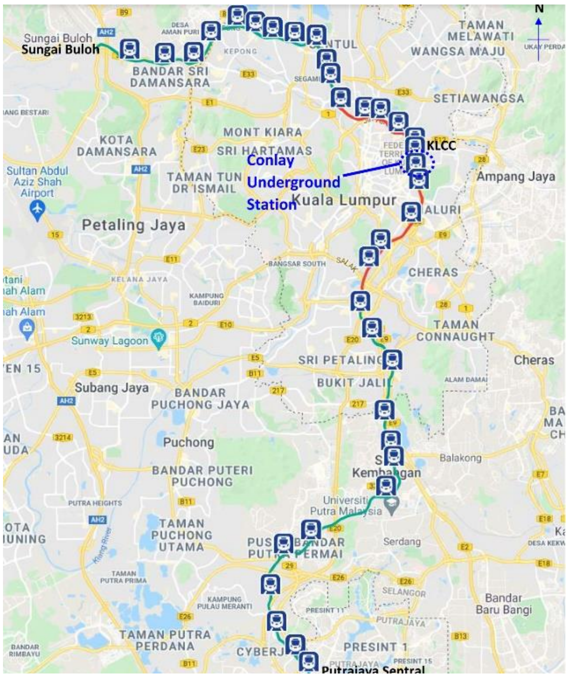

Figure 2. Location of the construction site

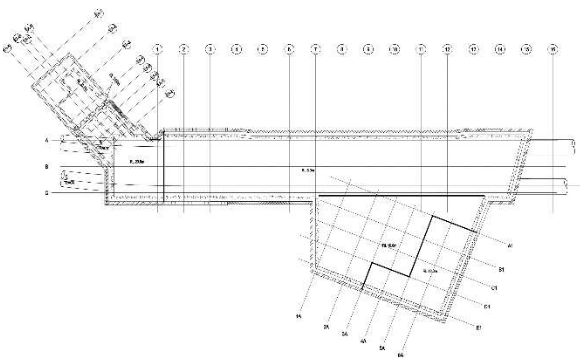
Figure 3. Construction layout plan of CLYS

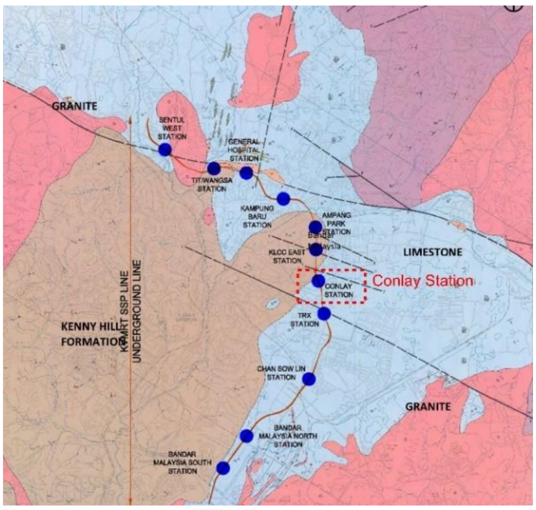

Figure 4. Geological Map of Kuala Lumpur

3 SUBSURFACE INVESTIGATION

The overburden soils above Kuala Lumpur limestone are mainly silty SAND and overburden thickness could vary significantly due to the irregular topography of the limestone bedrock. From the available borehole data and rock probing results, it was noticed that the depth of bedrock at CLYS varies from about 25mbgl to 38mbgl with localised deep limestone across the station footprint. Disturbed and undisturbed soil samples were collected for visual inspection and laboratory testing. Pressuremeter tests and field permeability tests were also carried out on the field in various depths to determine the elastic modulus and subsoil hydraulic conductivity. The highest groundwater table was recorded at 1m below existing ground level. The interpreted geotechnical parameters are tabulated in Table 1.

For the limestone bedrock, an in-situ field test had been carried out and rock core samples had been recovered during the subsurface investigation stage. Properties of rock had been assessed based on the following in-situ and laboratory test results: a. in-situ tests: Rock Quality Designation (RQD) and Packer (Lugeon) test; b. Laboratory tests: Uniaxial Compressive Test (UCT) with stress-strain measurement and Point Load test (PLT). Lugeon tests were carried out to obtain the hydraulic conductivity of rock due to fracturing within the bedrock. Meanwhile, the point load test is an index test for the strength classification of rock samples. Vertical and diametrical point load strength index, $I_{s(50)}$ can be correlated to UCT and $I_{s(50)}$ based on average results are UCS = $17(I_{s(50)})$ and UCS = $16(I_{s(50)})$ for vertical and diametrical directions, respectively.

Table 1. Interpreted geotechnical parameters

	Overburden	Bedrock
Material	Alluvium	Limestone
Depth	Varies; deepest recorded at 37.2m	Varies
Unit weight	19 kN/m^3	24 kN/m^3
RQD	-	0 – 100%
SPT-N	<10 ; average = 6	-
Average UCS	-	50 MPa
Effective shear strength	c' = 1kPa ; ϕ' = 29°	c' = 400kPa ; ϕ' = 32°
Elastic modulus, E'	18,000 kPa	1,000,000 kPa
Hydraulic conductivity, k	1x10^{-6} m/s	0 – 10 x 10^{-7} m/s

4 DESIGN FOR EXCAVATION WORKS

Conlay station is an irregular-shaped station comprising station box and entrances as shown in Figure 3. For the construction of the underground station, excavation from the existing ground level to the soffit of the base slab level is essential to facilitate the construction platform for the underground station. The excavation works involved the removal of overburden subsoil and vertical cut rock excavation down to the required final excavation level (FEL). The vertical cut rock slope is about 1m offset from the retaining wall face to minimize the excavation footprint and also to save time and cost for the construction works. The exposed vertical rock slope will be mapped by a qualified geologist and kinematic analysis will be carried out to determine the necessity of rock slope strengthening works by rock bolt reinforcement before proceeding to further excavation down to the FEL.

4.1 *Earth Retaining System Structure*

The earth retaining walls are designed as part of the temporary supporting structure to facilitate excavation to FEL. Secant bored pile (SBP) wall was selected as earth retaining wall for Conlay Station. The benefits of the SBP wall are (a.) Ability to vary pile length to suit the karstic bedrock condition where drastic rock head changes may occur during the construction stage. (b.) Ability to control water tightness to prevent water drawdown. Secant piles of 1480mm and 1180mm in diameter were designed to overlap 200mm to 370mm. The extents of overlapping of the secant piles are governed by construction tolerance such as verticality, deviation and pile length (CIRIA C580, 2003). SBP wall comprises the primary and secondary pile where the primary pile is cast first with concrete strength class of C16/20 without reinforcement and subsequently cut by secondary pile with concrete strength class of C32/40 with reinforcement. Figure 5 demonstrates the typical arrangement of the SBP wall.

Due to the deep excavation and irregular shape of the Conlay station, temporary support such as temporary ground anchor and temporary strutting system were selected for the different sections in the station box. The temporary ground anchor was designed for the area with sufficient space for ground anchor installation within the site boundary; whereas the temporary strut was designed for the remaining area. U-loop ground anchor was utilized as the temporary ground anchor to comply with the requirements of removal after construction. The anchor consists of multiple pairs of strands with different unit lengths. The reduction factor due to bending of the strand at turning point of 0.65 was adopted in the design stage based on the proofing test carried out during the KVMRT line 1 construction work with identical geological formation. The proofing tests were carried out to verify the reduction factor upon commencement of the construction stage. Summary of the anchor properties are shown in Table 2.

Laced struts system had been selected as the temporary strutting system for the excavation works by considering the depth and width of excavation required for CLYS. Loads considered in temporary steel strut design are summarised in Table 3. Eccentricity in the transfer of load from the waler to the strut had been considered for the design of struts. For walers made from a single section universal column (UC) or universal beam (UB), the eccentricity was taken as not less than 10% of the overall dimension of the strut in the vertical plane. Where the walers are constructed from twin beams the eccentricity in the vertical plane was half the distance between the webs of the two beams. The schematic of excavation works supported with the temporary strutting is as shown in Figure 6.

The analysis of the retaining wall was carried out with finite element modelling software, namely PLAXIS 2D. SBP had been modelled as a plate element in the analyses for the design of reinforcements for the SBP wall. Multiple models had been analysed to cater for the different depths of bedrock encountered at site. Analyses outcomes such as wall displacement, bending moment, and shear force were adopted for the structural design of the retaining wall. Load factor of 1.4 was applied on the obtained bending moment and shear force during the pile reinforcement design. From the design, it was observed that pile reinforcement ranges from 0.5% to 4% of pile cross-sectional area. The requirements of the reinforcement are dependent on rock head level. Deeper rock head require higher percentile reinforcement. The modelling input parameters and criteria for the PLAXIS modelling are presented in Table 4. Table 5 summarises the analysis outcomes of the retaining wall modelling based different types of support and depth of bedrock. It was observed that the wall lateral deflection ranges from 0.1% to 0.45% of the retained height which complies with requirement in project specification of 0.5%.

All secant bored piles were founded on the bedrock with rock socket lengths ranging from 1.5m to 5.5m. The termination criteria of rock sockets are based on continuous coring in competent bedrock with point load index strength $_{Is(50)}$>3 MPa. The point load index strength of 3MPa was determined based on the Author's past experiences with the mean value of strength measured from the limestone rock samples in Kuala Lumpur vicinity. The rock coring length shall be measured from the flattened horizontal bedrock surface. Rock socket length of the secondary pile shall be measured from 0.25 times of the design rock socket length of the secondary pile above the deeper rock level of adjacent primary piles or the rock level encountered when installing secondary pile as shown in Figure 7. A row of tie-back rock bolts was installed above the bedrock level to enhance the wall toe stability. Wall toe stability had been assessed based on BS8002:1994 with some modification on the replacement of passive resistance from subsoil by support from tie-back rock bolt to achieve a minimum safety factor of 1.2. For retaining wall supported with temporary ground anchor, additional check had been carried out to ensure the adequacy of socket length against resultant vertical loading due to ground anchor working load.

Table 2. Ground anchor properties

Descriptions	Properties
Working loads (kN)	212; 424; 636; 848; 1060
No. of strand	2; 4; 6; 8; 10
Strand diameter (mm)	15.24
Breaking load (kN)	260.7
Factor of safety	1.6
Strand U-loop radius (mm)	47.5
Reduction factor	0.65
Drillhole diameter (mm)	175
Allowable bond stress (kPa)	400
Free length	Varies
Bond length (m)	3; 3; 4.5; 6; 7.5

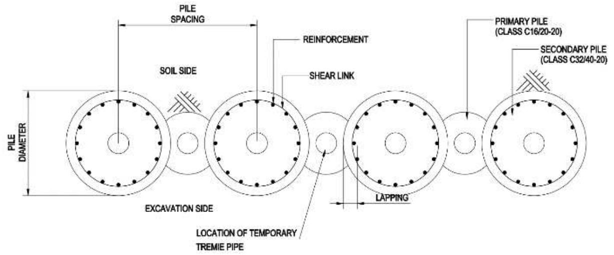

Figure 5. Typical Arrangement of secant bored pile wall

Table 3: Loadings considered in temporary steel strut design

Loading Type	Descriptions	Remarks
Axial load	Strut load	Excavation load includes earth, water and surcharge load from FEA
Bending about major axis of strut	Bending due to self-weight	Self-weight of steel struts
	Bending due to imposed load	Imposed load = 1.5kN/m
	Bending due to eccentricity	As discussed in Section 4.1
	Bending due to accidental load	50kN*
Bending about minor axis of strut	Bending due to accidental load	50kN*

Notes: FEA – Finite Element Analysis, *Accidental load is considered only one at a time in any one direction, i.e., the design accidental load causing bending about the major and minor axes do not co-exist.

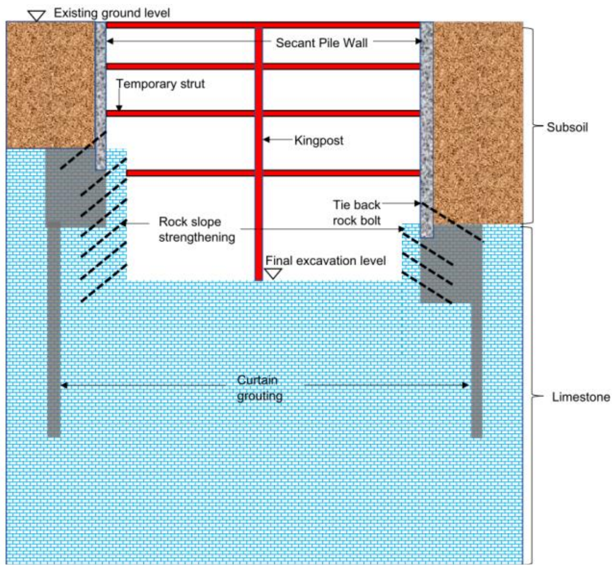

Figure 6. Schematic of excavation works supported with temporary strut

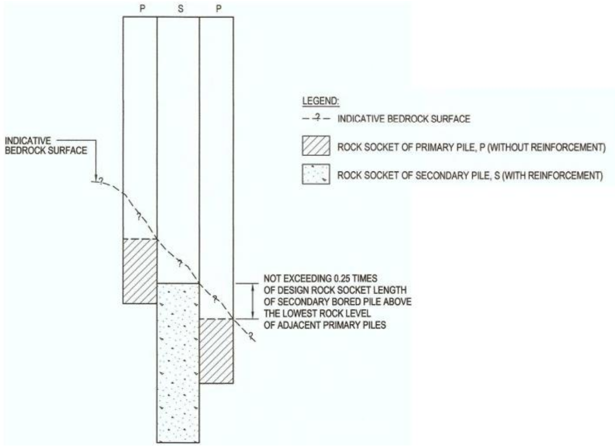

Figure 7. Typical elevation for rock socket requirements

4.2 *Rock slope strengthening works*

On-site rock mapping was carried out upon exposure of the rock surface. Kinematic analyses of rock slopes such as planar, toppling and wedge failure checking were carried out based on the rock mapping results to determine the necessity to install rock bolts as strengthening on-site. Subsequently, shotcrete was applied on the rock surface and the process repeated until FEL. If rock bolts are required within the clearance envelope of Tunnel boring Machine (TBM) drive, glass fibre reinforced polymer (GFRP) rock bolts will be used to create a "soft-eye" in the rock for TBMs to bore through. Similarly, if the rock head at launching headwall is low such that SBPs encroach into the clearance envelope of the TBM drives, GFRP reinforcement will be used in the infringing SBPs to create "soft-eye". The GFRP reinforcement in SBP is designed based on ACI440.

Table 4. Modelling input parameters and criteria

Descriptions	Modelling Input
Model Type	Plane strain analysis
Soil Model	Hardening soil model
Soil shear strength	Effective stress parameters
Soil material type	Drained
Soil loading stiffness	18,000kPa
Soil unloading stiffness	54,000kPa
Soil/wall interface factor	0.8
Wall element	Plate element
Wall bending stiffness	0.7 x EI (1)
Wall compression stiffness	0.7 x EA(2)
Construction surcharge	20 kPa
Groundwater condition	Phreatic

Notes: (1)(2) CIRIA 2003

Table 5. Summary of analysis outcomes.

Wall arrangement[#]	Estimated bedrock level[##]	Supporting system	Support layers	Deflection *
1180mm/ 880mm	10m – 30m	Ground anchor	3 -13	0.18% - 0.45%
1180mm/ 880mm	14m – 30m	Strutting	3 -5	0.14% - 0.44%
1480mm/ 1180mm	12.5m – 38m	Strutting	5 - 10	0.1% - 0.35%

Notes: *Reported deflection indicates the ratio of maximum wall deflection over the wall retained height in percentile.
# Denotes primary/secondary pile's diameter
denotes depth of bedrock below existing ground level

4.3 *Grouting Works*

Groundwater control is one of the vital elements for an excavation project. Uncontrolled groundwater drawdown could cause unwanted building settlement, which eventually lead to costly remedial works. Therefore, grouting works had been specified to reduce the rate of groundwater inflow into the excavation pit. Rock fissure grouting was carried out along the perimeter of the excavation area to form a curtain grouting as demonstrated in Figure 6. Fissure grouting involves injecting grout into the existing pathway, fissures, cavities and discontinuities within the bedrock. The grouting works consist of primary/ secondary/ tertiary grouting points set out in the spacing of 4m/2m/1m as illustrated in Figure 8 to cater for different scenarios. For the scenario where grout intake in the primary grouting point was more than 5m^3, then secondary grouting was in place after 24 hours to allow the setting of the grout. The same procedure was applied for the tertiary point when grout intake of the secondary point was more than 5m^3.

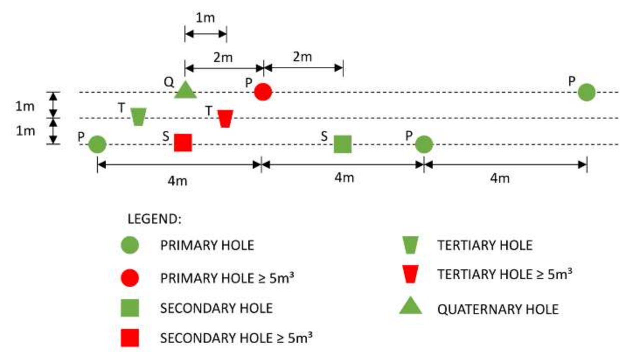

Figure 8. Fissure grouting pattern on site

5 ACHIEVEMENTS

Commencement of station box excavations started in early 2017. With continuous support from the design engineer and proper geotechnical input during the construction stage, the excavation of the station box was able to reach the final excavation level in April 2019. The recorded maximum wall deflection ratio was 0.18% which was within the project specification as well as the analysis outcomes. Vertical rock excavation upon reaching the bedrock level has resulted in considerable time and cost-saving compared with non-vertical rock excavation which will incur additional cost and time as well as challenges such as land acquisition caused by insufficient land.

With the proper planning, costly failure and delay of deliverables such as construction drawings and decision making for the unforeseen bedrock conditions in limestone formation were prevented. Also, it is vital to have continuous feedback from the site team on any anticipated issues and technical problems faced and close coordination with the geotechnical engineers to solve the construction issues without delaying the site progress and also to execute the project safely. Such model of cooperation between the geotechnical engineers and construction site team has proven as the main contribution to successfully implementing and executing the project as show in Figure 9.

Figure 9. Conlay Station at final excavation level (Courtesy of MMC-Gamuda)

6 CONCLUSIONS

Secant bored pile (SBP) wall supported with temporary support i.e., temporary strutting / ground anchors and rock strengthening were successfully implemented for underground station excavation works. The secant bored pile wall with improved water retaining capability in bedrock by grouting method prevented excessive groundwater drawdown and ground settlement. Overall, the proposed earth retaining system structure had performed satisfactorily and achieved the design requirements successfully.

7 ACKNOWLEDGEMENTS

The Author would like to express gratitude to G&P Geotechnics Sdn Bhd design team members and project team of MMC-Gamuda for the supports and efforts contributed for successfully implementing and executing the project.

8 REFERENCES

ACI 440.1R (2015). Guide for the design and construction of structural concrete reinforced with fiber-reinforced polymer (FRP) bar. *American Concrete Institute.*

BS 8002:1994. *British Standard Code of Practice for Earth Retaining Wall.*

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

Gue, S.S. (1999) "Foundations in Limestone Areas of Peninsular Malaysia", *Conference on Civil & Environmental Engineering – New Frontiers & Challenges, Bangkok, Thailand*

Muhinder S. (1999) "Limestone Profile along LRT System Two, Kuala Lumpur" *IEM/GSM-Forum "Karst: Geology & Engineering" August, 1999. The Institution of Engineers, Malaysia*.

Neoh, Cheng Aik (1997) "Design and Construction of Micropiles in Limestone Formation – Case Histories" *Short Course on Geotechnical Engineering, the Institution of Engineers, Malaysia and JKR Perak*

Ting, W. H. (1974) "Foundation of a 17-Storey Building on a Pinnacled Limestone Formation in Kuala Lumpur" *Proceedings of the Conference on Tall Buildings, Kuala Lumpur*

Yeap, E. B. (1985) "Irregular Topography of the Subsurface Carbonate Bedrock in the Kuala Lumpur Area" *Proceedings of Eighth Southeast Asian Geotechnical Conference, Kuala Lumpur, Malaysia*