Substructure Constructions for Multi-phases Highrise Development in Kuala Lumpur City Centre, Malaysia

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ABSTRACT

There is a mixed development consisting of office towers, serviced apartments and hotels in Kuala Lumpur City Centre (KLCC) area with a distance of around 700m from the Petronas Twin Towers. The challenges for substructure construction for this highrise development include its close proximity to the existing buildings where one of the neighbouring highrise towers is on raft foundation that requires stringent control on groundwater lowering, complicated underlying geological formations, interfacing issues between phases and controlled working hours with tight construction schedule. The Phase A of the project is underlain with meta-sedimentary Kenny Hill formation while Phase B of the project is located in the contact boundary of Kenny Hill and Kuala Lumpur Limestone formations. The most challenging geological formation is at project site for the other two adjacent phases where the limestone bedrock varies significantly from 20m to 111m deep with a layer of soft peaty clay varying from 18m to 60m deep before encountering Kenny Hill formation and Limestone bedrock at deeper depth. Interfacing walls have been designed and constructed to cater for the different depth of excavations for different phases. This paper will discuss the design and construction considerations for each of Phases A, B, C and D by taking into account the challenges mentioned above. The instrumentation monitoring results of the test piles and excavation system for the completed Phases A and B will also be presented as assessments on the performance of the selected foundation and basement walling systems.

INTRODUCTION

This multi-phases highrise development is an integrated mixed development located within the prominent area of KLCC, Malaysia. It comprises of multiple development components consisting of residential, offices, retail and hotel which are all systematically tied together with a 1.5 acre private park on a total of 9.1 acres land.

The project is developed in five phases where the demarcation of phasing and general view of basement levels is illustrated in Figures 1 and 2 respectively. Most of the existing buildings adjacent to the project site are on piled foundation except the 45-storey high luxury condominium Z was built on raft foundation.

Fig.1. Demarcation of Phasing.







GENERAL GEOLOGY AND SUBSURFACE CONDITION

By overlaying the geological map of Selangor onto Google map, it was found that the project site is underlain by Kenny Hill Formation as shown in Figure 3. However the subsurface investigation revealed that the site is located at the contact boundary of two different formations, where the meta-sedimentary Kenny Hill Formation overlying Kuala Lumpur Limestone of earlier age which is considered a typical geological formation in Kuala Lumpur as illustrated in Figure 4.



Fig.3. Superimposed geological map and Google map.





The age of Kenny Hill Formation is younger than Kuala Lumpur Limestone. Generally, Kenny Hill Formation was formed during Carboniferous to Triassic age and overlying the Kuala Lumpur Limestone. Kenny Hill Formation consists of meta-sedimentary rocks with interbedded meta-arenite and meta-argillite and occasionally some quartzite and phyllite. Due to intense weathering process of tropical climate, the meta-sedimentary rocks have already been transformed into residual and completely weathered soils (Grades V and VI). The overburden materials generally consist of sandy/gravelly SILT and occasionally of clayey SAND.

Kuala Lumpur Limestone is believed to be formed during Middle to Upper Silurian age. Due to the inherent karstic features of limestone bedrock, depth of the limestone bedrock is highly irregular. The overburden soils immediately above Kuala Lumpur Limestone are mainly alluvial and superficial deposits, predominantly medium to coarse silty Sand and clayey SILT with mixture of gravel size materials. The thickness of overburden soils varies significantly due to irregular topography of the limestone bedrock. Other than the overburden materials discussed above, very soft and highly permeable peaty deposits were encountered at localised area along the northern boundary in Phase D. The depth of bedrock varies significantly from 20m to 111m below ground level (b.g.l.). The limestone bedrock is fairly erratic.

The interpreted bedrock contour generated from the interpolation of all available boreholes is shown in Figure 5. Three dimensional (3-D) models of subsoil profiles are presented in Figure 6.

Generally, the subsoil strata for the project site can be clustered into three major areas:

- 1) Deep soft deposit overlying limestone
 - a) Encountered at the northeast side of Phase D.
 - b) The subsoil generally consists of 18m to 60m thick alluvium materials with high liquid limit before encountering Kenny Hill formation and limestone bedrock at deeper depth.
 - c) Limestone bedrock was encountered at RL -30m to RL -75m (around 111m b.g.l.).
- 2) Kenny Hill overlying limestone
 - a) Encountered at Phase C, southern sides of Phases B and D.
 - b) The subsoil generally consists of 20m to 40m thick residual soils of Kenny Hill formation before encountering limestone bedrock at deeper depth.
- 3) Thick Kenny Hill formation
 - a) Encountered at the northern of Phase B and whole Phase A.

Fig.5. Interpreted bedrock contour.





Fig.6. 3-D models of subsoil profiles.

b. 3-D view of overburden soils.



The geology and subsoil conditions of project site described above formed an important basis for the designer to decide on the earth retaining and foundation system to be adopted for each phases.

BORED PILE FOUNDATION

Bored pile foundation system is proposed for all the phases considering its advantages over the precast concrete pile as listed below:

- a) less pile point with the selected big diameter bored piles and resulting in more economical pilecap design with least load bridging transfer from column loading to group piles,
- b) able to be socketed into hard stratum or bedrock to carry the pile structural carrying capacity, and
- c) able to have empty bore above the pile Cut-Off Level (C.O.L) so that material wastage is reduced and ease of basement and pilecap excavation works.

Instrumented Test Pile Results for Phases A and B

All preliminary test piles for the completed Phases A and B were instrumented with proprietary Global Strain Extensometer technology (Glostrext method) using the access tubings attached to the reinforcement cages for sonic logging purpose as presented by Hanifah & Lee (2006). This system uses advanced retrievable pneumatically-anchored extensometers coupled with high-precision spring-loaded vibrating-wire strain gauge sensor and simple analytical technique to measure the loads transferred down the pile shaft and the toe of test piles. It is a post-installation instrument which can accurately measure the relative deformations of anchored segments across the entire pile length.

Figure 7 shows the photographs taken during the installation and set-up of kentledge system for compression pile load test. The typical sequences for installation of Glostrext sensors are as follows:

- 1) The sonic logging tubes are tied with the steel cage and lowered to the borehole prior the concreting of test pile. Sonic logging test was carried out after the concrete hardened and gained at least 70% of its design strength.
- 2) Glostrext sensors will then be lowered down to the sonic logging pipes before the stacking of kentledge blocks.

Fig.7. Set-up of instrumented test pile.



a. Lowering of steel cage with sonic logging tubes.



b.Preparation of Glostrext c.Kentledge set-up for sensors at site. compression load test.

The instrumented pile load test results for Phases A and B have been presented by Liew et at. (2010) and the coefficients of mobilised shaft friction resistance ($f_{s,mob}/N$) versus SPT-N values are plotted in Figure 8. The coefficient of mobilised shaft friction resistance reduces exponentially with increasing SPT-N values. The suggested empirical correlation by Chang and Broms (1990) and Tan, et al. (1998) for the ultimate shaft friction resistance (f_s) for residual soil, $f_s = 2 \times SPT$ -N is also plotted in Figure 8 for comparison.





The correlation of rock test results suggests a conversion factor of 13.3 as per Eq. (1) for estimating the unconfined compressive strength of limestone bedrock encountered in the project site as reported by Liew et at. (2010).

UCS =
$$13.3 I_{s(50)}$$
 (1)

where UCS = unconfined compressive strength of rock (MPa) $I_{s(50)}$ = point load index for 50mm diameter core (MPa)

Neoh (1998) recommended the maximum allowable rock socket friction for Kuala Lumpur Limestone in Malaysia shall be limited to 5% of either the rock UCS value or characteristics concrete strength of pile, whichever is smaller as per Eq. (2) below.

$$f_{s,all} = 0.05 * \{min(UCS, fcu)\}$$
 (2)

where	$f_{s,all}$	= allowable rock socket friction resistance
	f_{cu}	= concrete compressive strength at 28 days (35MPa)

Table 1 shows that the load transfer mechanism of earlier mobilisation of the upper portion of rock socket friction to much higher value has rendered relatively lower degree of socket mobilisation at the lower rock socket under the maximum test load.

For design purposes, the maximum allowable shaft friction resistance of limestone bedrock for this development is limited to 800kPa and may reduce to 500kPa depending on the quality and point load test results for the limestone samples collected during the bored pile installation at site.

$\begin{array}{c} Measured \\ f_{s,mob} \\ (kN/m^2) \end{array}$	*UCS (N/mm ²)	Ratio of f _{s,mob} / UCS	${}^{\#}\!f_{s,all}$ (kN/m ²)	Ratio of f _{s,mob} / f _{s,all}	Remarks
1670	14.18	0.12	709	2.4	Upper portion
1400	33.60	0.04	1680	0.8	Lower portion

Table 1. Summary of point load tests and mobilised rock socket frictions for Phase B test pile socketted in Kuala Lumpur Limestone.

* UCS is average UCS value estimated using Eq. (1) based on PLT results carried out on rock samples collected at every 0.5m spacing

Allowable rock shaft friction resistance is calculated using Eq. (2)

3-D Model of Foundation Design for Phase D

The limestone bedrock encountered in Phase D is as deep as 111m below the existing ground level which exceeds the reachable boring depth (~100m long) of typical bored pile equipment in Malaysia. Barrette pile that can be installed to deeper depth is not preferred as there was only very limited number of contractor can perform the work at the time of the tender, thus no advantages of getting competitive pricing. Hence, bored pile foundation with limit of installation depth (max. 100m) is finally proposed for the foundation of Phase D tower block.

Due to the existence of deep peaty soil and limit of installed pile length for foundation in Phase D, PLAXIS 3-D Foundation program was used to predict the settlement of the proposed pile group arrangement. Based on the analysis result, settlement reducer piles are required to be introduced at the northern side of Phase D in order to have a bow shape settlement as shown in Figure 9.





The differential settlement of the tower can be controlled to within 1/500 even without the settlement reducer piles, but it is important to note that the 1/500 limit is meant for stresses control and design within the structural frame of the tower only and will be magnified to large lateral top displacement of the tower if the settlement profile at the base of the tower is not controlled.

For instance, the tower height for the project is averagely 200m in height which will be translated to 400mm top displacement if the base of the tower is tilted to one side with a distortion angle of 1/500.

EARTH RETAINING SYSTEM

Based on the requirements of vertical facing, retaining height, ease of construction, space constraint, etc., diaphragm wall and contiguous bored pile (CBP) wall have been adopted as earth retaining system for basement excavations for this development. The deepest excavations within the lift pit and core wall foundation below the lowest basement level were either carried out with open-cut method or with the additional temporary CBP cofferdam.

To be successful in today's competitive consultancy service industry, engineers must deliver design on time and within budget. The observational method discussed by Peck (1969) is one of the common practice in Geotechnical Engineering to achieve economical design provided the design can be modified and improved as construction progresses with careful timely review of monitoring results.

Instrumentation Monitoring

Predictions with finite element analyses, at best, if not tailored with construction monitoring scheme or observational methods, are simply an educated guess only based on design assumptions and expectations derived from the engineer's knowledge of his/her proposed construction methodology and his/her understanding on the interpreted subsoil conditions.

An instrumentation monitoring programme is proposed for all the phases to monitor various parameters that may affect the proposed retaining system during actual work execution. All the instruments will be installed before the commencement of excavation works. The proposed instruments to assess the performance of the excavation works include the following:

- a) Inclinometer to monitor the lateral deflection profile of the walls and also the retained ground.
- b) Observation well / standpipe to monitor the changes of groundwater level.
- c) Ground settlement markers to monitor the ground settlement at the retained ground.
- d) Building settlement markers to monitor the settlement of adjacent buildings.

- e) Displacement markers to monitor the three directional movements of the ground/slab at the retained ground.
- f) Tiltmeter to monitor the tilting of adjacent buildings.

Figure 10 summarises the maximum wall deflections and ground deformations measured during the excavations for Phases A and B. The maximum recorded ground settlement and wall deflection is 45mm and 64mm, respectively. The recorded wall deflections for excavation of Phases A and B are well within the 0.5% of wall retained height implying the successful performance of the adopted earth retaining systems.



Fig.10. Summary of wall deflections and ground deformations.



Contingency Plan

The stability of the excavation is affected by numerous external and internal factors with uncertainty. Uncertainty bring in an element of risk in geotechnical engineering. Risk, in general, signifies situations where the actual outcome of an activity or event is likely to deviate from the predictions. Risk analysis in geotechnical engineering is an inevitable process of dealing with risks and uncertainties in a structured manner.

Hence, it is imperative to have proper contingency plan ready at the onset of the project to allow timely review on the performance of the retaining wall system so that necessary modification or even early rectification (if required) work can be in-placed before reaching to a point of no return and subsequent occurrence of unfortunate incident. Table 2 lists the typical contingency plan that was implemented in excavation works for the project.

Zone	Action Plan
	1. Increase the monitoring frequency to 3 times a week.
Ambar zona	2. Visual inspection and evaluation of the stability of the adjacent structures.
Allider Zolle	3. Back analysis, if required.
	4. Review the construction sequences, if necessary.
	1. Stop excavation works at the affected area.
	2. Backfill, if necessary at the affected area.
	3. Daily monitoring of the instruments.
Red zone	4. Visual inspection and evaluation of the stability of the adjacent structures.
	5. Back analysis.
	6. Review the construction sequences.
	7. Install additional props to improve the stability of retaining system.

Table 2. Contigency Plan.

Notes:

- 1. The threshold limits for Amber zone is 80% to 100% of the design predictions.
- 2. The threshold limits for Red zone is 100% to 120% of the design predictions or the serviceability limits of the adjacent buildings, whichever is stringent.

The threshold limits for Red zone is subjected to change based on the risk assessments carried out to study the effects of the proposed deep excavation works to the surrounding structures and it should be related to the serviceability limits of the adjacent existing buildings.

Excavation system for Phase A

Substructure construction for Phase A was taken around 17 months to complete. In view of Phase A being the first construction phase in this development and only consisting of two (2) level basements, 600mm thick diaphragm wall supported with inclined struts and open cut at two sides of the excavation as shown in Figures 11 and 12 had been successfully used to facilitate the basement excavation.



Fig.11. Temporary work layout for Phase A excavation.

Fig.12. Overview of project site for Phase A excavation.



Excavation system for Phase B

Substructure construction for Phase B was started six (6) months before the completion of substructure for Phase A. The lowest basement of Phase B is deeper than the completed Phase A basement and hence requiring interfacing wall to be installed at the interface of Phases A and B. The confirmation of the different in basement levels between Phases A and B was done during the early stage of substructure construction of

Phase A and hence permitting the installation of this interfacing wall under Phase A substructure package as variation work.

As shown in Figure 11, a row of contiguous bored piles was installed at the southern side of Phase A which was located along the toe of the cut slope. If this interfacing CBP wall was not installed during the Phase A substructure construction, it would be more costly and loss in basement space would be expected due to the required set-back of working space for installation of such interfacing wall abutting the completed Phase A substructure.

Alternative Design by Contractor

The original tender design for Phase B excavation was the diaphragm wall supported with two (2) layers of steel strutting at three sides and with open cut into the Phase C site at the other side. The excavation work for Phase C that was initially targeted to be completed concurrently with the excavation work for Phase B was postponed due to the re-design of Phases C and D.

Since the open cut excavation into Phase C site was no longer practical as it may also impose more constraints to the future basement excavation works and subject to the redesign of Phases C and D, the Contractor awarded with the substructure contract for Phase B had proposed the alternative 800mm thick diaphragm wall supported with E-shape floor slab with semi top-down method as shown in Figure 13.

Generally the alternative diaphragm walls with E-shape floor slab designed as lateral shoring support by the Contractor are shorter than the compliance design. In order to ensure the stability of the wall toe and control the lowering of groundwater table to acceptable limits, a contingency plan as discussed in previous section was implemented to safeguard the excavation works.



Fig.13. Overview of project site for Phase C excavation.

Finite Element Analysis

Two dimensional (2-D) plane strain finite element (FE) analyses using computer software "PLAXIS" were carried out by the Contractor to simulate the top-down excavation sequences for the alternative E-shape floor slab in order to estimate the ground deformations around the excavation and the performance of the retaining wall.

In the original submission, the Contractor ignored the fact that the nature of this excavation problem is actually a 3-D problem instead of typical excavation problem that can be easily simplified to 2-D half space problem with certain analysis assumptions made. Due to the unbalanced forces from each side of the excavation, the Wall Type 2C has potential of moving inwards to the retained ground as presented in Figure 14.

Fig.14. Schematic diagram of wall deflections under unbalance forces (Full model submitted by the Contractor prior the wall installation).



Thus, the Contractor was requested to carry out full model (with section cut across two walls) instead of half space model and with iterative process as discussed below in order to capture the behaviour of the diaphragm walls supported by E-shape floor slab.

The unbalanced slab force can be modelled either using the point load or applying the force through the "fixed-end" dummy anchor with negligible axial stiffness (to avoid double counting of propping stiffness) in PLAXIS program.

The responses of Wall Type 2C for half space model and full model with modelling of unbalance forces are presented in Figure 15 for comparison. It is worth to note that the wall deflection profiles predicted in half space model is different from the one predicted in full model. The incorrect prediction of wall responses for Wall Type 2C using half space model without taking into account the unbalanced force in E-shape floor slab will eventually lead to under-design of Wall Type 2C and thus potentially endangering the excavation works as the threshold limit for wall deflection will be wrongly set based on

inaccurate predictions of wall behaviour. Fortunately the need of remodelling the affected walls had been acknowledged by the Contractor after few rounds of technical discussions before the wall installation.



Fig.15. Responses of Wall Type 2C.

3-D Excavation Model for E-shape Floor Slab with Semi Top-Down Construction

3-D analysis using PLAXIS 3-D foundation program was carried out in order to understand the load distributions for the E-shape floor slab during excavation works. It was observed that there are concentrated high tensile stresses at corners of the floor slab due to differential movements of diaphragm wall that connected to E-shape floor slab as shown in Figure 16.

It is imperative to provide sufficient tension rebars in high tension zones as illustrated in Figure 17 to prevent potential tension cracks for the similar excavation works that subject to unbalanced forces during the excavation.





Fig.17. Indicative arrangement of tension bars to be provided in high tensile stress zones.



Back Analysis

3-D model set up for excavation works of Phase B had allowed the project team in understanding the behaviour of the E-shape floor slab in a semi top-down excavation and the distribution of loads in floor slab. However, the best fitting of the computed wall deflection profiles to the field monitoring data for 3-D model was not successfully done. Hence, the back analysis to best fit the wall deflection profiles was carried out with 2-D program with simplification to capture the unbalanced movement of walls at two sides.

Figure 18 shows the FE model and wall deflection profiles for back analysis of Wall Type 1. The computed wall deflections tally well with the field measurements except for pilecap excavation stage where 2-D model tends to over-predict the wall deflection for localised excavation which is well known as 3-D effects that reported by Yong (1996).

Table 3 shows the adopted soil properties in back analysis that are similar to the original design assumptions. In the back analysis of Wall Type 1, only half space model was set up but with the simulation of unbalanced movement of 10mm and adjustments of groundwater table according to the actual monitoring data.



Fig.18. FE model and wall deflection profiles for back analysis of Wall Type 1.

Table 3. Soil properties adopted in the back analysis of Wall Type 1.

Soil Layer	SPT-N	Effective Angle of Friction, φ' (°)	Effective Cohesion, c' (kPa)	Effective Young's Modulus, E' (kPa)	Unloading/ Reloading Stiffness, E' _{ur} (kPa)
Layers 1 and 2	$N \leq 10$	30	1		
Layers 3 and 4	$10 < N \leq 40$	30	3	2000N	6000N
Layer 5	N > 50	32	5		

The unbalanced movement of 10mm was determined based on the difference between the registered wall top movements in inclinometer I8 installed inside the Wall Type 2C during the earlier cantilever excavation stage and when excavation reached final formation. The wall top movement of inclinometer I8 during cantilever stage is 35mm and reduced to 25mm when excavation reached final formation.

The monitoring results of inclinometer I8 confirmed the tendency of Wall Type 2C moved inwards to the retained ground which is similar to the responses of wall deflection predicted by full model with modelling of unbalanced forces as discussed in previous section but with the magnitude of wall movements larger than the predictions. The back analysis for Wall Type 2C shall be carried out using 3-D program as it is real 3-D problem which can be simplified into 2-D model for design purposes only and not so appropriate for back analysis purposes as too many assumptions need to be made for boundary conditions in 2-D model that may be ended up with force fitting rather than best fitting.

Interfacing Wall for Phases B and C

The change in basement levels of Phase C from the original 3 level basements to 4 level basements was only confirmed after the diaphragm walls at the interface of Phases B and C were constructed. This design change had increased the construction cost where

additional retaining walls are to be installed abutting the previous constructed diaphragm wall in order to carry out deeper excavation for additional basement in Phase C.

Few alternative retaining wall schemes were proposed to the owner during the design stage for Phase C and the final decision was to use the cantilever retaining wall as interfacing wall between Phases C and D in order to maximise the footprint of the basement 4 as detailed in Option 2 of Figure 19.





Temporary Measures

As a precaution measure to prevent the potential lowering of groundwater table, recharge wells are proposed to maintain the groundwater level to a design level during the basement excavation which is the similar approach adopted in the excavation works for Phases A and B. This can be carried out by sinking boreholes at close interval around the site perimeter boundary and injecting clean water into the borehole to recharge the loss of groundwater behind the retaining wall via seepage towards excavation. Figure 20 shows the typical details of gravity recharge well.

Fig.20. Typical details of gravity recharge well.



The successful of excavation works in the completed Phases A and B clearly shows that the installation of gravity recharge well with automated discharging valve can minimise the effects of groundwater lowering for deep excavation in high permeable residual soils of Kenny Hill formation.

CONCLUSIONS

The following summary and conclusions were based on the above discussed sections:

- 1. Understanding on the geological formation and subsoil conditions of the project site is important to yield economical geotechnical design and identify the constraints for the selected foundation and earth retaining system.
- 2. Settlement reducer piles are introduced for the tower block foundation in Phase D in order to control the settlement profile at the foundation level to within 1/500 and with "bow" shape settlement profile to minimise the lateral displacement at top of the tower and also control the internal induced stress within tower structures.
- 3. User-defined iterative technique is used in 2-D model of E-shape shoring floor slab in semi top-down excavation to approximate the unbalanced forces at two sides of walls for more accurate prediction on the wall responses. The monitoring results confirmed the deflection profile of the walls and tendency of one side of wall moving away from the excavation side under the unbalanced force in E-shape floor slab shoring system.
- 4. All excavation works are 3-D problem and approximation into 2-D models shall be done with good engineering judgment to avoid under-design of wall elements.
- 5. 3-D program is useful to help the designer understand the real behaviour of the design elements but its use in practical engineering design is still very limited and benchmark of the design parameters are required to be established for gaining the confidence on using 3-D program to achieve economical design.
- 6. Effective monitoring scheme with early warning features allows the engineer to gather information so that the timely review on the performance of the proposed system can be carried out and the design modification or improvement deviated from original design can be timely performed whenever necessary for assurance of safety.
- 7. Early confirmation on the basement levels for buildings within same development is important to avoid costly solution for interfacing wall system.
- 8. Gravity recharge wells performed well as a contingency measure to prevent excessive lowering of groundwater level resulted from the excavations in high permeable residual soils of Kenny Hill formation.

ACKNOWLEDGEMENTS

The Authors would like to thank G&P project team members, Ir. Koo Chee Min, Ir. Tan Sje Ting and Engr. Chong Chen Way for their help and input to compile the relevant materials to complete the paper. The support and sponsorship from the Institution of Engineers Malaysia (IEM) to attend this Tripartite Seminar are also gratefully acknowledged.

REFERENCES

Chang, M.F. and Broms, B.B. (1990). "Design of Bored Piles in Residual Soils based on Fieldperformance Data". *Canadian Geotechnical Journal*, Vol. 28, p.200-209.

Hanifah, A. A. and Lee, S. K., Application of Global Strain Extensometer (Glostrext) Method for Instrumented Bored Piles in Malaysia, 10th International Conference on Piling and Deep Foundations, Amsterdam, June 2006.

Liew, S.S., Khoo, C.M., Tan, S.T. and Loh, Y.E. (2010). "Review of Load Test Performance of Bored Pile Foundation in Weathered Meta-Sedimentary Formation and Kuala Lumpur Limestone". *The 17th Southeast Asian Geotechnical Conference*, Taipei, Taiwan, May 10-13, 2010.

Minerals and Geoscience Department of Malaysia (2010). "Geological Map of Selangor, Kuala Lumpur, Malaysia". Sheet 94.

Neoh, C. A., "Design & Construction of Pile Foundation in Limestone Formation", *Journal of Institution of Engineers, Malaysia*, Vol. 59, No. 1, 1998, pp.23-29.

Peck, R.B. (1969). "Advantages and limitations of the observational method in applied soil mechanics. *Geotechnique 19*, No. 2, p. 171-187.

Tan, Y.C., Chen, C. S. and Liew, S.S. (1998). "Load Transfer Behaviour of Cast-in-place Bored Piles in Tropical Residual Soils of Malaysia". *Proceedings of 13th Southeast Asian Geotechnical Conference*, Taiwan, p. 563-571.

Yeap E.B. (1986). "Irregular Topography of The Subsurface Carbonate Bedrock in The Kuala Lumpur Area, Foundation Problems in The Kuala Lumpur Areas of Peninsular Malaysia". Geot. Tech. Div., IEM.

Yong, K.Y., Lee, F.H. and Liu, K.X. (1996). Three dimensional finite element analysis of deep excavation in marine clay. *Proceedings of the 12th Southeast Asian Geotechnical Conference*, 6-10 May, 1996, Kuala Lumpur, pp. 435-441.