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Deep excavation in Kuala Lumpur limestone formation for chan sow lin station of the Malaysia Klang Valley Mass Rapid Transit SSP Line

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ABSTRACT

The deep excavation design for the SSP line KVMRT underground Chan Sow Lin station with maximum depth of 40m in Kuala Lumpur limestone formation is described in this paper. Generally, limestone rockhead level of the site is not deeper than 10m from existing ground level except some localized limestone valleys with the deepest rockhead at about 40m from the ground level. Due to highly variable rockhead profile in limestone, selection of suitable and economical temporary Earth Retaining Structure System (ERSS) that exhibits flexibility to change in rockhead profile at site but yet to maintain its functionality becomes important. Ground treatment scheme in limestone is established to minimize drawdown of groundwater during deep excavation to prevent ground settlement that could cause potential distress to adjacent buildings. The design principles of the temporary ERSS together with vertical rock excavation to final depth of the station are discussed in this paper.

Keywords: Deep Excavation; Kuala Lumpur Limestone

1 INTRODUCTION

The Sungai Buloh – Serdang – Putrajaya (SSP) line is the second line of the Klang Valley Mass Rapid Transit (KVMRT) in Malaysia and consists of 11 underground stations. Chan Sow Lin (CSL) station is one of the underground stations which is located in Kuala Lumpur Limestone formation as shown in Figure 1.

The site is underlain by alluvium which consists of silty sand or sandy silt followed by Kuala Lumpur Limestone. The limestone rockhead level of the site ranges from 2m to 10m from existing ground level (EGL) except some localized limestone valleys with the deepest rockhead at about 40m. Figure 2 shows the borehole location around the CSL station while Figure 3 shows part of the simplified borehole logs. The existing groundwater level is about 1m below EGL.

Deep excavation of 22m to 40m from EGL is supported by temporary Earth Retaining Structure System (ERSS) to retain overburden soil and vertical rock slope with necessary strengthening measures i.e. shotcrete wall and rock bolt.

2 TEMPORARY EARTH RETAINING STRUCTURE SYSTEM (ERSS)

Selection of suitable and economical temporary ERSS that exhibits flexibility to the highly variable rockhead profile in karstic Limestone but yet to maintain its functionality becomes important. Deep Soil Mixing (DSM) wall has been adopted for the area where rockhead is shallower than 10m from EGL. For area where rockhead is deeper than 10m from EGL specifically at the localized deep valley, Secant Bored Pile (SBP) wall with temporary ground anchor as temporary supporting system has been adopted. Figure 4 shows the ERSS layout plan for CSL station.

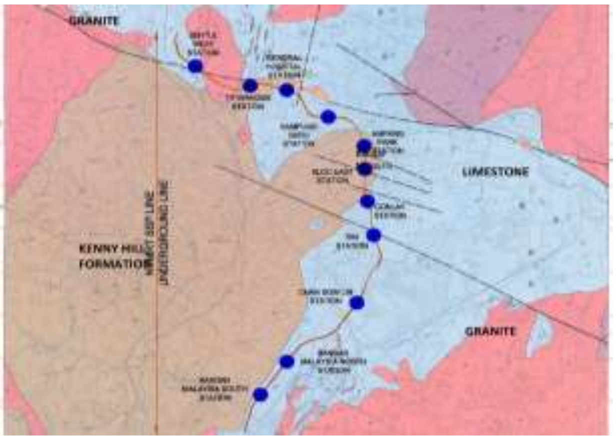

Fig. 1. Location of CSL station on Geological Map.

2.1 Deep Soil Mixing (DSM) Wall

Wet mixing method was adopted in which the cement grout with water to cement ratio of 1.0 is mixed with in-situ soil to form a DSM column. Series of DSM columns with 1.0m nominal diameter overlapped with each other to form a DSM block/wall. The thickness of DSM wall has been properly designed to ensure the weight of the wall is sufficient to achieve required externally stability against overturning and sliding. The volumetric ratio of cement grout to in-situ soil adopted is 0.5. Figure 5 shows the DSM wall details adopted for CSL station.

Fig. 2. Borehole location around the CSL station.

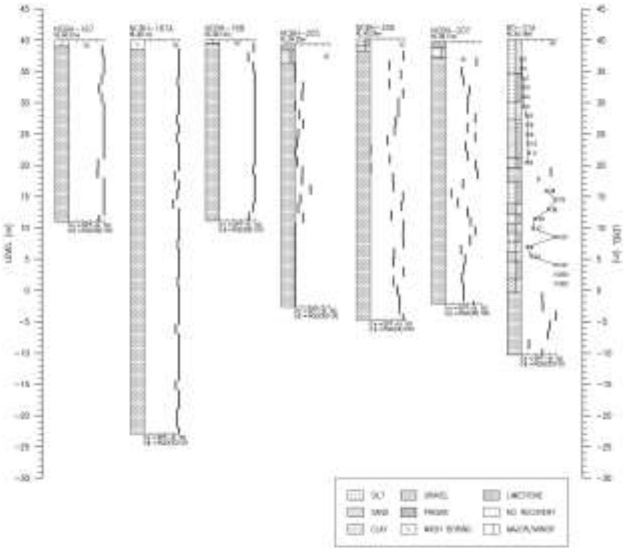

Fig. 3. Part of the Simplified Borehole Logs.

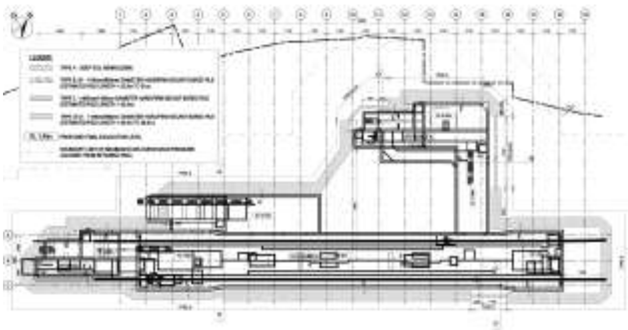

Fig. 4. ERSS Layout plan for CSL station.

Minimum unconfined compressive strength, q_u of 1MPa at 28 days has been specified for the DSM wall to ensure wall internal stability. 51 cored samples have been collected from the constructed DSM wall for unconfined compressive strength test and the results are summarized in figure 6. All the cored sample exhibit q_u value of more than 1MPa and majority of the measured q_u falls within the ranges of 2.0 to 4.0MPa which is much higher than the required q_u of 1MPa.

Fig. 5. Details of DSM Wall at CSL station.

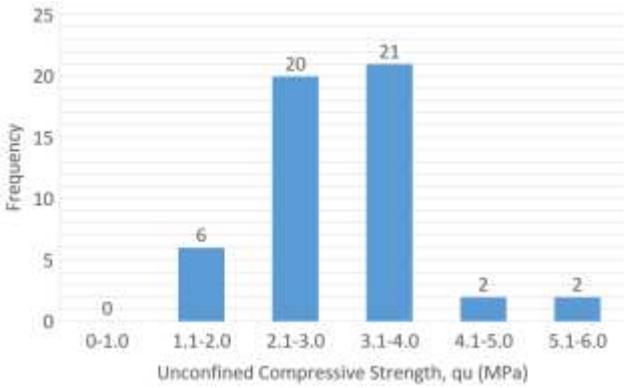

Fig. 6. Histogram of Compressive Strength of DSM wall.

Elastic modulus test has also been carried out on part of the cored samples during the unconfined compressive strength test. The secant Young's modulus of elasticity at 50%, E_{50}, of the unconfined compressive strength have been related to the unconfined compressive strength, q_u. Figure 7 shows the ratio of E_{50} to q_u is in the range of 120 to 415. The recommended ratio of E_{50} to q_u for wet mixing in similar ground conditions is 150.

Unit weight of cored samples were measured to verify the value adopted in the design. Figure 8 shows the summary of measured unit weight of cored samples. Majority of the cored samples weight between 15.1 to 19.0kN/m³ which is similar to the typical range of unit weight for alluvial soil over Kuala Lumpur Limestone. The test results are in good agreement with the FHWA-HRT-13-046 where the density of in-situ soil treated with deep soil mixing by wet method does not change significantly. For an initially saturated soil treated by wet mixing, the unit weight, γ_{mix} is given by the equation below:

$$\gamma_{mix} = \frac{\gamma_{soil} + VR\gamma_{slurry}}{1+VR} \qquad (1)$$

Where γ_{slurry} is the total unit weight of the slurry and VR is the ratio of volume of slurry to volume of in-situ soil, V_{slurry} / V_{soil}.

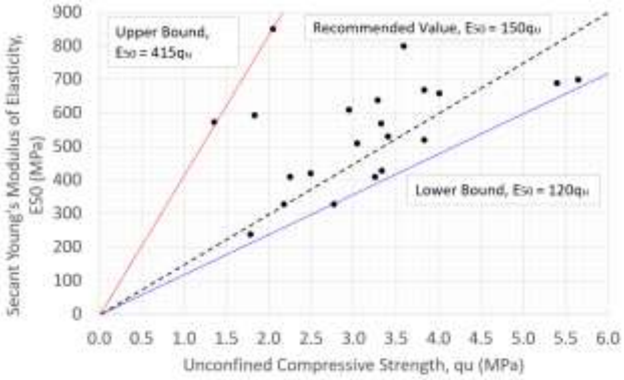

Fig. 7. Plot of E_{50} versus q_u.

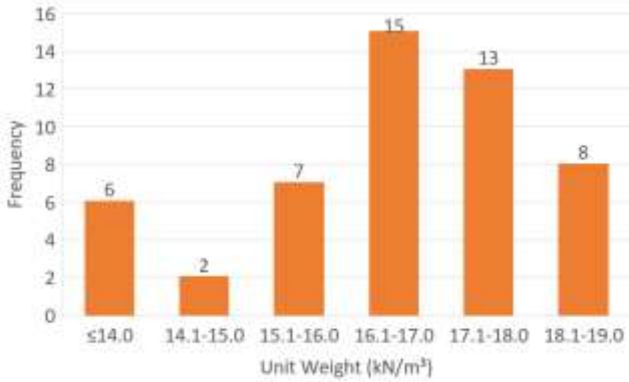

Fig. 8. Histogram of Unit Weight of DSM wall.

2.2 Secant Bored Pile (SBP) Wall

DSM wall become not viable for area with deep rockhead due to space constraint where thicker wall is required to achieve required external stability. Hence, SBP wall supported by temporary ground anchor is adopted at area where the rockhead level is deeper than 10m specifically at the deep limestone valley. SBP wall is selected due to its flexibility to socket the wall toe into bedrock by following the highly undulate rockhead profile of karstic limestone. In order to achieve sufficient toe fixity and bearing capacity, it is required to ensure piles are socketed into competent bedrock by fulfilling the following three criteria:

a. Change of tools to rock coring/excavation tools, and
b. The rock material shall be verified by carrying out point load test on at least five rock samples from not more than 0.5m layers to achieve a minimum index strength, $I_{s(50)}$ of 3MPa subject to site confirmation on typical rock lump size based on the size correction factor, F=(De/50)^0.45 where De is the equivalent core diameter in mm, and
c. Recovered rock materials of more than 70% are subjected to site calibration upon start work unless otherwise agreed by the supervising consultant.

If rock socket length of pile does not fulfill the definition of rock coring above, the socket length shall be extended by following criteria set in Table 1. Rock socket length of secondary pile shall be measured from 0.25 times the design rock socket length of secondary pile above the deeper rock level of adjacent primary piles, refer to Figure 9 for the schematic diagram of requirement of rock socket for secondary pile.

Table 1. Equivalent Rock Socket Length in relation to Index Strength, $I_{s(50)}$

MAXIMUM CONTINUOUS PILE LENGTH INTO ROCK THAT FULFILL THE DEFINITION OF ROCK CORING ($I_{s(50)}$ ≥3 MPa)	MAXIMUM CONTINUOUS PILE LENGTH INTO ROCK THAT DO NOT FULFILL THE DEFINITION OF ROCK CORING ($I_{s(50)}$ <3 MPa)	
ROCK SOCKET LENGTH AS SPECIFIED IN WALL SCHEDULE	$2.5 \le I_{s(50)} < 3.0$	EQUIVALENT ROCK SOCKET= 1.30 TIMES OF ROCK SOCKET LENGTH AS SPECIFIED IN WALL SCHEDULE
	$2.0 \le I_{s(50)} < 2.5$	EQUIVALENT ROCK SOCKET= 1.65 TIMES OF ROCK SOCKET LENGTH AS SPECIFIED IN WALL SCHEDULE
	$1.5 \le I_{s(50)} < 2.0$	EQUIVALENT ROCK SOCKET= 2.20 TIMES OF ROCK SOCKET LENGTH AS SPECIFIED IN WALL SCHEDULE
	$1.0 \le I_{s(50)} < 1.5$	EQUIVALENT ROCK SOCKET= 3.30 TIMES OF ROCK SOCKET LENGTH AS SPECIFIED IN WALL SCHEDULE

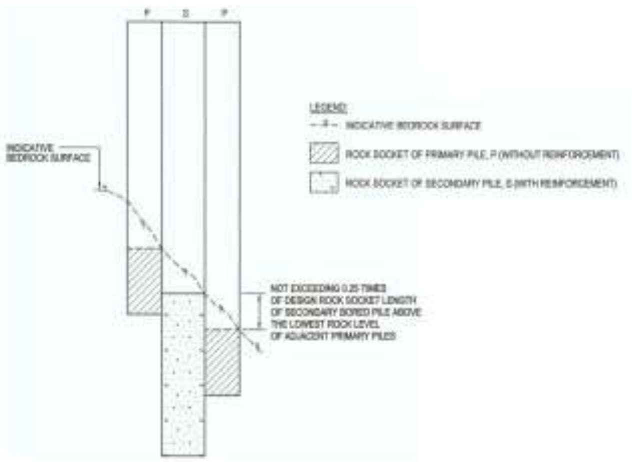

Fig. 9. Requirement of Rock Socket Length for Secondary Pile.

Two combinations of secondary and primary pile i.e. 1480mm/1180mm and 1180mm/880mm have been adopted dependent on the wall retained height. Geometry of primary and secondary pile in relation to retained height adopted is presented in Table 2. Overlapping is determined by allowing vertical deviation of 1:200 by both primary and secondary pile in opposite direction. At part of the SBP wall where the retained height is about 35m, the lateral earth pressure is so high and the wall becomes not economical to be constructed. Hence, a jet grout block was introduced behind the SBP wall with the following reasons (refer to figure 10):

a) To reduce lateral earth pressure acting on SBP wall and hence reduced the induced structural forces on the wall and supporting elements.
b) To control excessive water seepage through SBP wall as the overlap tends to reduce with depth as the control of verticality of wall become more difficult.

Table 2. Geometry of primary and secondary pile in relation to

retained height

Retained height (m)	Combination of Secondary/ Primary Pile			
	1480mm/1180mm		1180mm/880mm	
	Overlapping (m)	Pile Spacing (m c/c)	Overlapping (mm)	Pile Spacing (m c/c)
10	0.1	2.46	0.1	1.86
20	0.2	2.26	0.2	1.66
30	0.3	2.06	0.3	1.46
40	0.4	1.86	-	-

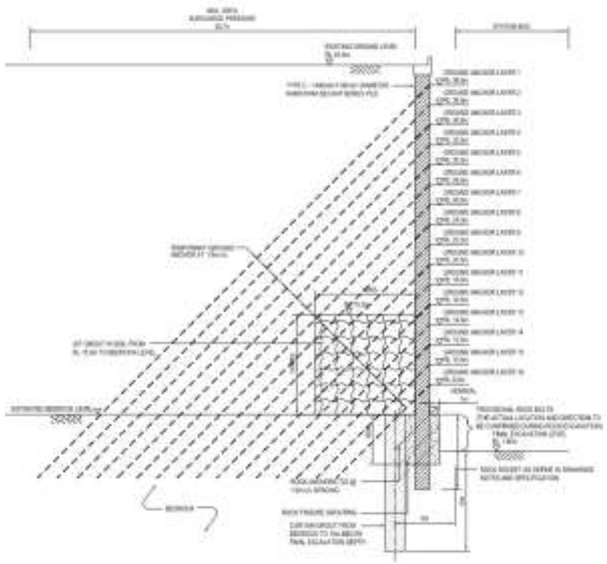

Fig. 10. Cross section of SBP wall with jet grout block.

2.3 Ground Treatment Scheme in Limestone

As the Final Excavation Level (FEL) i.e. 40m below ground level is deeper than the rockhead level of the CSL station (except some localized deep limestone valley), excavation by control blasting to form the vertical rock slope is required. Before the excavation, ground improvement by rock fissure grouting along the perimeter of CSLS station from rockhead level down to 10m below the FEL is carried out to minimize groundwater drawdown during excavation to prevent ground settlement that could cause distress to adjacent buildings. The grouting pattern consisting of primary/secondary/ tertiary holes as set out in a pattern of 4m/2m/1m as shown in figure 11 has been adopted. If grout intake for the primary hole is more than 5m³ per section for a specified pressure at a given depth, then secondary holes are drilled at the adjacent location after completion of grouting to the primary holes using closure sequence. The same goes for the tertiary holes when the intake in the secondary holes are still excessive i.e. more than 5m³ so forth.

3 CONCLUSION

Temporary ERSS with combination of DSM wall and SPB wall exhibit flexibility on termination of wall toe by following the highly undulate limestone rockhead profile and successfully been adopted to facilitate the deep excavation for CSL underground station. Perimeter rock fissure grouting adequately control water seepage during rock excavation. Figure 12 shows the excavation progress as of October 2018 where part of the station reaching 30m below EGL.

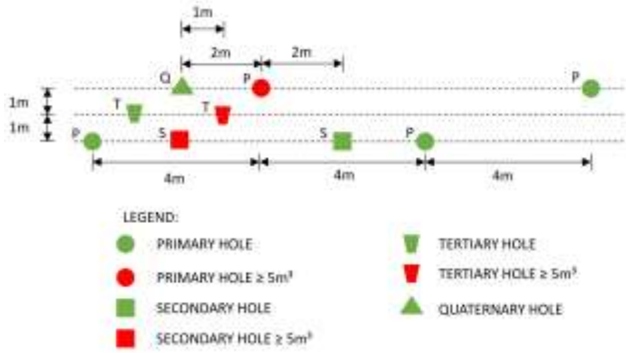

Fig. 11. Proposed Fissure Grouting Pattern

Fig. 12. Progress of excavation as of October 2018.

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FHWA-HRT-13-046 (2013). Federal Highway Administration Design Manual: Deep Mixing for Embankment and Foundation Support.