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Jet grout application for excavation in soft marine clay

Application de l'injection par jet (jet grout) pour l'excavation dans l'argile marine molle

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ABSTRACT

An excavation for the construction of a cut and cover tunnel has been carried out in deep soft marine clays. Due to practical and economical considerations, the retaining walls comprised braced sheetpiling to support the excavation have been founded in marine clays without being toed into a stronger soil layer at depth. Jet grouting has been used to stabilise the ground below the base of the excavation and to provide lateral support to the sheetpiling during excavation. The performance of the jet grout layer was verified by field tests and monitoring data.

RÉSUMÉ

Une excavation pour la construction d'une tranchée couverte est réalisée dans des argiles marines molles profondes. Pour des raisons pratiques et économiques, les murs de soutien composés de palplanches à treillis pour l'excavation ont été fondés dans des argiles marines sans être attachés dans une couche de sol plus forte en profondeur. L'injection par jet a été utilisée pour stabiliser le sol en dessous de la base de l'excavation ainsi que pour fournir un soutien latéral aux murs en palplanches pendant l'excavation. La performance de l'injection par jet est vérifiée par des essais sur le terrain et par le contrôle des données.

1 INTRODUCTION

Contract 421 (C421) of the 12 km long Kallang and Paya Lebar Expressway (KPE) in Singapore comprises the design and construction of approximately 1.5 km of dual 3-lane twin-cell box vehicular tunnel structure from the East Coast Parkway (ECP) to Nicoll Highway, crossing the Geylang River. The contract includes an interchange at KPE/ECP with four on-off slip roads, at grade and depressed roads, upgrading and recambering of the existing ECP as well as the construction of a ventilation building.

A 240 m length of the tunnel extending from the southern contract boundary at Ch -210 to the ECP at Ch +30 (see Figure 1) is located in an area of recently reclaimed land with

alluvial/fluvial deposits up to 50 m thick. The width of the excavation ranged approximately from 40 m to 60 m and the maximum excavation depth was 20 m. A new slip road lies to the east of the main alignment and was to be constructed simultaneously with the main tunnel between Ch -100 and Ch +30 approximately. The tunnel and slip road are constructed by the cut and cover bottom-up method using sheetpiling and internal bracing for support of the excavation.

Due to the practical installation constraints and economical considerations, the sheetpiles were founded in marine clays without being socketed into the cemented dense sands at depth. Jet grouting has been used to control basal stability and provide additional lateral support to the sheetpiles during excavation.



Figure 1. Section plan Ch -210 to Ch +30

2 GEOTECHNICAL MODEL

The geotechnical investigations indicated that the site was underlain by poorly compacted fill, 6 m to 15 m thick, overlying a series of marine clays (upper marine clay AuM and lower marine clay ALM) interbedded with fluvial/alluvial deposits (noncohesive soils F1 and cohesive soils F2) to 50 m depth. These deposits overlie cemented dense sands (a.k.a. Old Alluvium OA, comprising weathered unit OA-W1, two slightly weathered units OA-SW1 and OA-SW2, and the cemented unit OA-CZ).

The measured groundwater levels were within 1 m to 1.5 m below the ground surface level.

The marine clays are of particular geotechnical significance due to their low strength, high compressibility and low permeability. The field and laboratory test results show that the undrained shear strength S_u of the marine clays increases with depth and ranges between 10 kPa and 60 kPa. For design, a lower bound S_u of $0.25\sigma'_{v0}$ for AuM and S_u of $0.20\sigma'_{v0}$ for ALM were assumed, where σ'_{v0} = effective overburden pressure. Estimation of undrained Young's modulus for the marine clays was based on the assumption $E_u = 300S_u$. The drained modulus was calculated assuming a Poisson's ratio of 0.3.

Due to the variable subsurface conditions over short distances, soil profiles were established for both the western and eastern sides of the excavation. The geotechnical parameters adopted in the analysis are summarised in Table 1. These parameters include the total unit weight γ_t , effective cohesion c', effective friction angle ϕ ', effective Young's modulus E', the incremental increase in Young's modulus with depth E'_{inc}, undrained shear strength S_u, the incremental increase in undrained shear strength with depth S_{u-inc}, Poisson's ratio υ ' and permeability k.

The depths to the top of each layer in Table 1 correspond to a typical section at Ch 0 where the ground surface level was RL 103 m. At this location the excavation width was 50 m and the maximum excavation depth was 20 m (see Figure 2). The groundwater table was assumed to be at RL 101.5 m.

3 SUPPORT SYSTEM

The bottom-up cut and cover method was adopted for Section Ch -210 to Ch +30. For economic reasons, the temporary sheetpile retaining walls were designed to terminate in ALM at RL 71.5 m for the main alignment walls and at RL 75.0 m for the slip road walls. A jet grout layer was formed to act as a structural slab to maintain stability of the retaining wall below the base of the excavation. As the jet grout slab was formed by a series of short jet grout piles (JGP) it would be subjected to both axial compression from the opposite sheetpile walls and bending due to heaving on its underside. The thickness of the jet grout slab was designed so that the predicted maximum horizontal stresses in the slab were maintained at a level below the unconfined compressive strength (UCS) of the jet grout to avoid crushing failure. However, at the top of the jet grout slab adjacent to the sheetpile walls, very high horizontal stresses were predicted and could not be reduced by moderately increasing the slab thickness. Therefore, higher strength grout was used in the JGP for a 1.6 m width adjacent to the retaining walls. The jet grout was installed in marine clays with a slab thickness varying between 3.0 m and 3.5 m.

The embedment lengths of the sheetpile walls were determined based on basal heave stability criteria. Generally, the total length of the sheetpile walls was twice the depth of excavation. Temporary bored piles were installed to anchor the jet grout slab and prevent uplifting of the slab during excavation. As excavation progressed, internal struts and raker piles were used to provide lateral support. Figure 2 shows a typical temporary support system where the slip road is constructed simultaneously with the main tunnel. The various relevant structural components and their properties are summarised in Table 2.

The characteristic UCS of grout to be achieved on site was specified to be 1000 kPa with 95% confidence, corresponding to an undrained shear strength of 500 kPa. The standard deviation of UCS was assumed to be 700 kPa, a value obtained from similar projects in Singapore. Based on these criteria, a mean UCS of more than 2000 kPa could be achieved. For the higher strength jet grout adjacent to the retaining walls, the characteristic UCS was specified to be 1600 kPa. The ratio E_u/S_u for the jet grout was assumed to be 300.

Layer	Depth to top of layer		γ _t	c'	φ'	E'	E'inc	Su	S_{u-inc}	υ'	k
	West (m)	East (m)	(kN/m^3)	(kPa)	(deg)	(kPa)	(kPa/m)	(kPa)	(kPa/m)		(cm/s)
Fill(Clayey)	0.0	0.0	19	4	25	17000	-	40	-	0.3	10-5
Fill(Sandy)	4.2	2.9	19	2	28	10000	-	-	-	0.3	10-4
AuM	11.3	11.8	16	0.3	22	3887	390	15	1.5	0.3	10-7
F2	18.3	16.9	19	0.3	26	25000	-	50	-	0.3	10-6
ALM	23.0	22.5	16	0.3	22	8622	312	33.2	1.2	0.3	10-7
F2	37.0	-	19	0.3	26	25000	-	50	-	0.3	10-6
OA-W1	42.0	-	19	10	30	26000	-	100	-	0.3	10-5
OA-SW2	46.0	-	20	20	32	52000	-	200	-	0.3	10-5
OA-CZ	48.0	36.5	20	35	35	156000	-	600	-	0.3	10-6

Table1: Geotechnical Design Parameters

Table 2: Structural Components

Structure	Structure Type	RL (m)	EA (kN/m)	EI (kNm ² /m)
Retaining walls	Soldier pile 800×300-241 at 1.2m c/c + steel sheetpile SX10		7.934×10 ⁶	5.795×10 ⁵
Main excavation strut level 1	2×W24-155 Universal Beam at 7.5 m c/c	100.0	1.079×10^{6}	-
Main excavation strut level 2	2×W24-155 Universal Beam at 7.5 m c/c	96.9	1.079×10^{6}	-
Main excavation strut level 3	2×W24-174.1 Universal Beam at 7.5 m c/c	92.9	1.209×10^{6}	-
Main excavation strut level 4	2×W24-194.9 Universal Beam at 7.5 m c/c	88.9	1.357×10^{6}	-
Main excavation raker pile	2×W24-82 Universal Beam at 3 m c/c	91.0	1.428×10^{6}	-
Slip road strut level 1	2×W24-155 Universal Beam at 4 m c/c	100.0	2.023×10^{6}	-
Slip road strut level 2	2×W24-155 Universal Beam at 4 m c/c	96.5	2.023×10^{6}	-
Slip road strut level 3	2×W24-174.1 Universal Beam at 4 m c/c	92.5	2.266×10^{6}	-
Slip road strut level 4	2×W24-174.1 Universal Beam at 4 m c/c	89.0	2.266×10^{6}	-
Kingpost	H300-94 Universal Column at 3 m c/c (2 nd , 3 rd and 7 th row from west	main wall)	8.200×10^{5}	-
Bored piles	1.0 m dia. at 3 m c/c $(8^{th}, 9^{th}, 10^{th} \text{ and } 11^{th} \text{ row from west main wall})$		6.533×10 ⁵	-
Bored piles	1.2 m dia. at 3 m c/c $(1^{st} \text{ and } 5^{th} \text{ row from west main wall})$		6.533×10 ⁵	-
Bored piles	1.2 m dia. at 6 m c/c (4^{th} and 6^{th} row from west main wall)		3.267×10^{5}	-

4 NUMERICAL MODELLING

The interaction between soils, the jet grout slab (3.5 m thick), bored piles and sheetpile walls were fully modelled using the finite element analysis package PLAXIS. The staged construction of the temporary retaining structures was analysed under the plane strain condition. Consolidation of the soil associated with excavation was incorporated in the analysis. The Mohr-Coulomb model was adopted to simulate the elasto-plastic soil behaviour. The sheetpile walls, bored piles and kingposts were modelled as beam elements and the struts as anchor elements. The jet grout layer was modelled as "nonporous" soil elements, i.e., with no pore pressure build up, with the assumed jet grout strength and stiffness. The interface between soil and structural elements was modelled using strength reduction factors of 0.5 and 0.67 for clay and sand materials respectively.

A reduction factor of 0.5 was adopted for the interface be-

tween the jet grout and the bored piles to take into account theactual contact area between the two in the 3D condition.

The analysis modelled a full width of the excavation with varied subsurface profile. The bottom boundary was restrained from both vertical and horizontal movements while the side boundaries were free to move vertically but were restrained horizontally. The bottom boundary was considered as a closed flow boundary and the side boundaries as recharge boundaries where the water head remained constant. A uniform pressure of 20 kPa was applied to the pre-excavation surface next to the sheetpile wall to simulate construction loads and 5 kPa was applied to the natural ground surface to model general activities in the surrounding area. The side boundaries of the model were located 100 m west of the main excavation west wall and 74 m east of the slip road east wall. The bottom boundary was located at RL 30 m. The groundwater table within the excavation was considered to coincide with the excavation level, i.e. 1.2 m below the strut level.



Figure 2. Temporary support system for main alignment and slip road at Ch 0



Figure 3. Finite element mesh



Figure 4. Horizontal stresses in jet grout slab below main excavation

Figure 3 illustrates the finite element mesh adopted for the analysis. Figure 4 presents the calculated effective horizontal compressive stresses in the top and bottom fibre of the jet grout slab when excavation has reached final excavation level at RL 83.6 m. At bored pile locations small peaks in horizontal stresses were observed.

5 JET GROUT TESTS

Jet grout trial tests indicated double tube jet grouting using single nozzle could achieve the targeted characteristic jet grout UCS of 1000 kPa. A triangular pattern was adopted for jet grouting works and the water/cement ratio used was 0.9. The nominal diameter of the jet grout piles was 2200 mm.

Further field tests were conducted during the installation of the jet grout piles. Fifty five samples from thirty seven cores were obtained from field tests. The mean sample UCS was 3122 kPa, with a standard deviation of 1735 kPa. 10 samples obtained UCS of 4500 kPa or above. When test results over 4500 kPa were excluded from the data set, the mean sample UCS was reduced to 2363 kPa, with a standard deviation of 982 kPa. Sensitivity studies were carried out for a range of grout strengths based on the test results and the performance of the temporary support system was found to be satisfactory. For the data set excluding UCS results over 4500 kPa, the E_u/S_u ratio was found to be 258. The mean UCS for the samples obtained from the upper marine clay layer was 1937 kPa while the mean UCS for samples obtained from the lower marine clay layer was 2578 kPa.

No tests were undertaken on the higher strength jet grout adjacent to the sheetpile walls.

6 FIELD PERFORMANCE

The sheetpile wall deflections measured on site are compared against the "Class A" predictions, when the excavation reached its final level at RL 83.6 m. Figure 5 shows the wall deflections for the west main alignment wall, east main alignment wall and slip road wall. For the locations of the monitoring instruments refer to Figure 1. The predicted wall deflections were obtained directly from the original analysis and have not been subjected to any back-analysis. At monitoring points L5004H and L5005H the measured heaves of the jet grout slab ranged from 0 mm to 30 mm, compared with the predicted maximum heave of 35 mm.

7 CONCLUSION

Jet grout piles were successfully applied as a structural slab below the base of a cut and cover excavation, which allowed the use of shorter sheetpile retaining walls. Finite element analysis was used to establish the thickness and design strength of the jet grout layer. Measured data indicated that a jet grout layer was effective in controlling lateral wall deflections and base heave.

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Figure 5. Measured and predicted sheetpile wall deflections