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EXCAVATION ANALYSIS OF UNDERGROUND EXPRESSWAY CONSTRUCTION

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ABSTRACT

Construction of an underground expressway in Singapore was carried out by cut-and-cover method in soft clay and residual soils. The supporting system consists of heavy steel sheet pile braced with a system of ground anchors and internal strut to form a cofferdam crossing a major river. The movement of the combined bracing system during the excavation period was monitored by five inclinometers at different location around the cofferdam. The time-dependent deformation of each instrumented sections behaved differently due mainly to different excavation sequence and support installation and preload, in addition to differences in structural geometry and slight variation in soil condition. The results from the proposed analyses compare favorably with the field records. The influence of uneven excavation on the deformation of the excavation support system is also studied.

Keywords : Excavation support system, consolidation analysis, underground expressway.

INTRODUCTION

In Singapore, most excavations in deep deposits of soft clay are supported by sheet piles braced with internal struts or ground anchors. The analysis of many deep excavations have been successfully done by considering various construction aspects i.e. simulation of actual excavation sequence, excavation support system and sequence of installation of struts and anchors. Field data from such excavations indicate that ground movements and strut loads tend to change with time (Lee et al., 1986). This phenomenon may be attributed to the expansion of the retained soil and the soil below excavation level arising from the dissipation of excess negative pore pressures. Lee et al. (1989) and Parnploy et al. (1992) considered the effect of excess negative pore

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pressures generated during soil removal by fully coupling an elastic-perfectly plastic soils with Biot consolidation analysis. The proposed analysis was depicted using the Mohr-Coulomb yield surface criterion for illustrative the time-dependent mechanism of a 11m deep strutted excavation by Yong et al. (1989). The consolidation analysis gives results in between undrained analysis and drained analysis, the degree of consolidation analysis in each stages apparently ranges from 80% to 90% of the drained analysis

In this paper, a case analysis is presented on the time-dependent deformation of the underground supporting systems consisting of heavy steel sheet piles braced with a system of ground anchors and internal struts to form a cofferdam crossing a major river. The main reason of using the steel sheet piles to replace the conventional continuous bored piles well-performing on land construction is due to non-seepage interlocks. Following the governing equation used in the analysis, the behavior of a combined bracing system above is investigated. The effect of uneven excavation and high preloading of ground anchors is also studied.

SOIL MODEL AND GOVERNING EQUATION

An elastic-perfectly plastic model using the isotropic Mohr-Coulomb yield criteria with associated flow rule is used to describe soil behavior in the analysis so as to illustrate the ground movement mechanism and disregard the irrelevant feature of yield surface cap model. With this model, the governing equation for consolidation analysis was formulated based on the usual assumptions of saturated soil, principle of effective stress, incompressibility of soil solid and pore water, validity of Darcy's law, and small deformations. Finite element formulation for consolidation in soils was adopted with Britto and Gunn (1987)'s procedure utilizing a backward difference time marching scheme. With this procedure, the unknown displacement and pore pressure increments at each time step were computed using the following relationship:

$$\begin{bmatrix} K & L \\ L^T & -H\Delta t \end{bmatrix} \begin{bmatrix} \Delta u \\ \Delta p \end{bmatrix} == \begin{bmatrix} \Delta f_1 \\ H\Delta t\, p_1 \end{bmatrix} \quad \ldots 1$$

where **K** = the elasto-plastic stiffness matrix; **L** = coupling matrix; **H** = flow matrix; **Δu, Δp** = unknown displacement and pore pressure increments at the current time step; **Δt** = size of time step; **p** = excess pore pressure at the end of the previous time step and ; **Δf** = nodal load increments in the current time step

SITE PLAN AND GROUND CONDITION

Part of the underground expressway is aligned to cross under a Singapore river shown in Figure 1. To maintain flow along the river and to ensure that the construction of the river crossing does not cause flooding at the upstream end of the river, the cut and cover works were carried out in two stages. The 1st stage of the river crossing involved constructing a cofferdam on the South bank of the river, shaded as shown in Figure 1, and the other stage to be complete on the North bank.

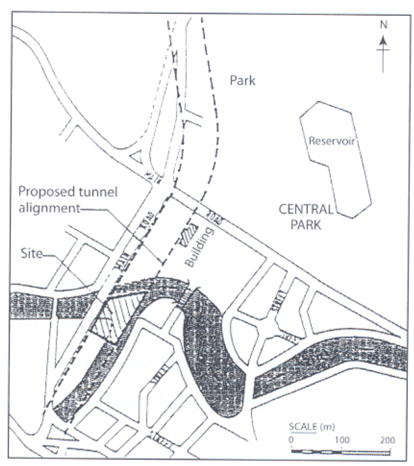
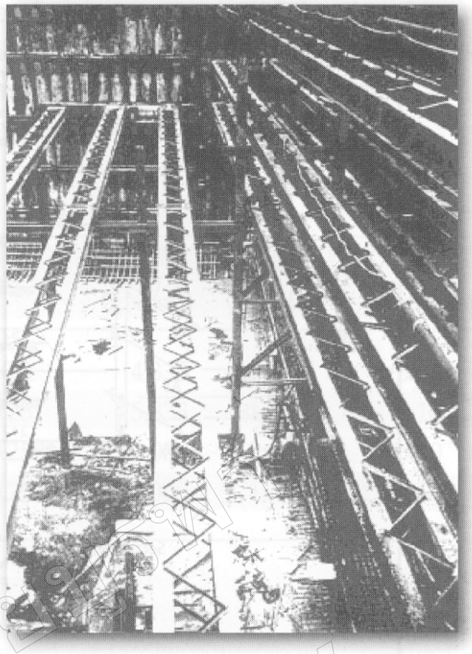


Figure 1. Underground expressway alignment and site location.

A plan layout of the excavation support system and site inclinometers monitoring during the construction of the 1st stage is shown in Figure 2 for part of the case to be presented. The 56 m-wide and 18 m-deep excavation was supported by heavy steel sheet pile braced with ground anchors and internal struts. In view of the configuration of the cofferdam layout and the need to provide free access for the construction of the underground box expressway tunnel, three systems of bracing were adopted; system x (Picture 1), system y and system z (Picture 2).



Picture 1. Excavation support system (Section x is on the right hand of photo) for underground expressway.

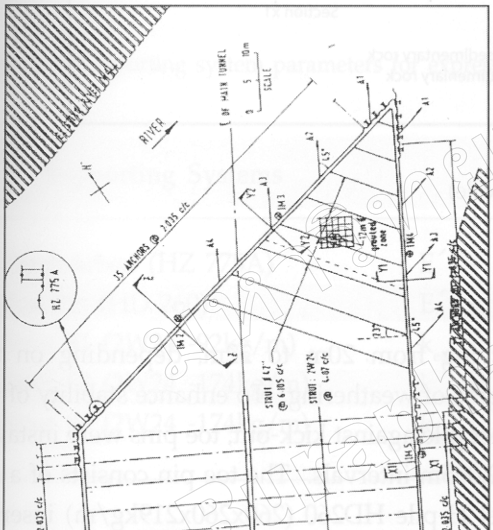
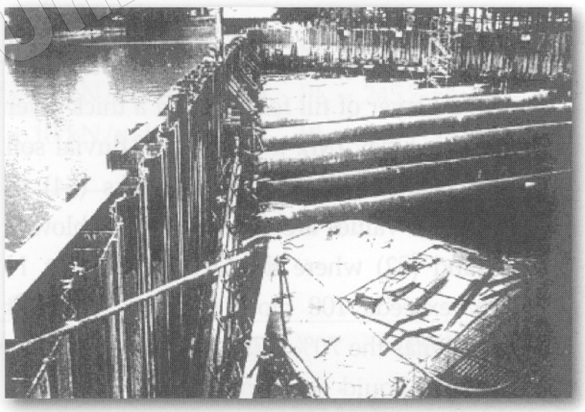


Figure 2. Layout of excavation support system.



Picture 2. Excavation support system (Section y) for underground expressway.

SECTIONS X1 AND X2

A cross-section of supporting system and subsoil conditions at sections x1 and x2 are shown in Figure 3. The corresponding inclinometers closest to these two sections are IM5 and IM1 respectively as shown in Figure 2.

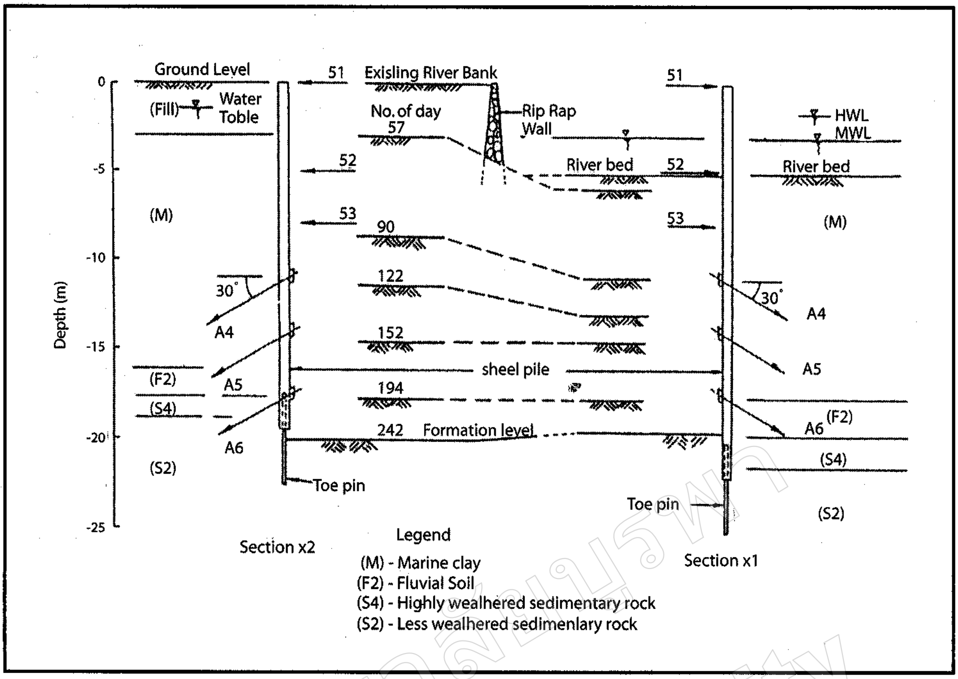


Figure 3. Supporting system at Section x2 and x 1 (not true scale)

The ground condition at section x2 consists of a layer of fill followed by a thick layer of soft marine clay (M), medium stiff fluvial soil (F2) and weathered sedimentary rocks (S4) in which SPT 'N' values are about 30 to 100 blows/300mm and (S2) where the SPT blow count, N generally exceeds 100 blows/300mm. The soft marine clay has the 70% - 75% nature water content, the 90% liquid limit and the 60% plasticity index with the SPT 'N' values of 1 to 2 corresponding to the undrained shear strength of 22 to 35 kN/m2. The medium stiff fluvial soil (F2) has higher N values of about 2 to 10 and the undrained shear strength of 28 to 40 kN/m^2.

The sheet pile consists of a combination of double I-sections interlocked with intermediary Z-sections to form a heavy composite section designated as HZ 775A. The HZ 775A with a section modulus of 7960 cm^3/m were driven into the weathered sedimentary rock to a depth varying from 20m to 22m, depending on the degree of weathering. To enhance stability of the sheet pile against kick-out, toe pins were installed at 2.065m intervals. The toe pin consists of a 5m steel H-pile HD260 (260x260x219kg/m) inserted into a prebored hole through the double I-section of HZ 775A. The upper 2m of the steel H-pile was then grouted to integrate with the TradeArbed HZ 775A leaving the remaining 3-m socketed into the S2 material.

At the 1st level, each strut consists of double H-beams (2W 610x180x92kg/m, Grade 50, hereafter designated as 2W24x92kg/m) brace together by short struts, L 75x75x9mm laced at 45°. At the 2nd and 3rd levels, the double H-beams (2W 610x325x174kg/m, Grade 50, designated as 2W 24x174kg/m) are large, because of the expected higher loads at the lower levels and the two beams are braced together by 45° lacing. The king posts (H 400x400x172kg/m with spacing of

4.07m center to center) were designed to support the struts and to provide the restraining force of strut in the direction of buckling. Bracing in the lower levels consists of 3 levels of ground anchors, A4, A5 and A6, inclined at an angle of 30° to the horizontal and with the 2.065m spacing.

Table 1. Material parameters for expressway excavation.

Soil Type	γ (kN/m^3)	ϕ'	C´ (kN/m^2)	Ko	ν'	E´ (kN/m^2)	k_v (m/sec)	k_h (m/sec)
M	16	22	0	0.625	0.3	4,333	2×10^{-8}	6×10^{-8}
F2/Fill	18	30	0	0.55	0.25	5,833	1×10^{-7}	1×10^{-7}
S4	20	30	30	0.80	0.25	90,000	2×10^{-8}	2×10^{-8}
S2	20	40	100	0.80	0.25	150,000	1×10^{-8}	1×10^{-8}

Table 2. Supporting system parameters for expressway excavation.

Supporting Systems	Stiffness	Remarks
TradeArbed (HZ 775A)	$E' = 16.7 \times 10^6$ kN/m^2/m	$\nu' = 0.25$
Toe Pin (HD 260)	$E' = 0.82 \times 10^6$ kN/m^2/m	$\nu' = 0.25$
Strut S1 (2W24 -92kg/m)	$k_{eq} = 43.08 \times 10^3$ kN/m/m,	No preload
Strut S2 (2W24 -174kg/m)	$k_{eq} = 80.00 \times 10^3$ kN/m/m,	PL = 245.7 kN/m
Strut S3 (2W24 -174kg/m)	$k_{eq} = 80.00 \times 10^3$ kN/m/m,	PL = 245.7 kN/m

Note : PL = preloading.

An anchor no.	Nearest inclinometer	Equivalent stiffness (kN/m)	Working load (kN)	Preload (kN)
A4	IM1	12800	1250	1200
A4	IM5	13250	1250	1200
A5	IM1 to IM5	14000	1500	1200
A6	IM1 and IM5	14000	1500	1200

MATERIAL PROPERTIES AND BOUNDARY CONDITIONS

The material properties used in the analysis are summarized in Table 1. All soil properties are obtained from site investigation report as well as the MRT geotechnical reports and as well checked with the correlations presented in Dames and Moore (1983) and Tan (1983).

Typical finite element mesh and boundary conditions used in the analysis are shown in Figure 4. The soil and sheet pile are represented by 8-noded quadrilateral elements. The equivalent Young's modulus for the sheet pile wall was obtained by matching the flexural stiffness of the sheet pile element to that of actual sheet pile. As shown in Figure 4, only half of the excavation was modeled. The unbalanced forces from any uneven excavation will be redistributed on both sides of the retaining walls with the force distribution factor of 0.5.

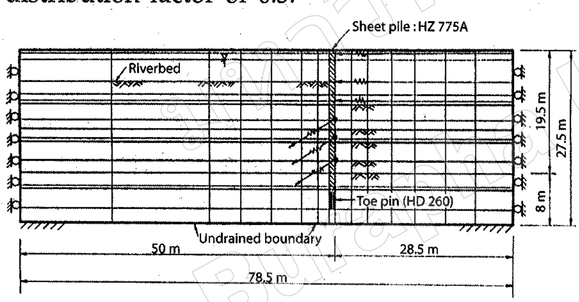


Figure 4. Finite element mesh and boundary conditions (section x1).

Each layer of struts and ground anchors is modeled by a one-dimensional spring element as shown in Figure 4 and k_{eq} of strut is determined by dividing the axial stiffness of struts by the horizontal strut spacing. The anchor stiffnesses are estimated from proof load tests in similar soil in the vicinity. Table 2 summarizes the properties of the supporting system and of ground anchors as well used in the analysis in sections x1 and x2.

The excavation sequence was not properly recorded and the simulation of excavation procedure in the analysis was estimated from the recorded dates of strut installation and ground anchor preloading with an assumed excavation rate of 2-3m depth per week. Before the comment of excavation, water inside the cofferdam was pumped out over a period of one week. Times used in the proposed consolidation analysis for each stage of excavation are indicated in Figure 3.

COMPARISON ON FIELD MEASUREMENT

Deformation of sheet pile wall

Sections x1 and x2 are located opposite to each other as shown in Figure 3. In spite of similar supporting system, the inclinometer IM1 and IM5 deformed differently in both magnitude and rate of deformation due to difference in ground elevation and sequence of excavation.

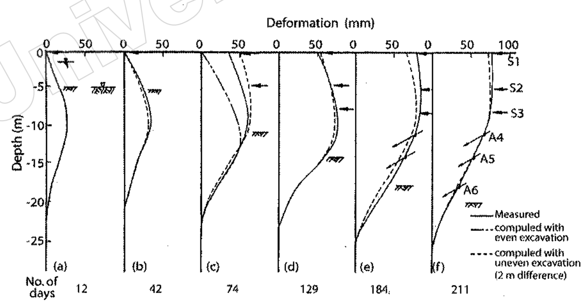


Figure 5. Measured and computed sheet pile deformation at section x1

Figure 5 shows the deformation of section x1 during the excavation period. After installation of the 1st level of struts, water was pumped out from the cofferdam over a period of one week. In Figure 5a, the wall bulged out with a maximum deflection of 25mm at about 10m depth due to water drawdown. An increase in deformation of sheet pile wall was observed in the lull period of one month (12-42 days) where no excavation was in progress (Figure 5b). Then, due to the schedule for installation of foundation bored piles along the side of section x1 the rate of excavation at sections x1 and x2 was different leading to an uneven excavation of about 2 to 2.5m across the site as shown in Figure 3. Figure 5c shows the increased in measured deflection of about 35mm at the top resulting from the unbalanced excavation. Without considering uneven excavation in the analysis, there is poor agreement between measured and computed result (Figure 5c). From back analysis using simplified iteration for unbalance excavation with the force distribution of 0.5, the 2-m difference in excavation area with double layer of strutting system gave a best match as shown in Figure 5c. After the upper three strut levels, the sidesway movement was reduced by controlling the excavation process to ensure that any uneven excavation is kept as small as possible.

As shown in Figures 5e to 5f when ground anchors were preloaded and installed, each stage of excavation yields less deformation increment compared to that measured for earlier stages of excavation supported by struts (Figures 5a to 5d). This is due to the fact that preloading of anchors appears to be effective in restraining deflection of the sheet pile system.

In the early stages of excavation and strut installation, the time-dependent deformation from both field observation and the proposed analysis was about 0.25 to 0.6mm/day when there was no excavation and 1.6 to 3.0mm/day during excavation. The time-dependent behavior of the monitored wall during the later stages when ground anchors were used appears to be smaller and was observed to be less than 0.2mm/day where there was no excavation and less than 1.25mm/day during excavation.

Figure 6 shows the lateral deflection of section x2 during excavation period monitored by IM1. First, the existing river bank was excavated up to a level of 0.5m to install the 1^{st} level of

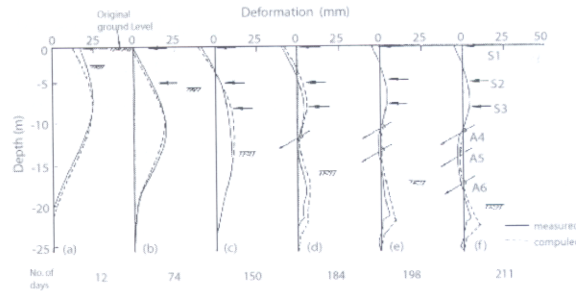


Figure 6. Measured and computed sheet pile deformation at section x2

struts. This took about 4 to 5 days. The wall at IM1 deflected in a cantilever manner by about 15mm at the top. After the 1st level of struts was installed at section x1 and x2, the wall at section x2 bulged out by a maximum 25mm at about 10m depth. As a result of the uneven excavation discussed above, section x2 at this stage was back analyzed using the simplified approach to redistribute the unbalance strut forces. The movement of about 20mm towards the retained soil mass (Figure 6b) was determined and found to be consistent with field measurements. Further preload of 2^{nd} and 3^{rd}-level struts pushed the wall backward by 2mm into the retained soil side (Figure 6c).

The effect of high anchor preload which is about 96% of the design working load caused a reversal in curvature in the retaining wall as shown in Figs.6d to 6f. As excavation proceeded after the 5^{th} level of support was installed (Ground Anchor A5), the toe resistance appears to be exceeded indicating a possible kick-out at the toe of the sheet pile wall. However, inclinometer measurements indicate that this is just local failure at the toe pin and overall stability was not impaired. When the formation level was reached, it was recommended to pour the floor slab concrete immediately for stability purpose. After the floor stab was cast, there was no further lateral movement in wall and toe pin. The rate of wall deformation during the early stages of excavation when strutting was used was found to be 0.2 to 0.4mm/day where there is no excavation and 1.0 to 3.0mm/day during excavation. Movement of retaining wall appears to be small and insignificant after ground anchors with high preload was installed.

Surface ground settlement

With a heavy section modulus sheet pile, i.e., TradeArbed, the ground settlements extend up to a distance d, equal to the maximum depth of excavation H. The computed ground settlements at IM1 are small corresponding to the small lateral deformation. The settlements at IM5, are more significant whereby the corresponding lateral deformation of the retaining walls also show relative large movement response. The ratio of maximum settlement to maximum lateral deformation for IM5 is 0.38. In general, ground settlements at the riverbed level behind the retaining wall in the present analysis are small and the distance of influences are limited within a distance of one times the maximum depth of excavation, or within Zone I of Peck's settlement profile given for soft to hard clay with average workmanship.

Apparent earth pressure diagram

Figure 7 shows the 'apparent' earth pressure computed from strut loads and anchor forces compared with apparent earth pressure diagram suggested by Terzaghi and Peck (1967) assuming m=1.0 and a weighted average shear strength of 32 kN/m^2. In both cases, surcharge is not considered. The shear strength value was obtained by weighting the undrained shear strength over the depth of excavation for each section and averaging it over the five sections. The average weighted unit weight of 16 kN/m^2 and the average height of excavation of 19.6m were obtained in a similar manner. This leads to the calculated pressure of 0.58γH shown in Figure 7. It can be seen that the

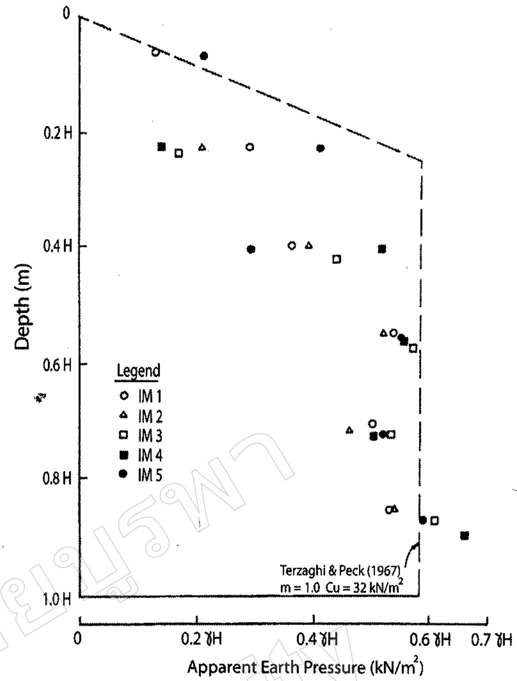


Figure 7. Apparent earth pressure diagram from computed strut loads and anchor forces.

computed envelope can predict possible support loads for retaining wall for given site conditions except anchor forces of each sections at depth of 0.9H. This slight discrepancy is due to the concentration of wall pressure below the last strut as the preloading effect. In general, Figure 7 confirms the validity of conventional pressure diagram suggested by Terzaghi and Peck (1967) when dealing with similar type of soil condition.

CONCLUSION

In general, the computed deformations of sheet pile walls based on consolidation analysis are in good agreement with monitored deformation and can account for the time-dependent behavior of excavation support system. The behavior of braced excavations in soft clay is mainly controlled by the timing of support installation and excavation processes. The combined support system, internal struts and ground anchors, provides more space for the substructure construction. The validity of conventional 'apparent' pressure diagram in bracing force computation is explicitly confirmed when dealing with similar type of ground condition.

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