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# The effect of deep excavation on pore water pressure changes in the Old Alluvium and under-drainage of marine clay in Singapore

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ABSTRACT: Many deep excavations for the construction of the North East Line of the Singapore Mass Rapid Transit System were carried out in the Singapore marine clay and the Old Alluvium (OA). Extensive geotechnical instruments were installed to monitor the behaviour of the ground, the retaining system and adjacent structures. This paper examines the large measured drawdown of the water pressure head in the OA and its effects of under-drainage on the marine clay. Numerical analysis has been carried out for better understanding of the mechanism of the responses of the pore water pressures in the OA to the excavation. The discussion will be focused on the drawdown of water pressure head in the OA, the consolidation settlement of the overlying marine clay due to under-drainage and the implications on the design of temporary support system for excavations in the Singapore marine clay and the OA.

## 1 INTRODUCTION

The Old Alluvium (OA) and the Kallang Formation are two of the four major geological formations in Singapore. The OA originated as a result of severe erosion of slopes on granite and low grade metamorphic rocks following changes in sea level and climate, Gupta et al (1987). Typically the OA is a well-graded, predominantly granular material. Some cementation has been found at depth, however where the SPT blow counts are low at shallower depth due to weathering, much of the cementation has been lost. The permeability of the OA is typically in the range of $10^{-7}$ to $10^{-10}$ m/s. The typical shear strength parameters are a friction angle, $\phi$' of 35 degrees and a cohesion, c' in the range of 0 – 30 kPa, see Wong et al (2001). The overlying Kallang Formation consists of younger deposits that are of marine, fluvial, littoral and estuarine origin. The most important unit of the Kallang Formation is the very soft to soft Singapore marine clay. The clay is normally to slightly over-consolidated with a water content close to liquid limit. The compression index is typically in the ranges of 0.6 to 1 and the permeability is low in the range of $10^{-9}$ to $10^{-10}$ m/s. Due to its low strength and high compressibility, large settlements have commonly been measured during tunnelling and excavations in it, Shirlaw & Copsey (1987) and Wen et al (2001).

The construction of the North-East Mass Rapid Transit Line in Singapore, involving 16 underground stations and linking tunnels saw many deep excavations in the Kallang Formation and the OA. An extensive instrumentation program was implemented during the construction to monitor the behaviour of the ground, the adjacent buildings and the underground utilities. The monitoring instruments included piezometers to monitor the pore water pressure changes both inside and outside the excavations. Large pore water pressure drop in the OA was monitored and where the OA underlay the very soft to soft marine clay, the loss in pore water pressure in the OA resulted in the under-drainage of the clay and large consolidation settlements took place at the ground surface.

## 2 TBM LAUNCH SHAFT AT FARRER PARK STATION

### 2.1 *Site conditions*

The launch shaft for the tunnel boring machine was at the north of the Farrer Park station. The ground condition is composed of a top layer of man made fill overlying a layer of fluvial sands. Underlying the fluvial sands are the soils of the OA. The OA is typically described as medium to coarse silty or clayey sands. The thickness of the fill is 2.5m. The fluvial sands are about 7m thick, see Figure 1. The residual soils of the OA are typically loose to medium dense with SPT ranging from 5 to 25 blow counts. The completely weathered OA is typically medium dense to very dense with SPT up to 100 blow counts.

The excavation was 17.4m deep and was supported by soldier piles with lagging up to 2m below the ground level and followed by a contiguous bored piles of 1200mm in diameter at 1700mm centre to centre. The gaps between the bored piles were sealed by jet grouting. Two levels of steel struts were erected to prop the excavation. Two Casagrande type piezometers, GWC7700 and GWC 7701 were installed next to the retaining wall at different depths to monitor changes of pore water pressures in the Old Alluvium. GWC7700 was at 8m below the ground level within the fluvial sands and GWC7701 was at 18m below the ground within the dense to very dense sands of the completely weathered OA.

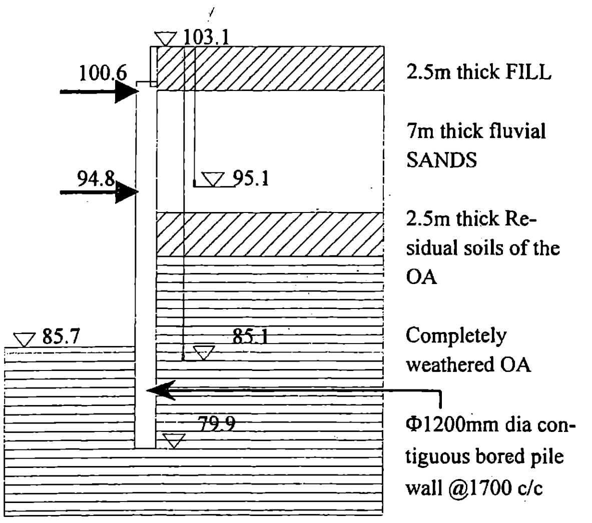


Figure 1. Soil conditions and piezometers at TBM launch shaft at Farrer Park station

## 2.2 *Monitoring results*

The excavation was carried out in stages. The first stage excavation was from the ground surface at Reduced Level (RL) 103.1m to RL 100.0m (3.1m deep), after which the first level of struts was installed at RL 100.6m. The second stage excavated to RL 94.1 (excavation depth 5.9m) followed by installation of the second level of strut at RL 94.8m. The final excavation stage excavated to the formation level of RL 85.7m, i.e. excavation depth of 8.4m, bringing the total excavation depth to 17.4m

The ground water table was at 102.1m, i.e. 1.0m below the ground level. The piezometric levels responded to the excavation almost immediately from the second stage excavation, see Figure 2. The water pressure head in GWC7700 and GWC7701 dropped 1.9m and 4.0m, respectively for an excavation depth of 5.9m in the second stage. Subsequently GWC7700 showed no change and the water pressure head remained between RL 100.5 and RL 101.0m. This is believed to be due to the fact that GWC7700 was installed with the fluvial sands and constant recharging from the ground water kept it almost unchanged during the subsequent excavation.

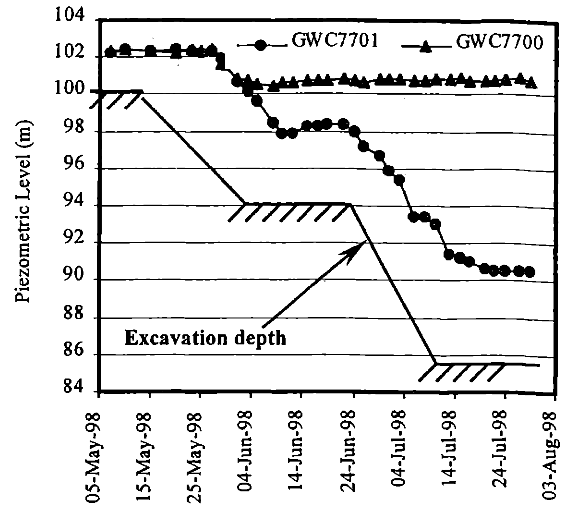


Figure 2. Piezometric level drop at TBM launch shaft at Farrer Park station

In the third stage excavation of 8.4m from RL 94.1m to RL. 85.7m, the water pressure head measured by GWC7701 dropped another 7.5m.

# 3 CUT AND COVER TUNNELS AT RACE COURSE ROAD

## 3.1 *Site conditions and details of excavation*

The cut-and-cover tunnels at Race Course Road are in the old Little India area of Singapore. To the east side of the tunnels are shop houses of 3 to 4 storey reinforced concrete construction. Many of these shop houses date back to the early part of the 20th century.

The ground in this area consists of man-made fill, fluvial sands / clays and the marine clay of the Kallang formation underlain by the Old Alluvium, see Figure 3. The thickness of the fill is typically 2 to 4 meters. Underlain the fill is a layer of fluvial sands overlain the very soft to soft marine clay. The thickness of the sand layer is 3 to 7m. The depth of the marine clay varies from 15m to 31m below ground level. Locally the marine clay is separated by a layer of fluvial deposits.

The excavation for the construction of the cut & cover tunnels was typically 17.5m below the ground level and was retained by 800mm thick diaphragm walls. 6 levels of H-section steel struts were erected to support the excavation. The diaphragm walls were terminated in the underlying Old Alluvium. The criterion for terminating the diaphragm walls was 2m into the Old Alluvium of SPT > 100 or as determined by load bearing requirement, whichever was deeper. In order to limit the wall deflection along Race Course Road, jet grouted slabs of 1.5m to 3m thick were constructed at the base of the excavation in areas where there was a significant depth of marine clay below the base of the excavation.

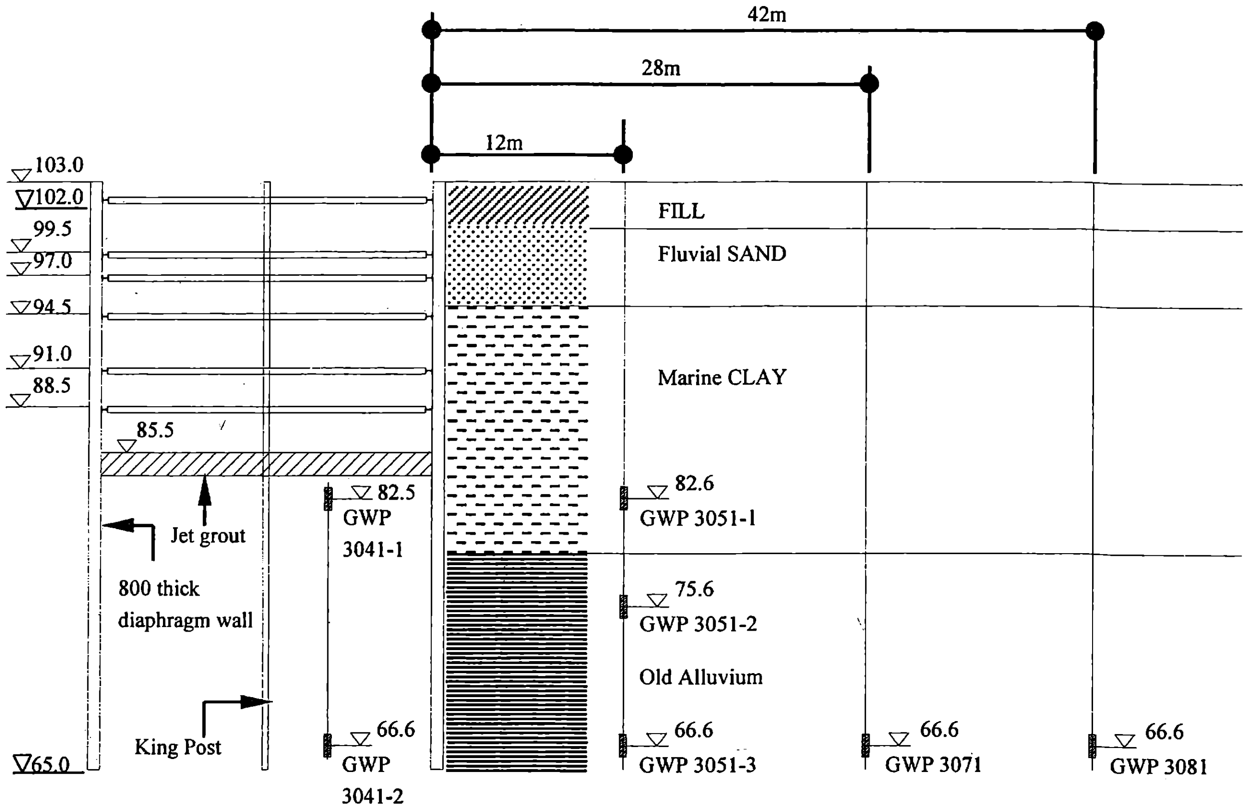


Figure 3. Excavation and instrumentation at cut and cover tunnel at Race Course Road

The jet grouted slab was installed before excavation commenced and was designed to provide effective support to the diaphragm wall to reduce the wall deflection and ground settlement. A typical cross section is shown in Figure 3.

## 3.2 *Monitoring results*

Figure 3 also shows the position of the pneumatic piezometers installed at one of the intensive monitoring sections for the cut and cover tunnels. The piezometers were installed both inside and outside the excavation, at different distances to the excavation and at different levels.

All piezometers experienced a large drop in pore water pressure head in the range of 6 to 8m when the excavation reached the final formation level, see Figure 4. It took two years after the base slab was cast for the pressure head to be restored to its original levels.

The water pressure head drawdown in GWP3041-1 (3m below the formation) followed closely with the excavation. GWP3041-2 and GWP3051-3 showed about 7m of pressure head loss when the excavation reached the formation. GWP3041-2 was inside the excavation and GWP3051-3 was outside the excavation. Both were installed 12 meters to the diaphragm wall at about the same level.

As the monitoring data for GWP 3041-1 and -2 were only available from March 1999 when the excavation was below the 5$^{th}$ level of strut, i.e. 12.5m below the ground level, Figure 4 assumes that the original ground water table was the same as that measured by GWP3051 at RL101m.

The measured drawdown of the pore water pressure head indicates that the diaphragm wall was unable to provide an effective cut-off even though the wall was installed well into the very hard layer of the OA. The Old Alluvium was permeable enough for the water pressure on both side of the wall to reach almost the steady state seepage water pressure when the excavation reached the formation. This implies that in designing the retaining system for deep excavation it is important to check the case with the steady state seepage pressure on both sides of the wall.

Unlike the case in the TBM shaft at Farrer Park station, there is a layer of compressible soft marine clay above the OA. The large pore water pressure head draw-down experienced in the OA resulted in the under-drainage of the clay. The maximum measured ground settlement at this intensive monitoring section was 110mm, of which 55% was due to the consolidation of the Marine clay as shown in Figure 5 for settlement monitoring point L641, which was 5.5m from the diaphragm wall.

The excavation was stopped for about three months at the third strut level to allow the underpinning works adjacent to the excavation to complete. Consolidation of the clay took place during this

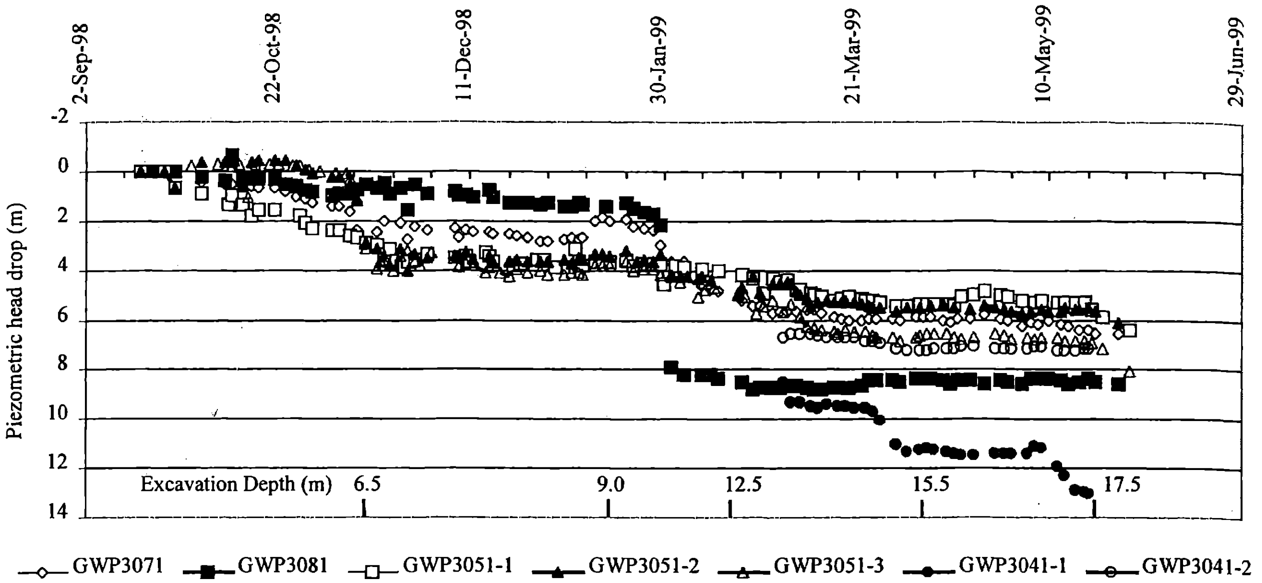


Figure 4. Piezometric head drop measured by pneumatic piezometers.

period. This is clearly shown by the straight line in the time (log scale) – settlement curve in Figure 5.

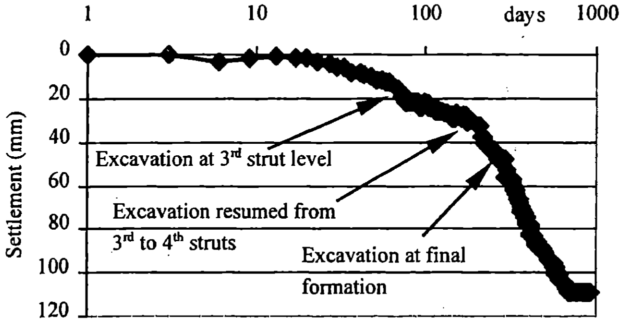


Figure 5. Settlement at L641

### 3.3 *Numerical modelling of excavation at Race Course Road*

Numerical models were set up to examine the mechanism of the observed drawdown of the water pressure head. In the analysis the typical parameters for the different soil layers were used, see Table 1. The finite element package SAGE CRISP was used for the purpose.

The parameters and the analysis model were first verified using the calculated deflection and the measured deflection of the diaphragm walls. Then the pore water pressure results were examined to see whether the general trends were matched between the calculated results and the measured results.

Figure 6 shows the calculated water pressure reduction outside the excavation. The calculated drawdown compared well with the measure values.

Table 1. Key parameters for numerical modelling

| Soil Types | Stiffness (MPa) | c' (KPa) | $\phi$' (Deg.) | k (m/s) |
|---|---|---|---|---|
| FILL | 10 | 0 | 30 | $1x10^{-7}$ |
| Fluvial SAND | 12 | 0 | 32 | $1x10^{-6}$ |
| Marine CLAY | 7.5 | 0 | 22 | $1x10^{-10}$ |
| OA (SPT < 50) | 65 | 5 | 35 | $1x10^{-9}$ |
| OA (SPT > 50) | 100 | 10 | 35 | $1x10^{-10}$ |

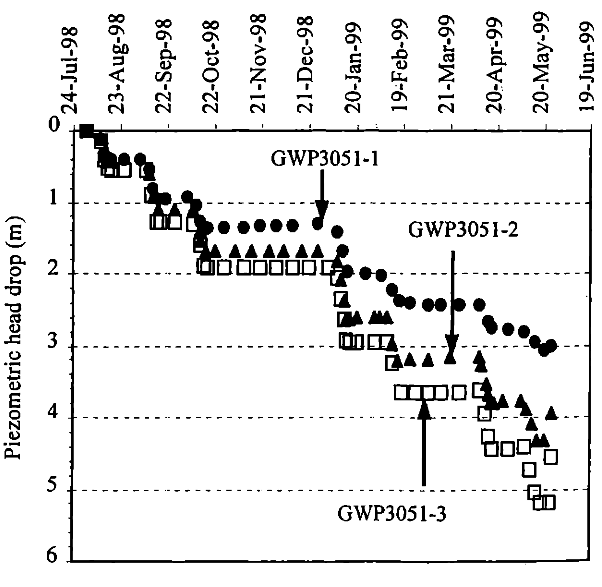


Figure 6. Calculated piezometric head draw-down

Other runs showed that the depth of the diaphragm walls did not reduce the large draw-down. This indicates that the reduction is primarily due to the stress changes in the ground. Within the excavation, the unloading caused the reduction in the major principal stress in the soil elements and this resulted in large reduction of water pressure below the formation level. Behind the retaining wall, the horizontal stress is reduced to the $k_a$ condition from the original $k_o$ condition, i.e. a reduction in $\sigma_3$'. In general the change in pore pressure can be calculated as:

$$\Delta u = B \{\Delta\sigma_3 + A (\Delta\sigma_1 - \Delta\sigma_3)] \quad (1)$$

where $B = 1.0$ for saturated soils
$\Delta\sigma_3$ = change in minor principal stress
$\Delta\sigma_1$ = change in major principal stress
A = pore water pressure coefficient

Behind the retaining wall, the overburden remains the same, i.e. there is no change in the major principal stress, $\Delta\sigma_1 = 0.0$. The change in the minor principal stress from $k_o$ to $k_a$ can be calculated as:

$$\Delta\sigma_3 = (k_a - k_o)\, \sigma_1' \quad (2)$$

Substituting Equation (2) into Equation (1) gives:

$$\Delta u = (1\text{-}A)\,(k_a - k_o)\, \sigma_1' \quad (3)$$

Based on triaxial test results, A-value for the clayey or sandy silt of the OA at GWP3051 is 0.35 and for dense silty sands at GWC7701 is –0.1, respectively. The calculated water pressure head draw-down by Equation (3) and by SAGE CRISP are plotted in Figure 7 against the measured draw-down by GWP3051-1, 3051-2 and 3051-3 at Race Course Road and GWC7701 at Farrer Park TBM launch shaft. The calculation assumes $k_o = 0.7$ and $k_a = 0.27$ for the OA. An average effective unit weight of 7 kN/m$^2$ for area with deep soft clay (GWP3051) and 10 kN/m$^2$ for area with OA soils (GWC7071) are used to calculated the effective overburden stress. The good agreement of the three sets of data confirms that the draw-down is induced by the stress changes.

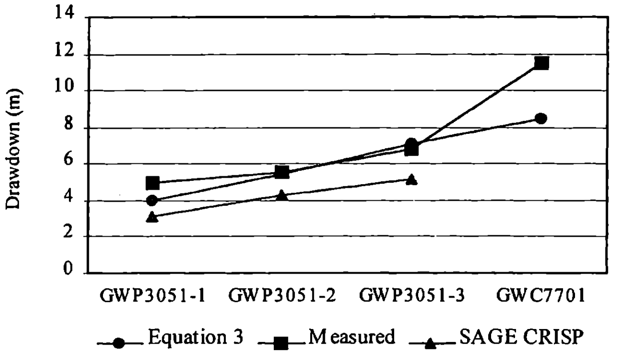


Figure 7. Comparisons of piezometric head draw-down.

# 4 CONCLUSIONS

The monitoring results of piezometers were presented for two deep excavation sites in the soils of the Old Alluvium. Numerical modelling indicates that the draw-down in pore water pressure head observed during the excavation is due to the stress changes as a result of the excavation. Deeper diaphragm walls as a form of cut-off will not reduce the draw-down. Where the OA underlies soft compressible clay, consolidation settlements will occur due to the under drainage of the soft clay. The draw-down can be estimated by using the pore water pressure coefficient A, the effective overburden stress, $\sigma_1$', the coefficients of active earth pressure, $k_a$ and the coefficient of earth pressure at rest, $k_o$.

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