Deep excavations in singapore marine clay

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ABSTRACT: Significant areas of the financial and commercial districts of Singapore have been constructed over deep deposits of near normally consolidated marine clay. Numerous deep excavations have been required for urban development and the supporting infrastructure. The design of these excavations needed to consider basal stability and (for walls taken to hard strata) net active forces below excavation level. A number of innovative solutions to these problems have been applied. Solutions have included the use of underwater excavation, lime piles, deep soil mixing and the formation of buried slabs using jet grouting. Generally, these have been successful. However, there have been a number of major excavation failures over the last 10 years. Examples of successful excavations are contrasted with some of the failures, and the causes discussed.

1 INTRODUCTION

Deep deposits of normally or near normally consolidated clay pose particular problems in the construction of deep excavations and tunnels. A significant proportion of the older urban areas of Singapore are built over deposits of soft marine clay. In particular the old Chinatown, Little India and Arab Street areas are built over deposits that extend typically to 20 m to 35 m below ground level. Reclamation south of the old Beach Road, carried out over the last 50 years, has resulted in areas for new development where the marine clay can extend to 45 m or more below ground level. The clay can still be consolidating under the weight of the reclamation fill twenty or more years after reclamation. The marine clay is the main constituent of the Kallang Formation.

Figure 1 shows the areas of Singapore where the Kallang Formation is encountered, and the areas of post war reclamation, which are underlain by soils of the Kallang Formation.

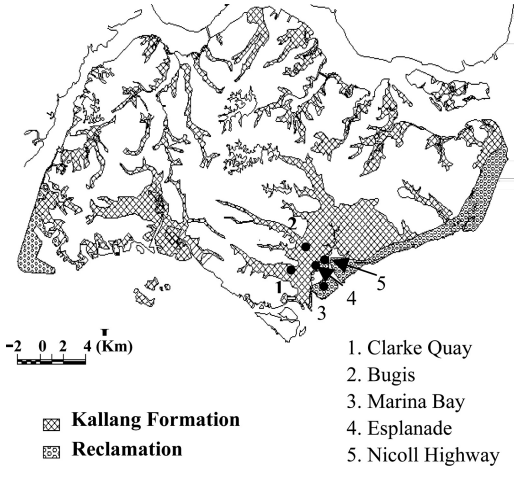

Figure 1. Singapore, showing areas of Kallang Formation deposits and recent reclamations.

The rapid development of Singapore over the last 40 years has required the construction of many deep excavations, for mass transit, underground roads and commercial and residential space. Many of these excavations have been in areas of deep marine clay. In the older urban areas, the large inward movement of the excavation support walls and the consequent settlement of the traditional Singapore 'shophouses', founded on shallow foundations or short timber piles, have been of particular concern. In the newly reclaimed areas the depth and very low strength of the clay have posed particular problems.

Some of the innovative solutions that have been successfully adopted in the construction of deep excavations in the marine clay will be described, using brief

case studies of completed excavations. In addition to the many successful excavations, there have been a number of failures of excavations, and the details of some of these will also be given.

2 THE MARINE CLAY

The marine clay is a unit of the Kallang formation. The Kallang Formation is a recent deposit consisting of soil of marine, alluvial, littoral and estuarine origins and covers about 25% of Singapore Island (PWD, 1976). Marine clay is the main constituent of this formation. In some instances, particularly under reclaimed areas, it can be over 40 m in thickness. The marine clay includes a Holocene deposit, referred to as the Upper Marine Clay (UMC), and a Pleistocene deposit, known as the Lower Marine Clay (LMC). The two beds are typically separated by a stiffer intermediate layer, considered to be the desiccated crust of the lower marine clay (Tan et al., 2002). The clay fraction of the marine clays is usually high, typically more than 50%. The principal mineral is kaolinite, although at some locations smectite is also prominent.

A study on Singapore Marine Clay by Tan et al. (2002), was based on samples from two sites. One, on the main island, was for the Singapore Arts Centre (SAC), now known as the Esplanade. The other was from the sea channel off the eastern coast (PT clay). The SAC site is within the central business district of Singapore, close to many of the deep excavations that have been carried out for urban development. The PT clay is in an area well away from urban development. As there are significant differences between the clay in the two areas, this paper will use only the data from the SAC site.

The liquid limit of the clay at the SAC site was 60%–80%. The activity of SAC clay is ~0.8 which is only slightly more active than Norwegian Drammen clay, a well-known lean clay.

Using high quality samples (Tan et al., 2002) and constant strain rate test, the compression behaviour shows the presence of high compressibility immediately after the preconsolidation pressure (p'_c) before joining the normally consolidated line, an indication of some microstructure. This behaviour after p'_c can be described using two compressibility indices, namely C_{C1} the compression index immediately after p'_c and C_{C2} the usual value along the normally consolidated line well after p'_c. At the SAC site C_{c1} ranged from 0.4 to 1.8. The value of C_{c1}/C_{c2} was about 1.2 for standard oedometer tests and about 1.7 for constant strain rate tests.

The clay was found to be slightly over-consolidated, with OCR averaging 1.2 from oedometer test and 1.5 from CRS test.

The undrained shear strength ratio (c_u/σ'_{vo}) measured using UCT (Unconfined Compression) tests was

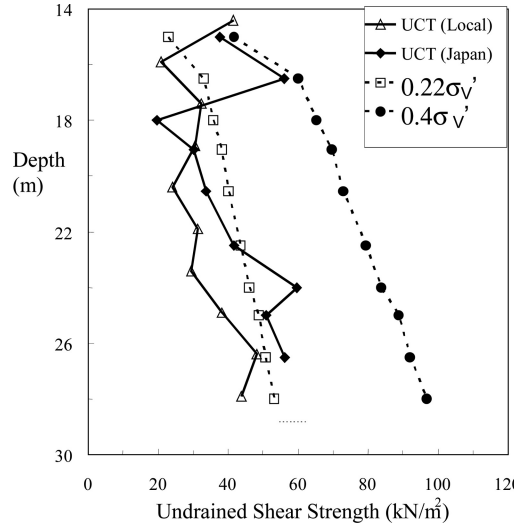

Figure 2. Effect of sampling method on undrained shear strength (UCT) for SAC clay.

about 0.22 (Figure 1). Samples tested after isotropic or Ko consolidation gave a higher ratio, of about 0.3, as did tests in extension. Data from Direct Simple Shear (DSS) tests, reported by Cao et al., gave a lower strength ratio of 0.18 (for the UMC) MC) and 0.23 (for the LMC). Typically, vane tests give an undrained shear strength ratio of about 0.25, in terms of current effective stress. This ratio takes into account the small degree of overconsolidation found in areas other than the more recent reclamations. Tan et al report that the ratio between the undrained shear strength and the maximum past effective consolidation pressure (p'_c) is 0.21, based on both DSS and field vane tests. The design of excavations is commonly based on the results of field vane tests.

Significant areas of Singapore have been formed by reclamation, often over deep deposits of marine clay. Although the SAC site studied by Tan et al. was reclaimed from the sea, both the thickness of fill placed and the thickness of the marine clay were lower than in some other areas. In areas of thick marine clay it can take 25 years or more for the clay to consolidate under the weight of the fill; many of the more recent reclamations still exhibit positive excess pore pressures in the clay. Due to the low effective stress in the clay, the strength can be significantly lower, at a given depth, than the values measured at the SAC site. A number of shear strength/depth plots used for design at various sites are summarized on Figure 3. Bugis and Clarke Quay Stations were constructed in old urban areas. The Esplanade and Marina Bay Station were built in reclamations that had been completed 23 and 10 years before excavation, respectively. A section

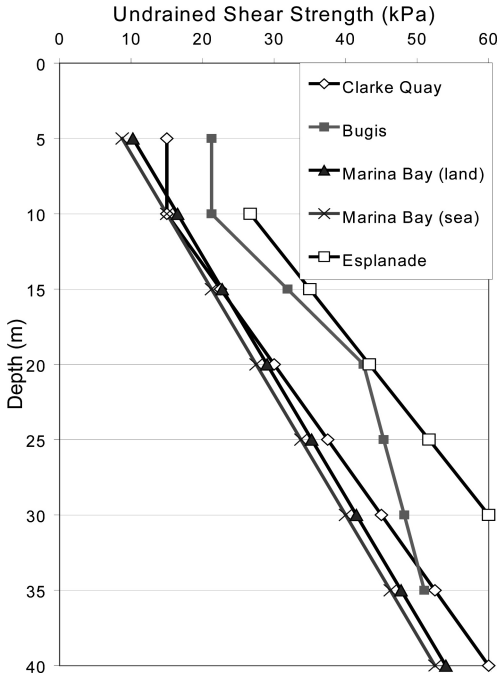

Figure 3. Undrained shear strength/depth profiles in the marine clay at various sites in Singapore, as used for design.

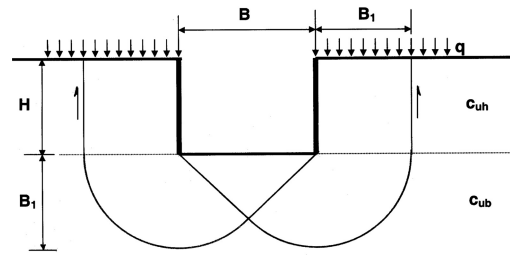

Figure 4a. Terzaghi's method (1943).

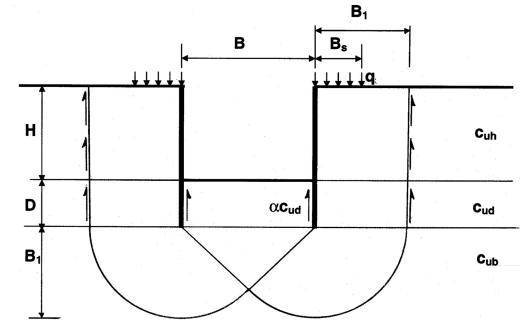

Figure 4b. Modified Terzaghi's method (Wong and Goh, 2002).

of the Marina Bay tunnels was built, by cut-and-cover methods, through a sea inlet (the Telok Ayer basin) that has subsequently been reclaimed.

The design shear strength profiles in Figure 3 are after the application of vane correction factors of 0.85 (for Bugis), 0.81 (for the Esplanade), and 1 at the other sites.

3 ISSUES FOR EXCAVATIONS

3.1 *Basal stability – undrained failure, hydraulic uplift*

For excavations into soft clay, where the soft clay extends to below the base of the excavation, there are two issues related to basal stability that have to be addressed. These are undrained basal heave and hydraulic uplift on the remaining clay below the base of the excavation.

3.1.1 *Undrained basal heave*

Where the clay below the base of the excavation has a low shear strength, and the walls do not extend to stiffer strata, then there is the potential for undrained base heave to occur (Figure 4).

There are many published methods for the assessment of basal heave stability of excavations in clay. Wong & Goh (2002) reviewed the methods proposed by Terzaghi (1943), Bjerrum & Eide (1956) and Goh (1994) and compared them with results from finite element analysis using Sage Crisp. Results indicated that Terzaghi's method would yield reasonable factors of safety for excavations involving flexible sheet-pile walls in clay but conservative results for stiff diaphragm walls. They have proposed an extension of Terzaghi's method to take into consideration the effect of wall stiffness. The modified method was validated against the finite element results.

Terzaghi's method (1943) catered for wide excavations with the width (B) greater than the depth (H). The failure mechanism is shown in Figure 4a. The failure surface extends from the ground surface to a depth of 0.7B below the formation level or to the top of the underlying hard stratum whichever is shallower. Wall penetration beyond the formation level is ignored. The effect of wall stiffness is not considered. Even though it is developed under plane strain condition, it is applicable to wide excavations of rectangular shape. The factor of safety F_s can be computed using Equation (1).

$$F_S = \frac{5.7\, c_{ub} B_1}{\gamma H B_1 - c_{uh} H} \qquad (1)$$

where c_{uh} is the average undrained shear strength above the formation level; c_{ub} is the average undrained

shear strength within the failure zone below the formation level; γ is the average bulk unit weight of the soil above the formation level; H is the maximum excavation depth; and B_1 is equal to 0.7B or depth to hard stratum below formation level (T) whichever is smaller.

This equation is expressed in terms of soil resistance over the net driving force. An alternate approach (Chang, 2000) is to keep all resisting forces in the nominator and all driving forces in the denominator as shown in Equation (2).

$$F_S = \frac{5.7\, c_{ub} B_1 + c_{uh} H}{\gamma H B_1} \qquad (2)$$

Results of the finite element analysis are in excellent agreement with those obtained from Eq. 1 and 2. Eq. 2 has no noticeable improvement over the original formulation (Eq. 1). Bjerrum & Eide (1956) pointed out that Terzhagi's method is reliable for shallow excavations ($B \geq H$) in a homogenous soil. The results may be unreliable when the surface clay has a stiff, desiccated, crust or when the excavation is narrow (i.e. $B < H$). Mana & Clough (1981) reported cases where excavations were successfully completed even when the calculated factor of safety was below unity. These field observations imply that Terzaghi's method may yield conservative results in some situations.

When a very stiff wall is used, the clay has to flow around the toe into the excavated area. Wong & Goh (2002) extended Terzaghi's method to accommodate this phenomenon as shown in Figure 4. The extension is based on Equation (2) where all the resisting forces are on the nominator and the driving forces at the denominator.

$$F_S = \frac{5.7\, c_{ub} B_1 + c_{uh} H + (1 + \alpha)\, c_{ud} D}{\gamma H B_1 + q B_s} \qquad (3)$$

where α is the adhesion factor; $B_1 = 0.7B$ or (T-D) whichever is smaller; T is clay thickness below formation level; and B_s is the width of surcharge loading where $B_s \leq B_1$. Two examples were used to verify the reliability of this method. The computed factors of safety for are summarized in Table 1. The computed F_s values using the modified method are in excellent agreement with those obtained from the finite element analyses. However, this method is only valid if the wall has enough strength to resist the potentially large forces imposed on it.

Due to the low undrained shear strength of the marine clay in Singapore, excavations where the retaining walls do not penetrate below the base of the excavation would typically fail by base heave at a total excavation depth of about 6 m, based on the shear strength typically found in the older urban areas.

Table 1. Factors of safety for 'rigid' walls in clay.

Case	Terzaghi Eq. 1	Modified Terzaghi Eq. 3
1	0.81	0.96
2	0.76	0.95

3.1.2 *Hydraulic basal stability*

The marine clay is typically underlain by fluvial sand (Bird et al., 2003), and then by Old Alluvium (Chiam et al., 2003) or the weathered rocks of the Jurong or Bukit Timah Granite Formations. These materials can be significantly more permeable than the overlying marine clay, so consideration has to be given to the potential for hydraulic uplift. This will occur if the water pressure in the more permeable layer is higher than the weight of the remaining soil in the passive zone of the excavation. Pressure relief wells have been used in Singapore to control this problem.

3.2 *The 'net active' pressure problem*

One way of avoiding undrained basal instability is to lengthen the supporting walls for the excavation, providing adequate penetration into a competent bearing stratum. However, for excavations in areas of deep soft clays, the wall deflection and induced bending moment can be exceptionally high. This is due to the 'net active' pressure problem, as described in Davies & Walsh (1984). 'Net active' pressure occurs when the active pressure on the wall exceeds the limiting passive pressure in the zone below the base of the excavation. This occurs once the stability number at the base of the excavation exceeds the critical stability number (Karlsrud 1986).

The undrained shear strength/depth profile for Clarke Quay (Fig. 3) is typical of those commonly used for design in Singapore, in areas other than those that have been recently reclaimed. 'Recent' in this context refers to areas that have been reclaimed in the last 25 years; due to the thickness and low permeability of the clay, primary consolidation generally takes 25 years or more to take place.

Figure 5 shows the calculated net active pressure (the active pressure minus the limiting passive pressure) below the base of a long, 18 m deep excavation, for the Clarke Quay strength/depth profile, assuming marine clay to a depth of 40 m. The calculation is based on:

$$\text{Net Active Pressure} = \gamma H + q - N_{cb}\, c_u \qquad (4)$$

Where q is the design surcharge and the other terms are defined above. It can be seen that there is a net active pressure on the wall almost throughout the marine clay, even below the base of the excavation. In the passive zone the wall has to transfer this load, upwards

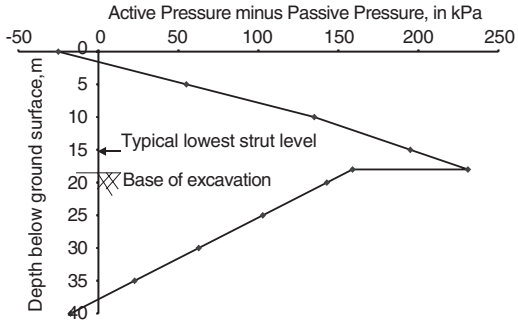

Figure 5. The 'net active' pressure on the wall of an 18 m deep excavation. The undrained strength profile used is that for Clarke Quay in Figure 3.

to the struts and downwards into the underlying hard stratum. In the example given the wall is effectively unsupported between the lowest strut level and the hard stratum, a span of 25 m. It is this large unsupported span that leads to the large deflections that are discussed below.

3.3 *Underdrainage during excavation*

Deep marine clays in Singapore are typically underlain by fluvial sands, and then by the Old Alluvium or the weathered sedimentary rocks of the Jurong Formation. Typically, these underlying units have a bulk permeability that is two to four orders of magnitude higher than that of the marine clay.

Stress relief during excavation results in large negative excess pore pressures developing in the marine clay in the passive zone. These reduced pore pressures are then transmitted outside the excavation through the more permeable underlying units. The confined nature of these aquifers and the high horizontal permeability of the fluvial sands can transmit the reduction in pore pressure for hundreds of metres from the excavation. A reduction in pore pressures in the fluvial sands outside the excavation is followed by underdrainage of the overlying marine clay, and resulting consolidation settlements. The process is driven by stress relief, not by active groundwater control, and can continue for months after the base slab has been completed. Shirlaw & Wen (1999) and Wen & Lin (2002) provide examples of this problem.

4 PRACTICAL IMPLICATIONS FOR EXCAVATIONS IN SINGAPORE

As discussed in section 2, the marine clay is generally, apart from the PT area, close to normally consolidated, or, in areas of post war reclamation, still consolidating. The shear strength is low, and consequently the factor of safety against basal heave, even for relatively shallow excavations, is low. Net active pressures develop for even quite shallow excavations. Very large lateral movements have been experienced with sheetpiled excavations. In Table 2, a number of cases of sheetpile supported excavations are summarised. The excavations listed were inland from the current coastline, and not in the more difficult recent reclamation areas, where the clay is weaker (as it is still consolidating) and thicker than at these sites. The depth to the base of the marine clay at these four sites was 14.7 to 24 m. It can be seen that, despite the relatively shallow excavations, the wall deflections were large in all four cases.

Table 2. Sheetpile wall deflections at four sites in Singapore.

Name	Exc. Depth (m)	Wall type	Levels of struts	Max. Wall Deflection (mm)
MOE	7	YSP IV	4	315
Rochor Complex	6.3	FSP IIIA	3	150
CTC Building	11	FSP IIIA	6	230
Novena Station	15.5	FSP IV	6	270

The maximum deflection of the wall at these four sites occurred below the base of the excavation. This shows that the large deflections measured were influenced by the 'net active' pressure that develops once the excavations exceed about 6 m in depth.

As discussed above, excavation induced stress relief results in large pore pressure changes, which in turn results in underdrainage of the marine clay outside the excavation. Consolidation settlements of 100 mm or more have been recorded during the construction of Newton Station (Nicholson 1987), Bugis Station (Shirlaw & Wen, 1999) and Farrer Park cut-and-cover tunnels (Wen & Lin, 2002).

The rapid development of Singapore has led to the need for some very deep excavations in the marine clay, often in areas where movement has to be controlled to low levels, due to adjacent structures. This has resulted in some innovative excavation systems being used. Most of these have been very successful, but, unfortunately, there have also been a number of failures. The following sections describe, first, some of the successes, and then some of the failures.

5 INNOVATIVE EXCAVATION SYSTEMS

5.1 *Diaphragm walls and lime columns (Bugis)*

Bugis Station was constructed between 1986 and 1989, as part of Phase 2 of the subway system in Singapore. The station excavation was 18 m deep. The depth from ground surface to the base of the marine clay varied along the station from 17 m to 37 m. Underlying the

marine clay was a thin bed of fluvial deposits and then the Old Alluvium (Fig. 6).

The excavation was carried out in an old urban area of Singapore, near Bugis Steet. It was specified that diaphragm walls should be used. 1.0 and 1.2 m thick walls were constructed, up to 54 m in depth, depending on the thickness of the marine clay. Up to 7 levels of strutting were required to support the walls for the 18 m deep excavation. In addition, lime columns were used to improve the soil at and immediately below final excavation level (Kado et al., 1987). Although the lime columns increased the average strength of the soil at formation level, by 60% to 100%, the installation of the columns, using a mandrel, caused some reduction in the strength of the clay above the treated area.

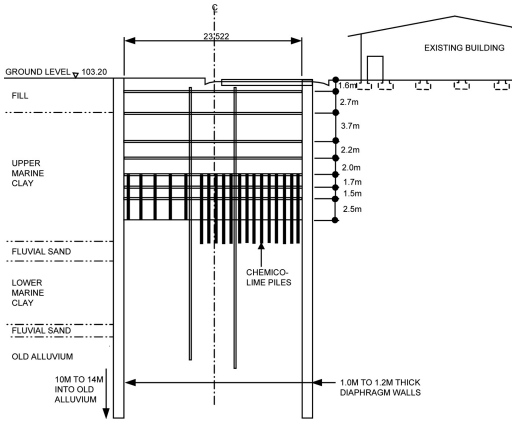

Figure 6. The excavation support system for Bugis Station.

The lateral movement of the diaphragm wall when the excavation was at final excavation level was 150 mm, in the section where the marine clay was thickest. Total settlement was up to 250 mm, of which about 100 mm was consolidation settlement.

The excavation resulted in up to 8.2 m of pore pressure reduction in the Old Alluvium outside the excavation, which caused underdrainage of the marine clay. Despite the installation of 11 recharge wells, the consolidation settlement was still continuing 9 months after the completion of the base slab. No pressure relief wells or deep pumping were carried out during the excavation.

5.2 *Underwater excavation (Marina Bay)*

Marina Bay Station was built between 1986 and 1989 in an area that had been reclaimed from the sea about 10 years before. The tunnels connecting the station to Raffles Place station were built across the Telok Ayer basin (Fig. 7), a tidal basin connected to the sea. The station and tunnels across the basin were constructed using underwater excavation (Clarke & Prebaharan, 1987). A substantial retaining wall consisting of 610 × 305 mm × 149 kg I sections welded to BXN sheetpiles,

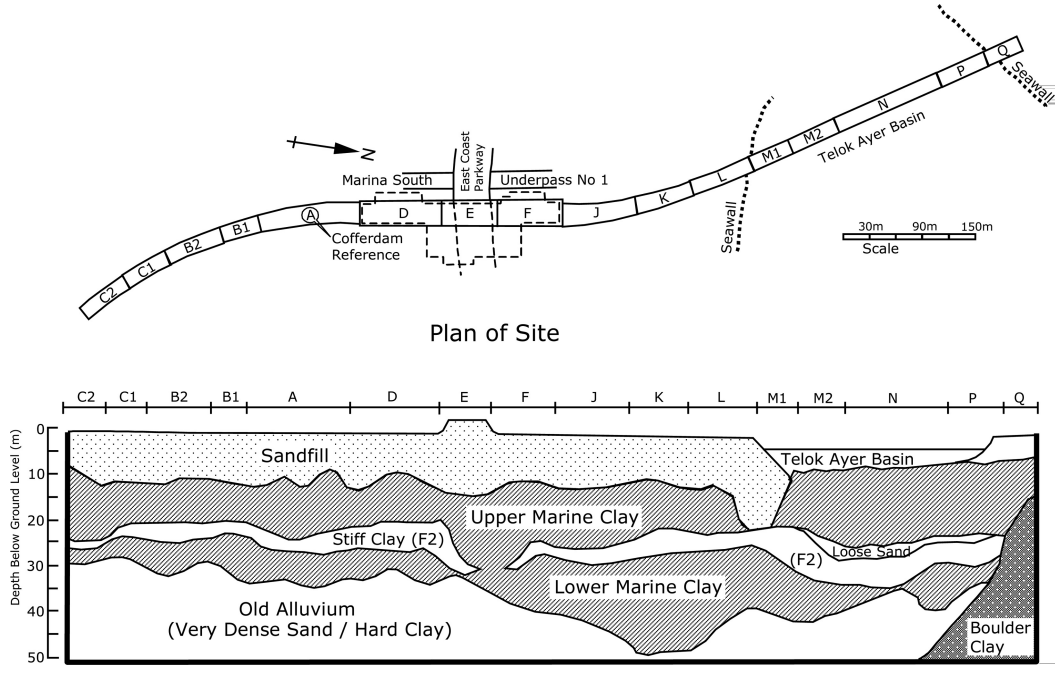

Figure 7. Marina Bay Station and tunnels (after Clarke & Prebaharan, 1987).

was installed. The walls were not taken right through the marine clay, but were stopped at a depth of 23 m to 30 m below ground level, in the stiff clay layer intermediate between the UMC and the LMC.

The excavation sequence is shown in Figure 8. Excavation to a depth of 6 m was carried out in the dry, and two levels of struts were installed. The rest of the excavation, to a depth of about 19 m, was carried out underwater. The water in the cofferdam was maintained at 1 m above ground level in the reclamation areas. The total weight of water within the cofferdam at final excavation level was calculated to be slightly greater than the total weight of the soil excavated after the initial, dry, excavation stage. As a result the deflection of the wall, 100 to 200 mm at the end of the dry excavation stage, hardly changed during the underwater excavation.

The permanent bored piles, required to support the station, were then installed. The piles obtained bearing in the Old Alluvium, and were designed for both compression and tension. A 1.5 m thick mass concrete slab was then tremied into the base of the excavation. Shear connectors were provided between the permanent steel casings of the piles and the tremie slab. The tremie slab was designed to provide lateral restraint to the walls and to resist base heave pressure, tied down by the permanent piles, following dewatering.

The base heave pressure was calculated using the formula:

$$P = q + \gamma H - F_1 N c_U - \frac{F_2 \pi d (h_1 + h_2 + h_3) c_U}{LB} \qquad (5)$$

Where:
q is the surcharge
γ is the average bulk density of the soil above the level of the base of the excavation
F_1 and F_2 are strain compatibility factors
N is the critical stability number for base heave (Bjerrum & Eide, 1956)
c_U is the undrained shear strength of the clay
h, h_1, h_2 and h_3 are as defined in Figure 9, and depend on the critical failure surface for the particular ground conditions and excavation geometry at the site
L is the spacing of the piles along the excavation
B the excavation width.

The tremie slab was unreinforced, but had to be able to withstand bending, due to the uplift force, between the piles. The bending capacity of the slab was derived from the compressive force exerted on the slab by the retaining walls.

Four pressure cells were installed below the base of the excavation. The calculated and measured total uplift pressure on the centre of the slab is given in Table 3.

In all fifteen, linked, rectangular cells, 11 in the reclamation and 4 in the Telok Ayer basin, were successfully constructed in this manner.

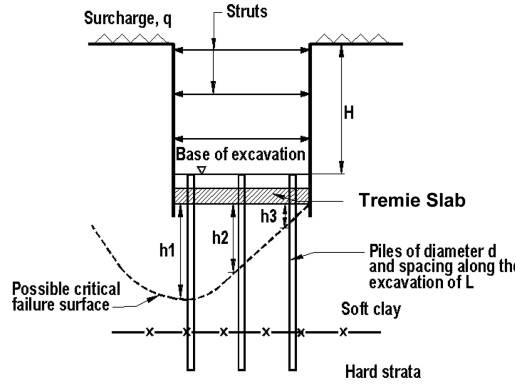

Figure 9. Basis for the estimation of uplift pressures on the tremie slab, Marina Bay Station.

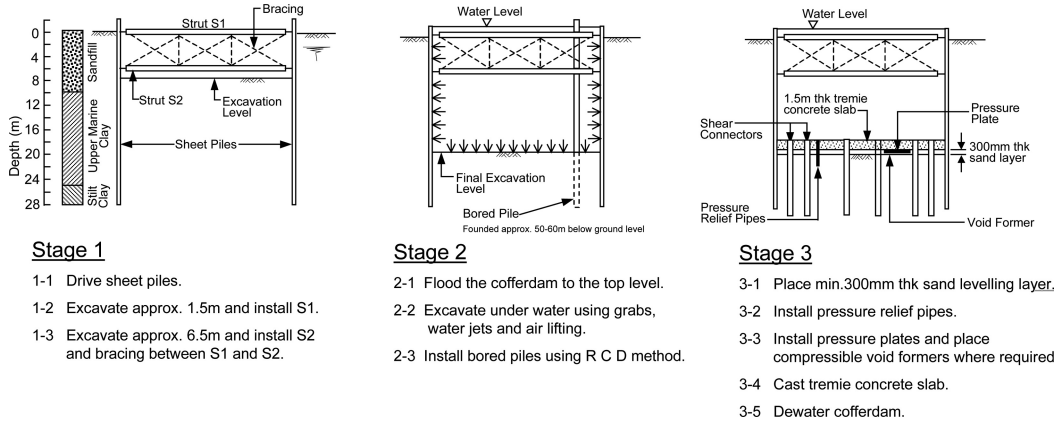

Figure 8. Excavation sequence for Marina Bay Station and tunnels.

Table 3. Predicted and measured uplift on the tremie slab, Marina Bay (after Clark & Prebaharan, 1987).

Cofferdam	Predicted uplift (KN/m^2)	Measured uplift (KN/m^2)
A	85	78, 78
F	83	63, 60, 57, 42
J	83	100, 70
M2	72	62, 57

5.3 *Diaphragm walls and jet grout slabs (Little India)*

The cut-and-cover tunnels between Little India and Farrer Park Stations formed part of the North East line of Singapore's MRT system. The total depth to the base of the marine clay was between 18 and 35 m, while the depth of excavation was 17.5 m. The retaining system consisted of 0.8 m thick diaphragm walls with 6 levels of struts.

In order to reduce the deflection of the wall, and therefore the settlement to adjoining properties, a jet grouted slab was formed at the base of the excavation. This was done only in those areas where the adjacent buildings were particularly sensitive to settlement. Sections of slab were either 1.5 m or 2 m in thickness, and the slab spanned between the two diaphragm walls (Fig. 10).

A comparison has been made of the measured deflections with those for other excavations, using diaphragm walls, where no jet grout slab was used (Fig. 11). It can be seen that the slab had little benefit when the hard stratum was close to the base of the excavation. Much greater benefit was obtained, in terms of reduced wall movement, when the hard stratum was deeper.

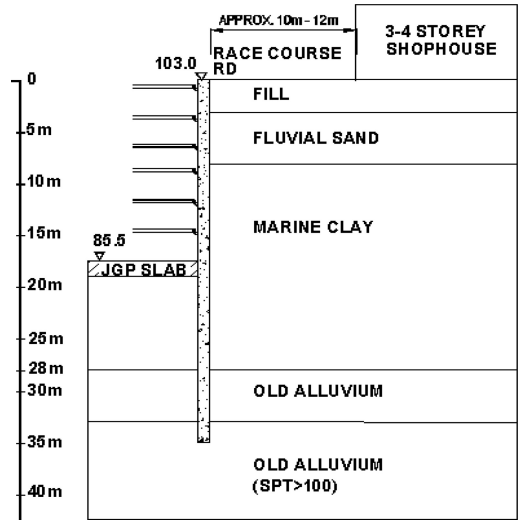

Figure 10. Jet grout slab used as a buried strut, cut-and-cover tunnels along Race Course Road.

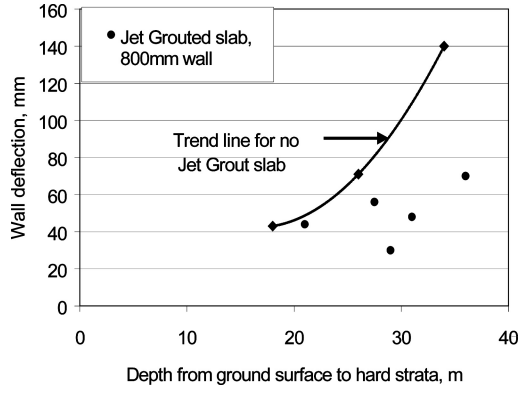

Figure 11. A comparison between the recorded deflections for 17.5 to 18 m deep excavations, with and without a single jet grout slab just below the base of the excavation.

5.4 *Floating sheetpiles and piled jet grout slabs (Clarke Quay, Kallang/Paya Lebar Expressway)*

Another excavation for the North East Line was that for Clarke Quay Station. The marine clay in this area was underlain by weathered rocks of the Jurong Formation; the depth to the weathered rock from ground surface was 20 m to 27 m. The main station box was excavated using strutted diaphragm walls taken into the Jurong Formation. However, two of the entrances were constructed using a support system that combined many of the features of the Marina Bay excavation with the jet grouted slab, as used at the Little India to Farrer Park tunnels.

The jet grouted slab was used in place of the tremie slab, and designed in a similar way. The sheetpiles for the 9 m deep excavation were driven to a depth of 12 m, so the excavation and support system 'floated' in the marine clay. The excavation was carried out in the dry, using 3 levels of strutting as well as the jet grout slab (Fig. 12). The design of the jet grout slab was based on equation 5, previously used of the tremie slab at Marina Bay. Measurements of the heave of the slab during excavation gave a value of only 3 mm. Further details of the design and construction issues for jet grouted slabs are given in Shirlaw et al (2000a), Shirlaw et al. (2000b) and Shirlaw (2003).

Similar 'floating' retaining systems, but for deeper excavations, have since been used for some of the excavations for the Kallang and Paya Lebar Expressway, which commenced in 2002. One example is shown in Figure 13. The excavation was much larger than those

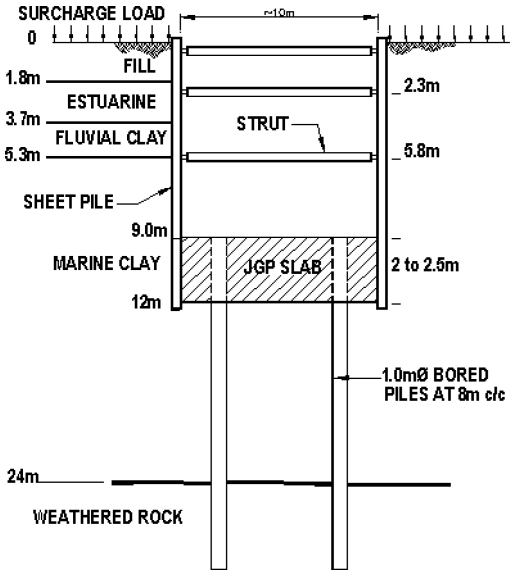

Figure 12. The 'floating' retaining system, Entrances 1 and 2, Clarke Quay Station.

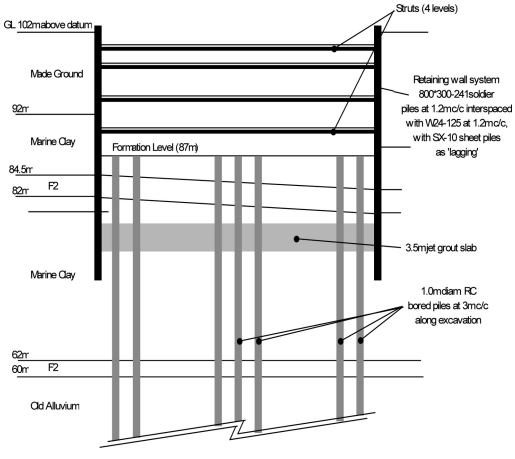

Figure 13. An example of a 'floating' wall system, used for part of the KPE construction.

of the entrances at Clarke Quay. The width of the excavation was 40 m and the depth 15 m. In this case the monitored heave on the jet grouted slab, at the end of excavation, was 47 mm. The position of the jet grout slab was chosen to ensure that the grouting was in the marine clay and not in the stiff intermediate (F2) clay. The column diameter that could be achieved in the stiff clay was significantly smaller than that in the marine clay. In order to form a slab by installing multiple, overlapping columns, it was considered advantageous to ensure that the slab was in the marine clay.

Jet grouted slabs have been in use as part of excavation support systems in Singapore for about 20 years. In numerical models the jet grout is generally treated ground as a strong, perfectly elastic/perfectly plastic soil of considerable strength. However, as discussed in Shirlaw (2002a), it is advisable to also assess the jet grout slab as a weak structure. Strain compatibility between the soft clay and the jet grout, and the potential for a brittle 'post peak' behaviour of the jet grout also need to be considered. Once the jet grout slab is considered as a weak structure, it is apparent that the interfaces between the jet grout slab and the walls and piles are critical to the effectiveness of the slab. These interfaces are, in turn, dependent on the layout and size adopted for the jet grout columns. The designer of the jet grout slab therefore needs to be involved in the details of the jet grouting, to check that the construction practice is going to provide the resistance assumed in the design.

5.5 *Diaphragm walls, piled jet grout slabs and cellular excavation (Esplanade)*

The Esplanade – Theatres on the Bay is touted to be the landmark of Singapore with the two unique domes as shown in Figure 14. It is a world-class cultural and arts centre. The 1,600-seat concert hall boasts the biggest reverberation chamber in the world and the 2,000-seat theatre has an adjustable proscenium arch and orchestra pit with two full-sized ancillary stages. The arts centre covers an area of about 18,000 m^2 and has the shape of a segmented semi-circle with a radius of about 90 m (Fig. 15).

There are no floor slabs at the ground and first basement levels. The aim was to create a large opening spanning across the entire site with a height clearance of about 10 m to accommodate a theatre and a concert hall. A 1 m thick diaphragm wall was constructed along the perimeter of the semi-circular arch. The wall was supported by a series of buttress walls at 8 to 10 m intervals perpendicular to the wall. The idea was to transfer the soil pressure behind the wall onto the slab at the second basement level and then down to the bored piles supporting the slab. The design configuration is shown in Figure 16 (Wong et al., 2002). The site is located along the sea front near the central business area. The 2 hectares site was reclaimed in 1975.

The ground water level was about 2.5 to 3.0 m below surface. The soil condition was very variable across the site. The soil profile along the perimeter is shown in Figure 17. The fill consisted mostly of gravelly sand to clayey and silty sand with thickness varying from 4 to 15 m. The average standard penetration blow count (SPT 'N') was about 10. The thickness of upper marine clay varied from 0 to 5 m and the lower marine clay had a thickness up to 20 m near the southwest corner. Marine clay was absent at the northwest corner. The

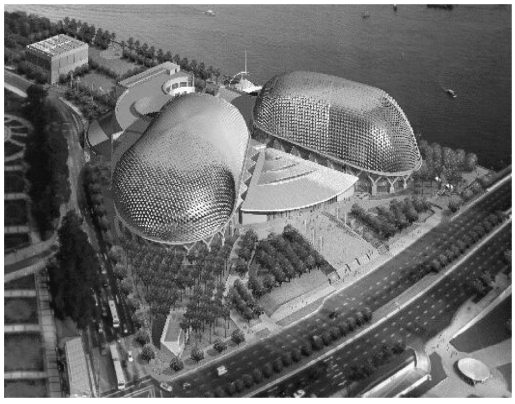

Figure 14. Artist's impression of the completed project.

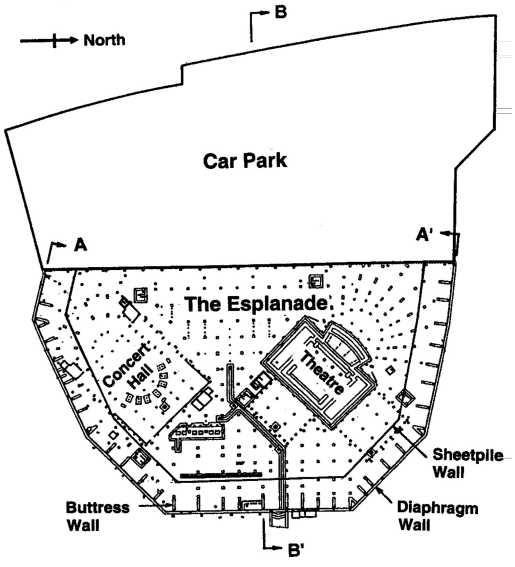

Figure 15. Site plan and layout of car park and centre proper.

clay has a liquid limit of around 75 and a plasticity index of 45 existing fill. The measured c_u/p' ratio was about 0.25. Underlying the Kallang Formation was the Old Alluvium Formation, which consisted mainly of silty sand and sandy silt. The depth to hard stratum, where SPT 'N' $\geq$ 100, varied from 27 to 35 m below ground surface.

Results of consolidation tests indicated that the clay was essentially fully consolidated under the weight of the fill.

The final design took advantage of the semi-circular shape by constructing a 16 m wide concrete slab around the inner perimeter of the site butting against the diaphragm wall at formation level. It was intended

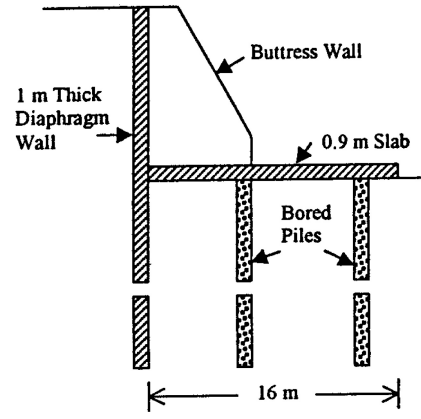

Figure 16. Configuration of permanent wall.

for this concrete slab to act as a compression ring to keep the diaphragm wall in-place as illustrated in Figure 18.

In order to construct the 16 m wide concrete slab around the perimeter at formation level, a trench 18 m wide and 10 m deep was dug around the perimeter. A row of temporary sheetpile wall was installed at 18 m parallel to the diaphragm wall as shown in Figure 5. Results of the finite element analysis indicated that the maximum wall deflection due to trench excavation alone would exceed the allowable limit of 75 mm. In order to reduce the wall deflection, jet grouting was considered. With the presence of a 2 m thick jet grout slab, the computed deflection reduced from 82 to 38 mm. Further analyses were carried out to optimize the design of the diaphragm wall in terms of: (i) extend of jet grouting; (ii) wall length and (iii) wall bending moment. These analyses resulted in several wall types (I, II, IIA, IIB and IV) as shown in Figure 18.

The buttress wall was designed to provide permanent support to the diaphragm wall to resist the earth pressure. The buttresses were 1 m thick and 6 m wide, and spaced at 8 to 10 m. A continuous waler beam connected all the buttresses at mid-height of wall. The buttresses acted partly as a wall stiffener and partly as a medium to transfer the soil pressure to the base slab and then onto the supporting bored piles as illustrated in Figure 16. The construction sequence is summarised below:

Stage 1: Construct 1 m-thick diaphragm wall around the perimeter and install the sheetpile wall 18 m away from the diaphragm wall.

Stage 2: Install a 2 m-thick jet grout slab between the diaphragm and sheetpile walls below the formation level. Because of favourable soil conditions at the northeast corner, a small section at that area was not treated. The jet

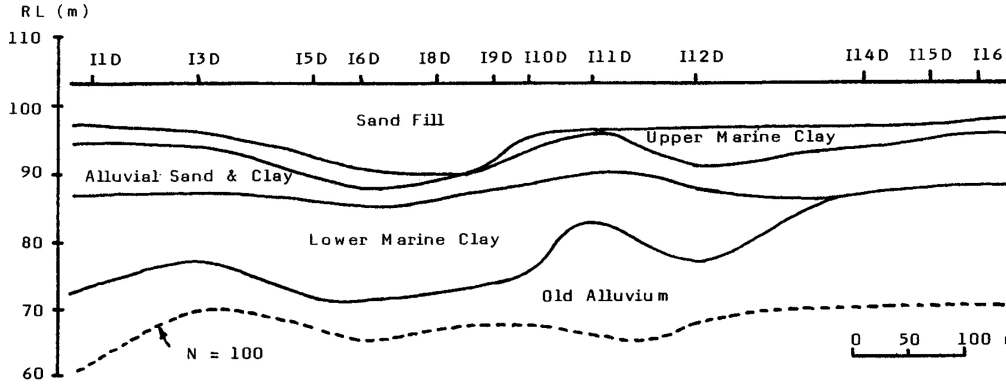

Figure 17. Soil profile along the perimeter of excavation from south to north (Section A-A′).

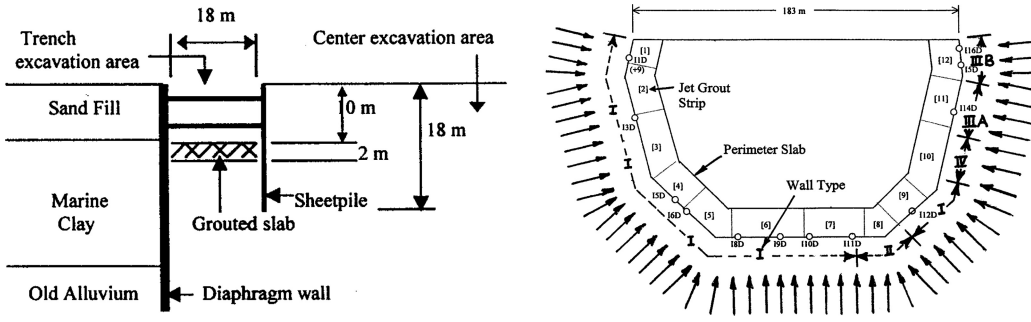

Figure 18. Perimeter concrete slab (16 m wide and 0.9 m thick) at formation level acting as a compression ring.

grouting work was completed by mid-June 1997.

Stage 3: Install bored piles.

Stage 4: Excavate the soil between diaphragm and sheetpile walls to formation level at Reduced Level (RL) 92.5 m. Struts were installed at RL 101.3 m and 97.0 m. The trench excavation was completed by early November 1997.

Stage 5: Excavate pits for pile cap construction.

Stage 6: Construct concrete slab (16 m wide and 900 mm thick) and buttress walls. At the interface between car park and art centre proper, the slab was connected to the lower basement slab of car park. The construction of the buttress walls was completed by the end of 1997.

Stage 7: Excavate the central area bounded by the sheetpile wall uniformly. Remove the struts as excavation level reached below the strut level.

Stage 8: Excavate to formation level and remove the sheetpile. Excavation work in the central area started in early January 1998 and completed in April 1998.

This project was heavily instrumented. The wall deflections were monitored using in-wall and in-soil inclinometers around the perimeter. Earth pressure cells were installed in two diaphragm wall panels to measure the earth pressure acting on both sides of the wall. In-soil and in-pile inclinometers were installed in the central area to monitor the pile and soil movements during excavation. Piezometers and water standpipes were installed to monitor the changes in pore pressure and ground water table respectively. Strain gauges were mounted on the perimeter slab and the buttress walls to measure the compression forces in these elements. Settlement markers and survey points were installed on the ground surface to monitor the ground movement.

At the end of trench excavation, the maximum inward wall deflection varied from 20 to 33 mm. The displacement increased to 27 to 50 mm during the construction of pile caps and buttresses. Since the wall was pushed outward during jet grouting, the net wall deflection was relatively small at this stage.

During excavation in the central area, the ground surface was lowered ‘uniformly’ downward. Strict control was exercised to keep the difference in ground level not to exceed 1 m during excavation. The aim

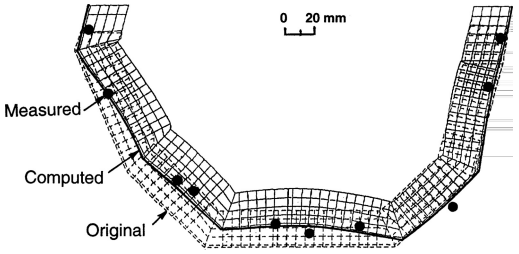

Figure 19. Computed and measured deflections.

was to minimize movement of the installed piles. The maximum wall deflection due to excavation in the central area alone varied from −5 mm (outward) to +25 mm (inward). When combining with deflections during trench excavation, the total deflection after jet grouting varied from 37 to 65 mm. Due to further excavation work at the lyric theatre and local pile caps, there was an increase in movement by about 3 to 5 mm. If the wall movements due to jet grouting were taken into consideration, the final wall deflection from the original vertical position varied from 29 to 70 mm.

Another item of interest is the performance of the perimeter slab acting as a compression ring. The computed and measured horizontal displacements of the perimeter slab are shown in Figure 6. The measured maximum displacements at the crown varied from 10 to 17 mm and the computed displacement was 15.6 mm. The overall deformation patterns are in good agreement.

5.6 *Secant piles and DSM berms (Toa Payoh)*

For the construction of a 3-level basement car park for the HDB Centre at Toa Payoh, a 12 m deep excavation running approximately 150 m parallel to a Rapid Transit Structure (RTS) was needed (Tan et al., 2001). A very stringent limit of not more than 15mm was imposed on the resultant movement allowed for the RTS structure. There were also two key features in this project which require special attention, namely the wide span of the excavation, approximately 180 m, making it difficult to install horizontal braced struts and that the excavation will be in an area with marine and organic clay. As a solution, inclined struts were used. To facilitate this, the ground some distance away from the retaining wall was to be excavated first, and a soil berm was left to restrain the inwards movement of the retaining wall. However, initial design analyses suggest that this would mean almost no movement could be tolerated during the installation of the diaphragm wall, clearly not a satisfactory solution.

To reduce the likely movement during the excavation to provide some additional room for the construction of the diaphragm wall, the berm was improved using lime columns to provide adequate support to the

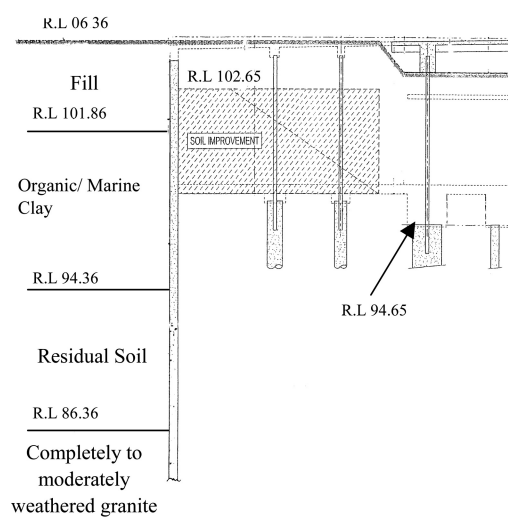

Figure 20. Use of deep soil mixing to provide an improved berm, HDB Centre at Toa Payoh.

retaining wall to limit movement (Fig. 20). The existing soil needs to be improved only a little. The main chemical improving agent used is quick lime (CaO), which absorbs water from the surrounding causing the lime to swell and formed slaked lime ($Ca(OH)_2$). According to Tan et al. (2001), the process of slaking leads to a volume expansion which in turn causes a lateral consolidation effect on the soil adjacent to the lime columns, and moisture content in the soil is reduced. As a longer term chemical effect, the calcium ion in the slaked lime are then absorbed by the negatively charged surface of clay minerals, thereby acting as the cementing agent leading to the improved strength of the soil. The shear strength of the original soil is around 15 kPa. The shear strength of the individual pile is about 150 kPa and the shear strength of the composite treated soil is improved to 20 to 25 kPa. More importantly, the Young's Modulus of the composite soil was improved to 8 MPa, significantly higher than the value of the original unimproved marine clay, which will be about 1 to 2 MPa. However, such improvement is considered low compared to more conventional soil improvement technique such as jet grouting mainly because of the very wide spacing between columns. The measurement during construction indicated that the final resultant movement was just over 10 mm. This indicates that the choice of a low intensity improvement was just right for this job.

6 FAILURES

Although there have been many successful excavations in Singapore, failures do occur. A Workshop on

'Avoiding Failures in Excavation Works', was organised by the Building Control Authority (BCA) in 2003. Included in the workshop were 13 brief case studies of excavation failures that had occurred over the previous 10 years. All of the cases involved relatively shallow excavations, from 2 m to 8.5 m in depth. The majority involved excavations in marine or estuarine clays of the Kallang Formation. The cases involved either outright collapse of part of the retaining system (ULS failures), or movements that caused severe damage to adjacent structures (SLS failures). In many cases, the adjacent buildings were so badly damaged as to require demolition and rebuilding. The lateral deflection of the retaining walls ranged from 200 mm to complete collapse. In several cases insufficient consideration of hydraulic pressures was blamed for the failure.

One of the cases cited in the BCA seminar is discussed in outline, in 6.2 below. Two other major collapses, not included in the BCA workshop, are also discussed.

6.1 *Central Services Tunnel*

The Central Services Tunnel (CST) was built by cut-and cover methods through the Telok Ayer basin (shown in Figure 7). The area was reclaimed from the sea following the construction of the tunnels from Marina Bay Station.

The reclamation took place in about 1992, 10 years before construction of the CST. Under 8.5 m of fill there was 17m of upper marine clay, 7 m of fluvial deposits, 3 m of lower marine deposits and then Old Alluvium (Fig. 21).

The 16 m deep excavation was constructed within a combined sheetpile/solider pile retaining wall. The retaining wall was taken to 3 m below final excavation level, in the upper marine clay. To provide basal stability, a 2.5 m thick slab was to be constructed at the base of the excavation. The jet grout slab was tied down by two rows of king posts, driven into the Old Alluvium. The king posts also provided restraint against buckling for the four levels of strutting.

The failure occurred during excavation for the fourth level of strutting (Lim & Tan 2003). The jet grout slab suffered a failure in bending along the centre of the excavation over a length of about 50 m. Buckling of the jet grout slab was accompanied by the king posts punching upwards, effectively taking out the strutting system, and accompanied by inward rotation of one wall of the excavation. Photographs taken after the failure are shown in Figures 22 and 23.

Other areas that had been designed on a similar basis were successfully excavated. According to Lim & Tan (2003), the major differences between the failed area and the other, successfully, completed areas were:

1. There was a stockpile of soil adjacent to this section of the excavation, amounting to about six times the design surcharge of 20 kPa.

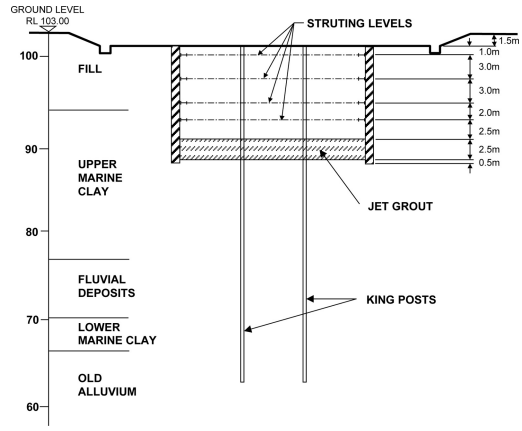

Figure 21. The excavation for the Central Services Tunnel (after Lim & Tan (2003)).

Figure 22. The failed section of the excavation for the Central Services Tunnel, showing the failed base just above the jet grout slab and the north wall rotated inwards.

Figure 23. The failed excavation during recovery. Note the kingposts in the failed section, some of which had been pushed upwards by 3 m, and the rotational failure of the retaining wall.

2. The design precut of 1.5 m, was not carried out on the side of the excavation with the large surcharge.
3. The jet grout slab was constructed to a thickness of 2 m, rather than the design 2.5 m.
4. The king posts were driven to a set, rather than the design penetration in Old Alluvium. As a result, the king posts stopped just at the top of the Old Alluvium, and therefore did not have the capacity to resist the uplift on the slab.

The combination of a massive additional surcharge, and the failure to construct the jet grout slab and to tie it down as required in the design, provide a ready explanation for this failure.

6.2 *Lorong Limau*

This failure occurred during the construction of the underground parking levels for a residential condominium. The planned 8.1 m deep excavation was in 10.5 m of recent deposits, mainly marine clay. The excavation was supported by a combined sheetpile/H-pile retaining wall, and three levels of struts. Due to the shape of the excavation, there were a large number of corners, where the wall was supported by angled struts. At one of these corners, there was no continuity of the walers around the corner. Although the walers were welded to the wall, there was insufficient resistance along the back of the wall to take the component of force along the walers (Figure 24). The return wall and the walers racked into the excavation. The struts on the main wall were therefore ineffective, and the retaining wall deflected by an estimated 600 mm+, with a similar magnitude of settlement on the road and pressurised gas main immediately adjacent to the excavation. The gas main did not rupture, and no one was hurt.

This problem, involving the axial component of force, in walers, from angled struts, is a common cause of excavation failures in Singapore, and is not limited to marine clay.

Figure 24. The walers in this photograph had moved longitudinally, allowing the wall to deflect by over 600 mm.

Figure 25. Nicoll Highway area after the collapse.

6.3 *Nicoll Highway*

On the 20th April 2004 a section of cut-and-cover tunnel near Nicoll Highway collapsed, killing four people working at the site (Fig. 25). The Committee of Inquiry into the collapse commenced in August 2004 and issued a final report on 13th May 2005. The following brief summary is based on the final report (Magnus et al., 2005), of the Committee of Inquiry and on a report in the New Civil Engineer (Mylius 2005).

At the time of the collapse, excavation was in progress to allow installation of the 10th, and final, level of struts, and was over 30 m deep (Mylius, 2005). Nine levels of struts had already been installed to support the 800 mm thick diaphragm walls. Two jet grout slabs had previously been installed. One of these jet grout slabs, placed above the ninth strut level, was sacrificial and had been removed. The other slab was positioned just below final excavation level.

Expert witnesses testifying at the court of inquiry identified problems in the design and the construction of the work. Design problems involved both the structural design and the geotechnical analysis (Mylius, 2005).

The structural design problems related to the design of the struts and the strut/waler connections. Omission, on the drawings, of splays at the end of the struts (which were assumed in the design calculations), and errors in the design of the strut/waler connection resulted in a strutting system which had about half of the ultimate design capacity that it was supposed to have. This was compounded by the substitution of 'C' channel for

plate stiffeners in the level 7 to 9 walers. This substitution resulted in a system that failed in a brittle, rather than ductile, manner. The final failure was initiated by buckling of the webs of the walers at the 9th level of struts.

The geotechnical analysis involved an incorrect soil model for the marine clay in the finite element analysis used for the design. The analysis used effective stress parameters with a Mohr-Coulomb model, with the material type set as 'undrained'. This was compounded by the use of a pore pressure distribution in the passive zone that was hydrostatic with respect to the base of the excavation. The results of these modelling errors were:

- The design moment capacity of the wall was about half of what it should have been
- The predicted movement of the wall was about half of what it should have been
- The jet grout slabs experienced higher strains than predicted
- The design load in the 9th level of struts was about 10% lower than it should have been. However, the total design load of the 1st to 9th levels of struts was about 20% higher than would have been required in the analysis recommended by the Committee of Inquiry.

As a result of the under-design of the wall, most of the experts considered that the wall had formed a plastic hinge by the 17th April, three days before the collapse.

In addition the Committee of Inquiry reported on many other factors. These included:

- Insufficient penetration of the diaphragm wall into the main bearing stratum (the Old Alluvium). The original design did not allow for sufficient penetration due to the incorrect modelling; several of the wall panels in the area of the collapse failed to achieve even the penetration required by the design
- 66 kV cables crossing the excavation, which resulted a weak point in the support system
- The diaphragm walls were curved in plan
- There was a deep buried channel which was not fully identified by the designers
- A poor instrumentation and monitoring system. A lack of experience and skill in many of the personnel involved was particularly mentioned
- Delayed installation of the 10th level of stuts; 8 bays were not installed at the time of the collapse
- Poor quality control
- The lower jet grout slab was weaker and thinner than assumed in the design

The report commented on problems, and a lack of clarity, in the chain of command and communication within the builder's organisation. Although the report states that there was insufficient evidence to assess the state of the remaining jet grout slab at the time of the collapse, a number of issues concerning the design and construction of jet grout slabs were covered, including recommendations on limiting strains in the jet grout.

7 CONCLUSIONS

The deep deposits of near normally or normally consolidated marine clay that occur in Singapore can present significant problems for the design and construction of deep excavations. For excavations in excess of about 6 m in depth basal stability becomes an issue, if the walls are not taken to hard strata. Where the walls are taken to hard strata, then the walls have to be designed for 'net active' conditions in the marine clay, and consideration has to be given to consolidation settlements due to under-drainage of the clay. A variety of special techniques have been used to allow safe excavation and to restrict wall movement and settlement. These techniques have included jet grouting, lime piling, underwater excavation and sequenced construction, often used in combination.

While the basic problems of carrying out deep excavations in the marine clay are well known, there have been a number of significant failures over the last decade. Many of these failures fall neatly within the four main causes of failure suggested in CIRIA Report C580 (2003). These include:

- Inadequate understanding of the geological and hydrological conditions (Several of the cases cited at the BCA seminar in 2003, Nicoll Highway)
- Poor design and construction details and poor standard of workmanship, particularly of support Systems (Central Services Tunnel, Lorong Limau and Nicoll Highway)
- Construction operations and sequences that differ from those in the design (Nicoll Highway)
- Inadequate control of site operations including excessive surcharge (Central Services Tunnel, Nicoll Highway)

The Nicoll Highway case also involved issues related to the use of numerical methods for design of deep excavations. These issues included the soil model used, and how the numerical analysis should be incorporated into the final design. Such issues are generally not covered in current codes. With the increasing use of numerical methods as part of the design process, detailed consideration needs to be given to how, and to what extent, numerical methods should be incorporated into the design of deep excavations.

Another important geotechnical issue arising from the Nicoll Highway collapse involves the design and construction of jet grouted slabs (or other forms of ground treatment), where these are a part of the excavation support system. Although these are commonly modeled as hard soil, consideration also needs to be

given to them as weak structures, and to the potential for a brittle mode of failure. The construction procedures for such slabs should be reviewed by the designer, to ensure that the resistances assumed in the design can be achieved in practice.

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