GROUNDWATER CONTROL IN DESIGN AND CONSTRUCTION OF DEEP BASEMENT EXCAVATION IN SINGAPORE

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ABSTRACT

In a highly built-up city like Singapore, newer developments revolve around constructing deep underground basements due to its limited land space. This constraint constantly challenges engineering methodologies especially in groundwater control and in the design and construction of deep basement excavation works. Most excavations below the natural ground water table will inevitably induce pore pressure reduction and drawdown in groundwater table. Some of the key factors that affect the change in water table are type of geologies, excavation support system and the excavation depth. Control of groundwater during deep excavation can also be attributed to reducing horizontal stress behind the retaining wall leading to pore pressure control, high permeability soil constituents underneath the toe of the wall and leakage through the gaps or openings. In this paper, three case studies are featured where 18m to 25m deep excavations were carried out for underground basement construction in the heart of Singapore City. How the control of groundwater was considered in design and implemented in construction methodologies of deep basement excavations is presented and groundwater behavior observed during the course of excavation in different soil conditions are presented coupled with the field monitoring results.

1 INTRODUCTION

In Singapore, many of its underground infrastructures such as underground railway tunnels, locally named Mass Rapid Transit (MRT), are situated close to the existing buildings. As most deep excavations are carried out below the natural groundwater level, seepage-induced consolidation and drawdown may trigger ground movement related problems especially if the permeability of the soil is relatively high (Lee et al., 1993). Consequently, excessive soil deformation may pose a serious threat to the nearby existing structures supported on shallow or pile foundations. Therefore, stringent control over the wall deformation, subsequent soil deformation and excavation induced groundwater drawdown become essential requirements in all excavation design and methodologies. Unlike the constitutive behavior of soils which can be estimated by a comprehensive soil lab tests, groundwater behavior broadly depends on various factors such as soil permeability, weathering grade, structures in different geological formations, type of earth retaining walls and workmanship-induced factors. In following sections, three case studies of deep basement excavation were carried out in three different geologies and presented in the aspect of groundwater performance. All these projects were located in prime heart city areas and are easily recognized as a significant landmark structures in the neighborhood. Excavation works for the construction of deep basements are one of the key challenges in all the three projects because of the close proximity to the existing MRT stations and existing heritage buildings.

2 DEEP EXCAVATION WORKS IN SINGAPORE

2.1 BACKGROUND GEOLOGICAL CONDITION OF SINGAPORE

The geology of Singapore can be broadly divided into four main types: (a) the igneous rocks, consisting of the Bukit Timah granite and the Gombak Norite in the north and central north; (b) the sedimentary rocks of the Jurong Formation in the west and southwest; (c) the quaternary deposits of the Old Alluvium in the east; and lastly (d) the recent deposits of the Kallang Formation of the alluvium member, transitional member and marine clays distributed throughout the island (PWD, 1976; Pitts, 1984). Occasionally, a fifth type of geological unit known as Fort Canning Boulder Beds (FCBB) are encountered in the southern area of the island. The geological map of Singapore is shown in Figure 1.

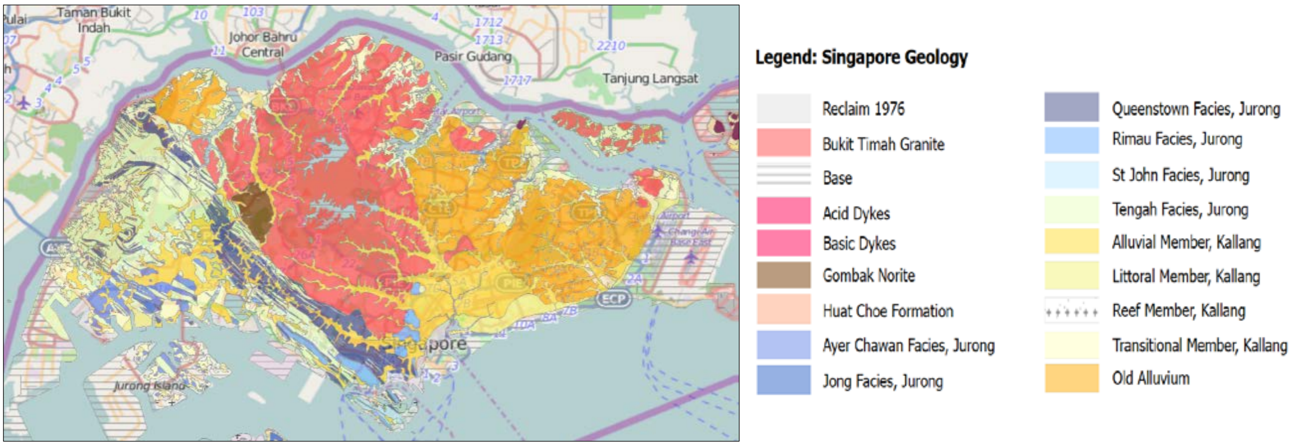


Figure 1: Geological Map of Singapore

The Kallang Formation, which was laid down over the last 15,000 years, covers about 25% of the land area of Singapore. It is usually found near the river valleys and commonly overlies the eroded upper surface of the Old Alluvium, Bukit Timah Granite and Jurong Formations and therefore rapid changes in thickness of the stratum are common. Marine Clay is the most dominant member of the Kallang Formation and others consist of the Fluvial Sand (F1), Fluvial Clay (F2) and Peaty Clay (E) layers. The Old Alluvium (OA) is typically described as a dense to very dense silty gravelly sand and very stiff to hard sandy clay or sandy silt. Due to its depositional environment, the OA is laterally and vertically highly variable with rapid and frequent variations leading to difficulty in correlation of individual lithological horizons. The Jurong Formation was lain down approximately 190 million years ago. The Jurong Formation is represented by different facies comprising beds of siltstone, mudstone, sandstone, conglomerate and limestone. It is typically tightly folded and fissured with the fold orientated from southwest to northeast. Due to the existence of folds and faults, rapid variation of materials and material properties is expected. A down-thrust which resulted in a near vertical cliff of Jurong Formation, followed by the collapse of this cliff by a sequence of minor events during down-thrusting resulted in the origin of the FCBB boulder bed. The FCBB, typically consisting of materials with sources from Jurong Formation, is generally found to be in direct contact with the underlying Jurong Formation.

2.2 COMMON PRACTICE IN DEEP EXCAVATION PROJECTS

With the development of underground infrastructures and basements, Singapore has gained well established knowledge and past experience in deep excavation and tunneling works. Various excavation problems commonly experienced in Singapore involve 1 to 3m deep for sewage and gas pipes, 8 to 26m deep for underground expressways, 12 to 30m deep (now up to 40m deep) for underground MRT railway stations, and other deepest excavations such as 40 to 60m deep for drainage sewer pipes and 72.5m for pumping station. With the careful control over the ground deformation and its related problem, wide variety of excavation methodologies have been adopted for successful implementation of excavation works. Some common methods include bored tunneling, cut and cover excavations with different types of earth retaining and stabilizing systems or open cut excavations. Generally, the selection of excavation methodology is mainly driven by subsurface conditions and the presence of nearby existing structures and services. Usually, the existence of adjacent structures requires a robust earth retaining system to restrain any movement due to unbalanced earth pressure imposed from basement excavation. The earth retaining stabilizing system also needs to provide effective water cut off so that groundwater table is maintained outside of the excavation. Some of the key factors influencing the selection of the appropriate retaining wall system include;

- Soil profile and design parameters
- Groundwater conditions
- Excavation depth and propping requirements
- Allowable deflection limit for retaining wall to minimize impact on the surrounding buildings/structures
- Use of retaining walls as part of the permanent works
- Maximize the available working space inside the site boundaries for the installation of temporary and permanent structures
- Cost benefits

Based on these factors, different types of excavation support system are adopted. This commonly includes flexible retaining walls such as sheet piles or steel pipe pile walls and stiff retaining walls such as Diaphragm Wall (800mm to 1500mm thick), Contiguous Bored Pile (CBP) walls and Secant Bored Pile (SPB) wall (diameter ranging from 800mm to 1500mm).

2.3 PERFORMANCE CRITERIA AND MONITORING REQUIREMENTS

In Singapore, one of the key criteria in the design of excavation is to meet the structural serviceability requirements of the nearby existing structures. The lateral movements of the retaining wall need to be limited to ensure that the works do not mobilize excessive ground movements as well as the groundwater drawdown that may result in the damage of neighboring structures. Building Construction Authority (BCA) advisory note 1/09 on ERSS limits the wall deflection based on the distance between the existing structures and the edge of the excavation depth. Basically three zones are categorized, namely Zone 1 ($x/H < 1$), Zone 2 ($1 \le x/H \le 2$) and Zone 3 ($x/H > 2$) where x = distance from excavation face and H = excavation depth. For all the major deep excavation designs, wall deflections are limited not to exceed 0.5%H, 0.7%H and 1.0%H for Zone 1, Zone 2 and Zone 3 respectively. For movement sensitive structures such as MRT stations and tunnels, the Code of Practice for Railway Protection by Land Transport Authority (LTA) stipulates a movement tolerance of 15mm.

With above mentioned stringent regulatory requirements and sensitive structures so close by new basement works, pro-active and extensive monitoring of wall deformation, ground deformation and groundwater responses naturally become important parts of the excavation design. In terms of construction risk assessment, the Alert, Action and Suspension Levels are defined as 50%, 75% and 100% of the allowable values respectively for each type of instruments. A typical instrumentation plan includes in-wall inclinometers for monitoring of wall deflection, in-soil inclinometers, rod or magnetic extensometers for subsurface movements, settlements markers for ground and building settlements and water standpipes and piezometers for groundwater monitoring. It is required that the excavation site is well equipped with appropriate instrumentation and monitoring system prior to the commencement of any major excavation works. Throughout the excavation processes, the performance of earth retaining and stabilization system (ERSS) is closely monitored by implementing a robust monitoring system with the predetermined review levels.

3 CASE HISTORY 1: CAPITOL DEVELOPMENT

3.1 PROJECT BACKGROUND

CAPITOL MIXED USE DEVELOPMENT site is located at the corner of major public roads, adjacent to underground City Hall MRT station. The development locates next to two to four storeys existing buildings called Capitol Theatre, Capitol Building, Stamford House and the Capitol Centre. The site also is bounded by Telephone House Complex, Chinese Chamber of Commerce and the Burhani Mosque. Development comprises the construction of 12 storeys high tower with a six level (25m) deep main basement. Two underground connections are also made with a 110m long B2 level connection to an adjacent City MRT Station. These two excavations would happen beneath the busy North Bridge Road and whilst a three level deep basement excavation linkway would connect to the new Stamford House hotel. Once completed, the 12 storey Capitol Development will boast one of the deepest basements of its size in Singapore. Figure 2 shows the layout of the development site and the areas it encompasses.

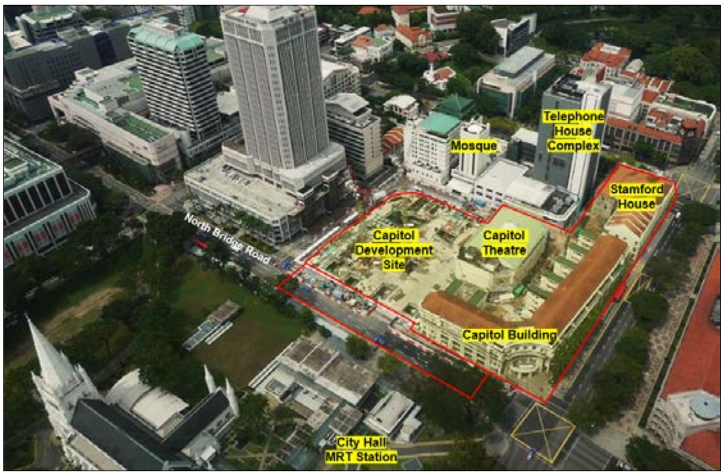


Figure 2: Site Layout and its Neighboring Structures (Capitol)

3.2 GROUND CONDITION AND KEY CHALLENGES

From the geological map, the project area is located at the intersection point of the main geological formations of Singapore, sitting on the Kallang Formation and underlain by the Jurong Formation. Nearby are areas of Fort Canning Boulder Bed and to the north Bukit Timah Granite. From the ground investigation carried out in 27 boreholes spread across the site up to depths of 80m below ground level, it was revealed that the Fort Canning Boulder Bed were discovered underneath the site and boulders were identified at approximately 70m below ground level. Generally the stratigraphy of the site appears consistent across the footprint with the exception of the Kallang Formation which forms a thick wedge at the southeast end of the site thinning to around a metre in the north and west. In general, the Jurong Formation was observed in the upper 30m and the SPT-N values are consistently above 100 at 60m below ground level.

There were a number of key geotechnical challenges on the project. The shallower ground conditions made it difficult to ascertain the geology (to distinguish between Jurong Formation and FCBB), and hence deeper GI was required to establish this. To add to this, the basement layout was complex, irregular in plan, shape and of varying levels. The main basement is six levels deep and two underground links connects to two other structures including the MRT and Stamford House via a two level deep passageway and three level deep L shaped basement respectively. The main challenge of this project is the proximity of sensitive buildings to the main basement excavation. All adjacent buildings may potentially be affected by the excavation works and therefore the effects of pile settlements, additional imposed forces and the structural capacity of the existing foundations and structures has to be considered and checked. Furthermore, it is located close to the City Hall MRT Station and its two running tunnels. Movement of MRT structures in any direction is limited below 15mm and measures were taken during construction to control inflow of groundwater. Although the Jurong Formation is relatively high in clay/silt content (typically in excess of 50%) there are pockets of sand and locally sandy areas which may result in sudden water influx, especially at deeper depths in the excavation where water pressures are higher.

3.3 ENGINEERING SOLUTION

For a stiffer support system of the retaining wall in order to minimize the ground movements, top-down construction method was adopted for the Capitol project. This allows for both superstructure and basement construction to go concurrently. It was proposed that the new building was to be founded on piled raft foundations. The basement and link sections was constructed inside the Contiguous Bored Pile (CBP) retaining wall. The CBP piles were 1.5m in diameter and had a 1.6m centre to centre spacing. A smaller diameter grout plug pile was used in between to prevent water ingress into the excavated site. For the retaining wall of neighboring underground pedestrian linkway excavation, CBP piles of 0.9m in diameter and 1m centre to centre spacing were proposed. Figure 3 details the geometry of the proposed CBP wall.

Figure 3: Proposed Contiguous Bored Pile (CBP) Wall

The typical construction sequence begun with the installation of contiguous bored pile walls, basement columns and foundation piles (steel plunge columns/king posts) prior to any excavation work. Then the top floor slab (L01) was constructed with at least one construction hole left open to allow for the removal of excavated soil. Further excavation down the opening took place only when the L01 slab has gained the sufficient strength. The excavation works were divided into three main zones, denoted as Plot A1 for main 6 level basement, Plot A2 for the linkway underneath the road and Plot A3 for the 3-level linkways connecting to Stamford House as indicated in Figure 4. The excavation was commenced at Plot A1 with a six storey basement structure using top-down construction, allowing the residential tower to be constructed simultaneously. Access for the link tunnel excavation in Plot A2 was provided via basement 2 of Plot A1. The third link tunnel behind Capitol Theatre then connects Stamford House and the new Capitol development. Access for this excavation was provided via basement level 2 of zone A1.

3.4 FIELD PERFORMANCE

Extensive instrumentation and monitoring system was implemented to closely monitor the performance of excavation support system during basement excavation. As shown in Figure 4, proposed monitoring plan includes a typical clustered instruments (consisting of inclinometer, water standpipes and piezometers), ground and building settlement makers, tilt meters and real time monitoring system for underground MRT station and tunnels. Ground settlement markers are installed with average spacing of 5m along the site boundary and around the North Bridge Road. To monitor the groundwater drawdown, a total of 6 clusters of water standpipes paired with piezometers installed with multi-tips at different depths. The tips were positioned near formation level and at the toe of retaining wall.

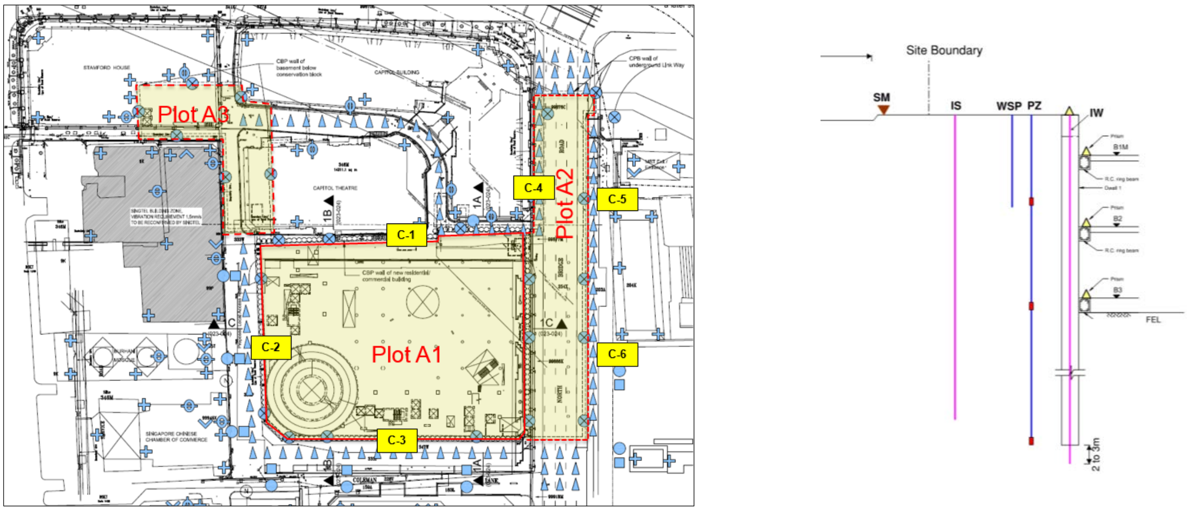
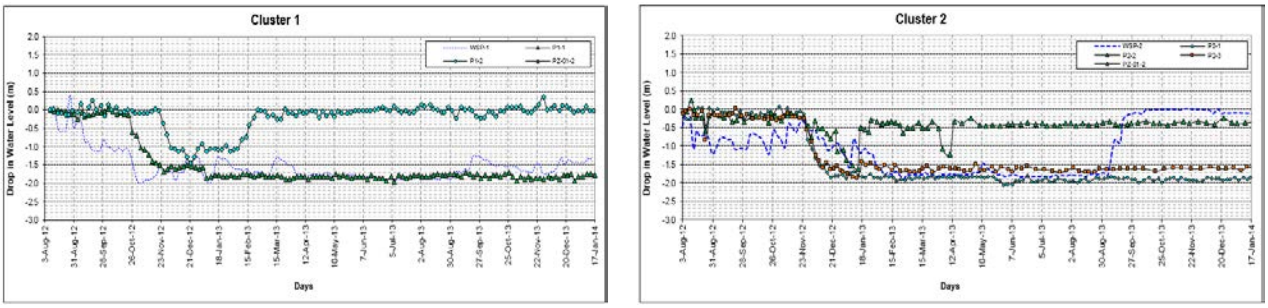


Figure 4: Instrumentation Layout Plan

Typical measurement of water standpipes and piezometers are plotted in Figure 5. Main excavation in Plot A1 area was commenced in August 2012 and completed in mid of 2013. Excavation for construction of linkway underneath the North Bridge Road begun in April 2014. In general, the water level dropped up to 2m over a 10 month period of the 28m massive excavation in Plot A1. The pore pressure response in piezometers was observed only when the excavation proceeded from B2 to B3 level. The late response of pore pressure was likely due to the fact that the excavation was sequenced in stages with small zones minimizing the disturbance to the soil. The soil behaved more in undrained manner and may have cause the delay in pore pressure drop. Since excavation was started from the Capitol Building side, relatively early sign of response was observed in Cluster 1 (C-1). Most of the piezometers reflected the typical trends of pore pressure changes, which demonstrated drops in piezometric levels as excavation progressed. Maximum drop was recorded upon reaching the final formation level. It was noted that after the drop in pore pressure, it took time to recover even after the casting the basement slab.



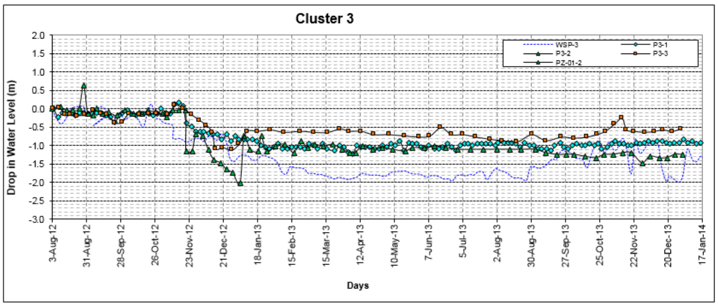


Figure 5: Groundwater Monitoring Vs Time

The deflection of CBP retaining walls recorded from the in-wall inclinometers were well within the predicted design values. It was also observed that the exposed face of CBP walls were dry throughout the basement excavation period with no sign of any leakage through the walls. This proved the effectiveness of grout piles installed at the gap between the piles. The pore pressure drop registered in piezometer tips were also considered relatively low considering the excavation depth up to 25m. It is likely due to the minimum deformation of walls as well as the undrained behavior of the, shown in Figure 6.

Figure 6: Face of excavated soil exposed during Basement Excavation

4 CASE HISTORY 2: TANJONG PAGAR DEVELOPMENT

4.1 PROJECT BACKGROUND

TANJONG PAGAR MIXED USE DEVELOPMENT comprise of a 290m tall office and residential tower, a medium-rise hotel block and a six-storey podium for retail shops. It will also house a three level basement car park of 18m deep. The development will tower over the other buildings in its vicinity with its soaring 64 storeys, making it the tallest building in Singapore. Direct links will also be constructed to connect the development to an existing underground MRT station, located right next to it. One of the biggest challenges of this project was to control the movement of the live MRT structures within the stringent criteria set by the Land Transport Authority. There were also two other buildings founded on shallow foundations located less than 20m away from the proposed development - Maxwell Chambers and shop houses at 76 Peck Seah Street. These adjacent sensitive structure necessitate the need to understand likelihood movement. Figure 7 shows the layout of the development site and the areas it encompasses.

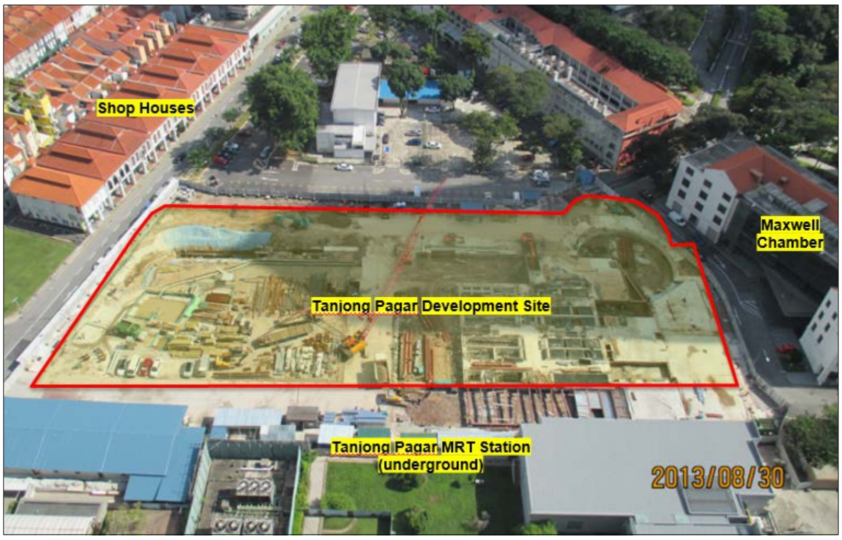


Figure 7: Site Layout and its Neighboring Structures

4.2 GROUND CONDITION AND KEY CHALLENGES

From the geological map, the project area is known to be underlain by the Jurong Formation. Generally, the stratigraphy of the site appears relatively consistent across the footprint of the proposed structure. Soil profile starts with around 3m of Fill, underlain by Residual Soils of the Jurong Formation and Completely Weathered Jurong Formation. The Jurong Formation soils in the first 5m of top soils are typically residual soil to completely weathered with SPT-N value of between 50 and 100. Below that, SPT-N values are consistently above 100 (extrapolated), indicating completely weathered soil and rock interface. The intermittent fractured rock layers are found but not continuous. What is inferred to be a continuous layer of highly weathered rock was encountered in 4 of the 20 ground investigation boreholes. Water standpipe readings recorded during site investigation stage indicates that the natural groundwater table vary at 1 to 2m in depth below ground level.

The presence of conservation buildings and MRT assets surrounding the site necessitates more rigorous control of ground movements to prevent damage to the nearby structures. The piles control the tower settlement influence zone to meet the stringent settlement criteria set by the Land Transport Authority on MRT structures adjacent to the site. They also limit differential settlement underneath the tower due to eccentric loadings from the towers sitting on varying soil conditions.

4.3 ENGINEERING SOLUTION

A full top-down construction method was adopted for the basement excavation works. For earth retaining system, a combination of 1.1m diameter CBP wall, shown as blue line in Figure 9, and 1.3m diameter SBP wall, shown as red line in Figure 9, were proposed. A water cutoff was achieved by providing a smaller diameter LSS pile in between the CBP. Basement excavation was staged in two phases for better control on ground disturbance and to enable the early start of superstructure construction. Considering that the site is underlain by relatively weathered sandstone/siltstone of the Jurong Formation, a piled raft was adopted as the foundation system to support the column and wall loadings of the above tall superstructures. The piled raft system consists of bored piles of diameter ranging from 900mm to 1800mm spaced at 2.5D (D = diameter of pile). They were placed mostly under the perimeter columns and the central core of the tower, where most of the loads originate, and the raft serves to work the piles in tension. The rafts were a 4m thick slab under the Office Tower (Phase I) and 1.5m thick slab for Hotel Tower. (Phase II). The visualization of the combined retaining wall and foundation system is provided in Figure 8.

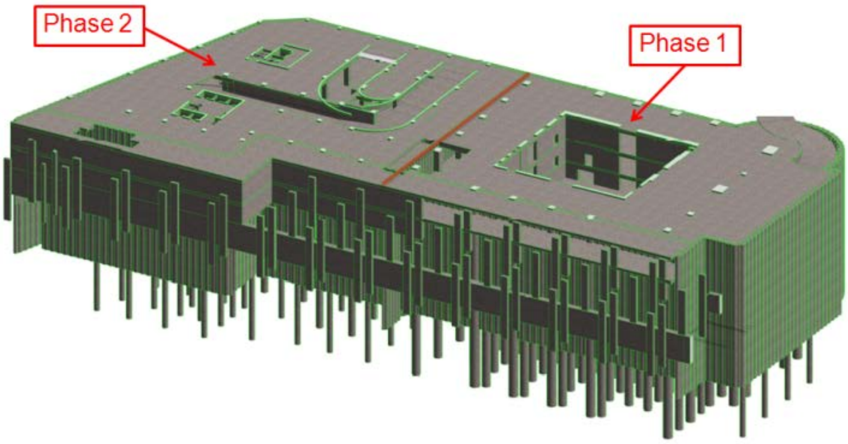


Figure 8: Pile Raft Foundation with Bored Pile Retaining Walls

4.4 FIELD PERFORMANCE

Being located adjacent to the existing underground MRT station and sensitive buildings, tight control over the construction works was required. Comprehensive monitoring scheme was implemented to ensure that the construction induced movements stayed within the allowable limits. Performance of ERSS walls were measured by the cluster of instruments which consist of inclinometer, water standpipe and 3 tip piezometers. They were positioned around the main excavation area, Phase I and Phase II, as shown in Figure 9. In additional to the cluster instruments, a series of ground settlement markers were provided along the surrounding roads for the monitoring of surface settlement induced by groundwater drawdown during excavation.

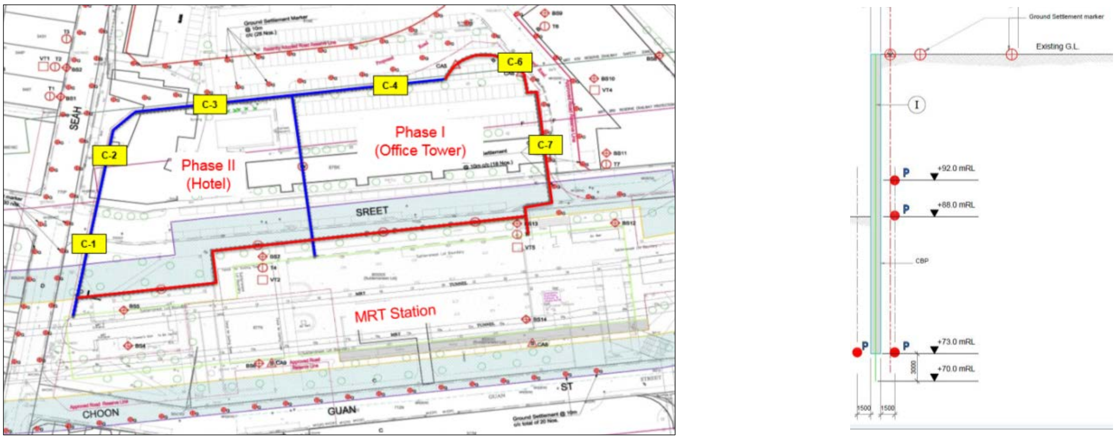


Figure 9: Instrumentation Layout Plan

Basement excavations were carried out in two phases, with Cluster 4, 6 and 7 for Phase I excavation and Cluster 1, 2 and 3 for Phase II excavation. Figure 10 below shows the trend of groundwater behavior in response to basement excavation works. From the graphs, it could be seen that pore pressure dropped as excavations deepens and reaches a maximum drop at final excavation level. It then stabilized and gradually recovered. All the piezometers demonstrated the same trend of pore pressure response, with the most significant response observed in the highest piezo-tips. Maximum drop in piezometeric pressure was as high as 10m. On the other hand, the groundwater table drawdown as measured from water standpipes varied ranged from 2m at Cluster 6 and Cluster 7 to maximum 6m at Cluster 4 and Cluster 1.

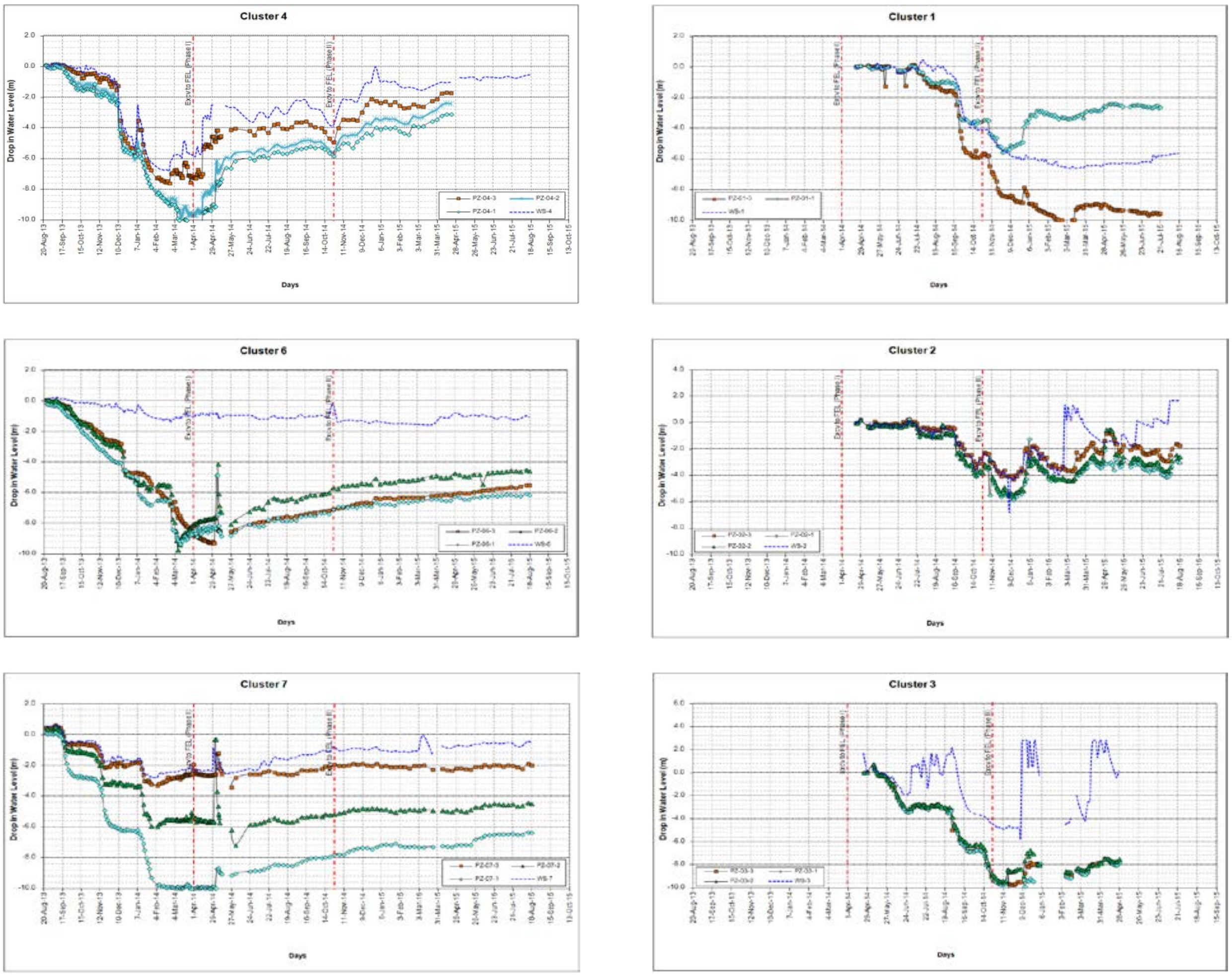


Figure 10: Pore Pressure Response over time graphs

Other than the overall response behavior, it was learnt that piezometers can provide quite accurate responses to localized scenarios encountered during the course of basement excavation.

Scenario 1: Localized excavation for Storage Fuel Tank (Phase I)

There was a localized excavation for construction of a storage fuel tank right behind the CBP wall occurred during the period of 4 days (03/12/2013~07/12/2013) in Mar 2013. The excavation was 5.5m deep and 2m away from Cluster 4. Timber laggings were used to retain soils. The layout and inside view of the excavation is shown in Figure 11. It can be seen from the photos that there were some water leakage at the face of timber lagging, which was reflected as a sudden drop of water table and pore pressure at Cluster 4 around Dec 2013. Although immediate remedial action was taken by pouring concrete behind the timber lagging to stop or slow down the water leakage, the dramatic drop occurred over a short period did not recover. However, pore pressure was maintained as the excavation went deeper. It was also noted that ground settlement markers along the road adjacent to the excavation registered no significant increase in ground settlement.

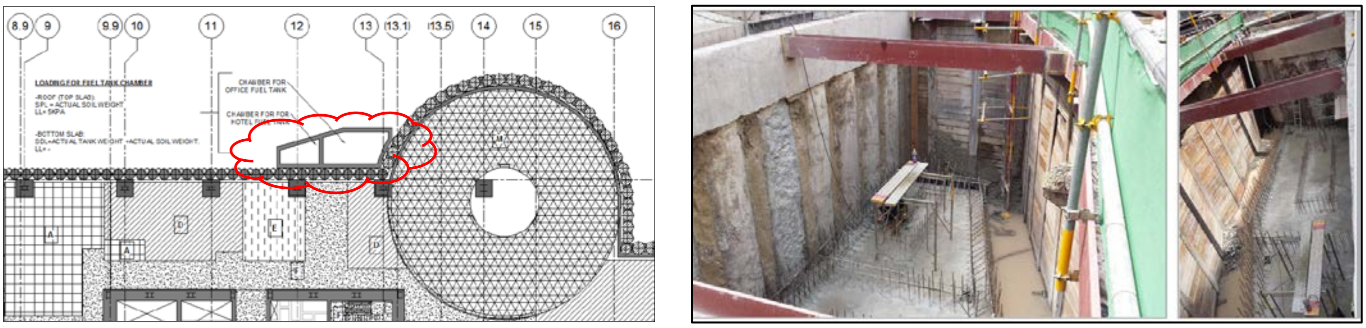


Figure 11: Localized Excavation for Fuel Tank

Scenario 2: Casting of 4m Thick Base Slab (Phase I)

Having a large footprint area of the 290m tall tower, Phase I area includes the casting of 4m thick base slab after the completion of excavation to final formation level. From records, the casting of the slab was done (around end of April 2014) with the continuous pour of concrete over two and a half day period. Dehydration during curing process of massive concrete volume resulted in rise of temperature to the surrounding soil. As a consequence, the piezometer readings were interfered by the effect of temperature changes, as can be seen in Cluster 4, 6 and 7 (29 April 2014) of Figure 10. However, the effect was short and the readings were back to a normal trend within a week.

Scenario 3: Groundwater Performance of CBP and SBP Walls

As discussed in Section 4.3, a combination of CBP and SBP was proposed as the retaining wall system. Considering its close proximity to sensitive structures such as Maxwell Chamber and an underground MRT station which sits on a raft foundation, SBP wall was proposed for the retaining walls facing them. From the water standpipe readings, it was noticed water standpipes behind the SBP wall showed minimal draw down (2m at Cluster 6 and 1m at Cluster 7) as compared to CBP walls. The rest registered the water table draw down of 4 to 6m. This suggests that in terms of groundwater performance, SBP wall performed better than CBP wall. Figure 12 shows the photos of CBP and SBP wall faces exposed during the excavation.

Figure 12: Exposed View of CBP Wall (left) and SBP Wall (right)

From overall study of piezometer and water standpipe readings recorded over the basement excavation period, a significant reduction in piezometric pressure was observed for both excavation phases. The large reduction in the pore pressure can be attributed to the Jurong formation and its behavior during construction. The stress relief during excavation causes the fissures and fractures to open and significantly increases the permeability from its preconstruction state, a phenomena noted elsewhere in other excavations in Jurong Formation, where estimated permeability's of 1.15×10^{-6}m/s was observed (quoted from Ong 2006). The depth of the SBP/CBP walls does not reduce the large draw-down as 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_0 condition, i.e. a reduction in σ_3 (confining stress)'. In soft and compressible subsurface ground condition, the localized piezometric drawdown can lead to consolidation settlements and subsequent ground settlement. However, in the vicinity of Tanjong Pagar site along the Wallich Road, the subsurface ground conditions are relatively stiff residual soil underlain by weathered weak rocks. Ground settlement is rather minimal due to the reduction in pore water pressure.

5 CASE HISTORY 3: SOUTHBEACH MIXED DEVELOPMENT

5.1 PROJECT BACKGROUND

THE SOUTH BEACH DEVELOPMENT project is a high-quality mixed development featuring two mixed-use towers of 45 and 34 storeys, 4 podium blocks and retail spaces and 4 conservation buildings. Occupying a complete street block of approximately 3.5 hectares wide, the proposed site is bounded by major public roads. To the west of the development site is the existing underground Esplanade MRT station. Figure 13 shows the layout of the development site and the areas it encompasses. Figure 13 shows the layout of the development site and the areas it encompasses.

Figure 13: Site Layout and its Neighboring Structures

5.2 GROUND CONDITION AND KEY CHALLENGES

A desk study on the reclamation history of Singapore reveals that the site is located between Beach Road and Nicoll Highway comprises a mixture of "FILL" materials from two different phases of reclamation. Top soils at the project site are general or ancient fills and another are fills from Phase III reclamation for the construction of the Nicoll Highway. In overall, the site is underlain by very dense silty sands, silts and hard clay of the Old Alluvium (OA). Above the OA are recent deposits of the Kallang Formation comprising of soft to firm clays of marine, fluvial, estuarine and fluvial sands. Notably, the top level of the OA gets deeper towards south and east of Nicoll Highway and shallower towards north and west of the Esplanade MRT station.

As shown in Figure 14, a notably deeper valley of the Old Aluvium was observed in the central and the south-eastern part of the project site, with depth of Kallang Formation highly varying from 5m to 40m. Such deep deposits of soft clays can cause substantial ground movements during the excavation. The deep valley of F1 sand layer that was found to be about 18m thick in the west end of site also posed the additional challenge in construction of diaphragm walls, and subsequent groundwater associated risks, such as water drawdown and consolidation. The complex and variable relationship between the fluvial sands, marine clays, which interchange laterally, is ideal for consolidation settlements. This problem was compounded by the presence of the adjacent conservation buildings. The four conservation buildings are located near Beach Road, with the distance to the retaining wall as close as 0.9m. All the buildings were founded on the piles except one which was founded on a mixed foundation system and therefore prone to differential settlement if the soft clays beneath these foundations consolidate. The Esplanade MRT station is located at Bras Basah Road and it is next to the basement. The project site lies within the railway protection zone which requires the strict and stringent control over the surrounding ground movements.

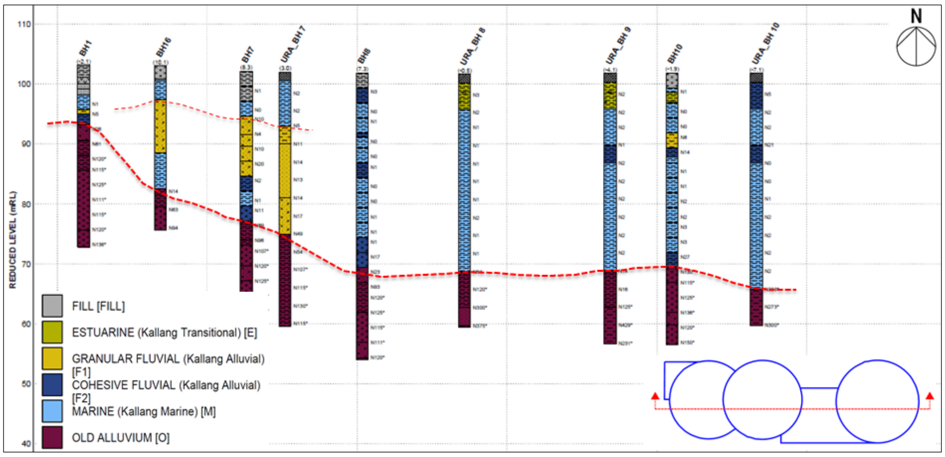


Figure 14: Longitudinal Soil Profile across the Site

5.3 ENGINEERING SOLUTION

Considering the site constraints and the need of stringent control over the ground movements, it was proposed to use two circular diaphragm wall cofferdams of 85m diameter and 90m diameter connected by linear T-shaped diaphragm wall panels. The use of diaphragm walls arranged in triple circular configuration enabled the 18m deep excavation to be carried out in poor ground conditions with very minimal deformations without any steel strutting or soil improvement works. The two complex circular diaphragm walls are 1m thick and the adjacent diaphragm wall facing the MRT station is 1.2m thick. Top-down excavation method is proposed for this solution. To tackle the risk of sudden water drawdown and subsequent settlement, the excavation phases were split into two. Excavation was done within the 1.2m thick diaphragm wall in the first phase (Zone 2 and Zone 4) and was followed by that in 1.2m thick diaphragm wall and the linear T-shaped diaphragm wall (Zone 1 and Zone 3). The arrangement and layout of diaphragm walls are shown in Figure 15.

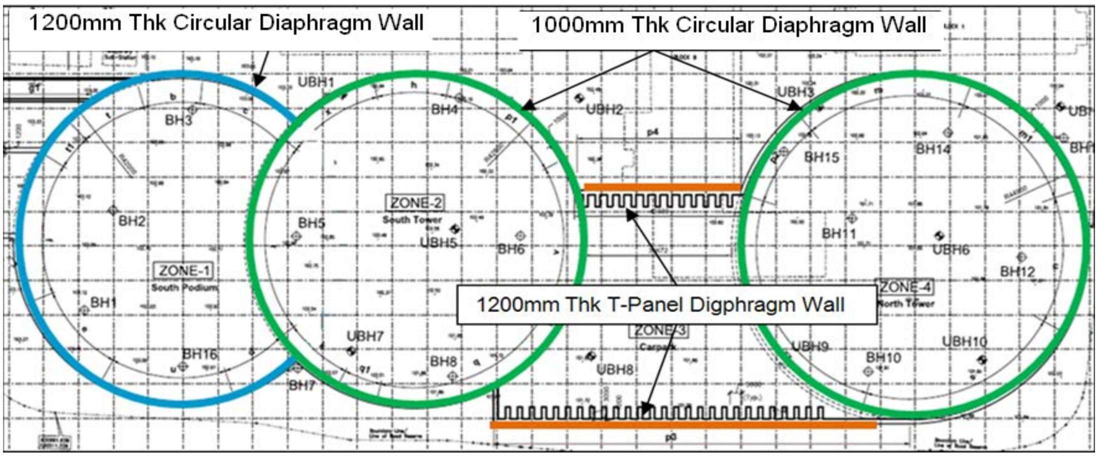


Figure 15: Proposed Diaphragm Wall Layout Plan

5.4 FIELD PERFORMANCE

From the extensive monitoring, it was observed that the triple circular diaphragm wall system had performed satisfactorily in controlling the wall deformation. In additional to the overall stability of retaining system, the water standpipes paired with piezometers were installed with multi tips positioned at different depths. There are three tips installed at the approximate depth of 0.5H (H=excavation height), near the final excavation level and near the toe of the diaphragm wall. The first and second tips were mostly located in the marine clay and all the lowest tips were embedded into OA. Extensive amount of settlement markers were also installed around the site, with average spacing of 5m along the site boundary and with 10m spacing on the middle road divider. Full instrumentation layout plan for excavation monitoring is shown in Figure 16.

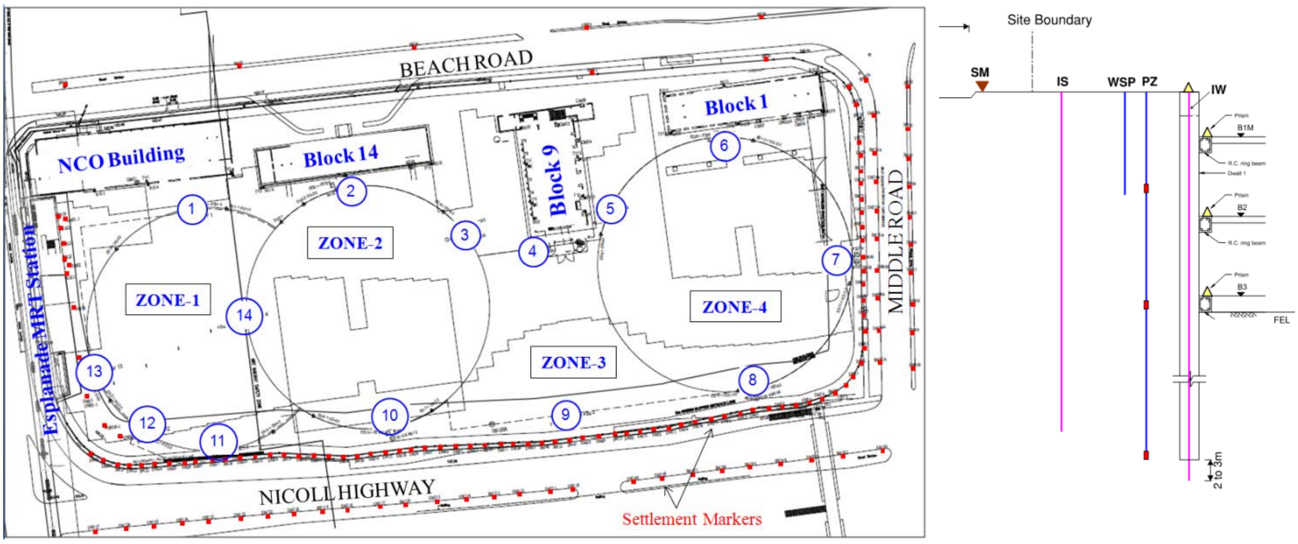


Figure 16: Instrumentation Layout Plan

Generally most of the piezometer tips registered the drop in pore pressure almost immediately after the massive excavation progressed to the first ring beam level. Only PZ08, installed in Zone 4 with the presence of 40m thick marine clay, showed the relatively slow response to the excavation. This was due to the first and second tips being located in marine clay layer, with the distances of 20m and 10m respectively above OA layer, whilst the tips of others piezometer were closer to OA layer. Generally, the first tips at shallow depth were showing smaller drop in pore pressure, while the lower tips located in OA layers or closed to OA layers measured higher piezometric drops. Most of the piezometers reflected the typical trends of pore pressure changes, which showed that the drops in piezometric levels continued with the progress of excavation works with the maximum being at the final formation level. It was noted recovery only happened after the completion of basement 3 casting. The piezometers and water standpipes at Zone 1 showed no drops in water level. This was due to adequate recharge wells installed along the perimeter of Zone 1 circular ring, and activated during the Zone 1 excavation works. The Table 1 below summarizes the drops in piezometric pore pressure, shown in equivalent water head (m).

Table 1 Summary of Piezometers.

Tip	Soil at tip	Depth of tip	Max measured piezometric drop (m)						
			PZ5	PZ6	PZ7	PZ8	PZ2	PZ3	PZ10
WS	FILL	12	-2	-4	-2.5	-1	-2.5	-3	-2.5
1	FILL	7					-4.5	-4.5	-2
	MC	7	-6.5	-5	-3.5	-3.5			
2	MC	16	-8	-6	-6	-5.5		-6	
	OA	16					-5.5		-7.5
3	MC	26			-9				
	OA	36	-8.5	-6.5		-5.5	-5.5	-7	-7.5

A series of settlement markers were pre-installed at a greater spacing for monitoring during diaphragm wall trenching works. Maximum ground settlement about 15mm to 20mm was recorded in Zone 4 area where, thick layers of soft clay were present. The settlements measured at other areas before the commencement of excavation were less than 10mm. Generally, during basement excavation, increase in ground settlement of 45mm and 60mm were recorded during the 18m deep basement excavation works in the circular cofferdam of Zone 2 and Zone 4. The ground settlement got significantly larger towards the south-eastern end of project site where the underlying marine clay was thicker up to 30 to 40m. On the other hand, very small ground settlements were recorded in the vicinity of Zone 1, where OA level was significantly higher. The recorded trend of groundwater behavior during the main of excavation in two circular coffer dam of Zone 2 and Zone 4 are illustrated in Figure 17 together the nearby ground settlement readings.

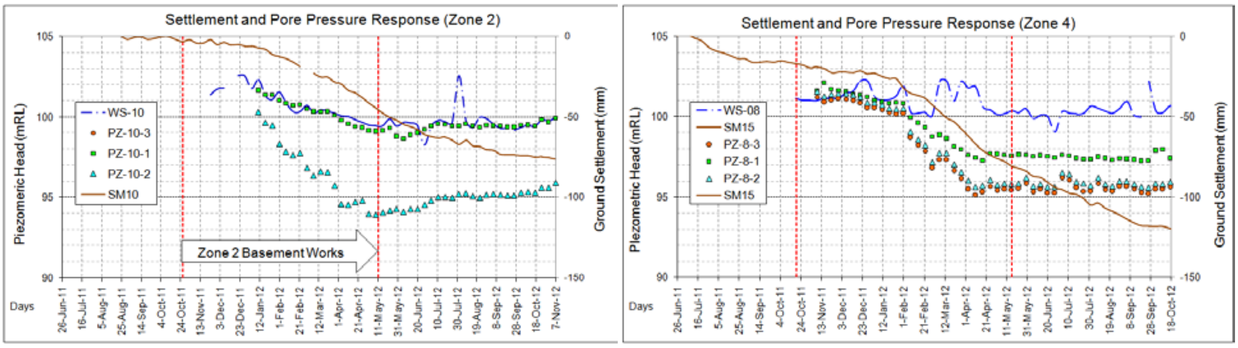


Figure 17: Settlement and groundwater response to excavation

6 DISCUSSION ON GROUNDWATER BEHAVIOR

In general, the pore pressure response poses a major concern in addition to the overall stability of retaining system. The concern is mainly due to damage risks towards existing foundations that could be affected by post- excavation settlements triggered by a water drawdown. It is well understood that all the deep excavation works will lead to a piezometric drop in undrained soils behind the retaining walls. This could be the result of a combination of one or more factors such as groundwater table drawdown, seepage through the retaining walls either due to permeability of the wall material or through the joints, relatively high permeability of retained soil which encourage seepage flow underneath the toe of the retaining wall and reduction of horizontal stresses behind the wall, which lead to the change in pore pressure.

Out of the three case studies presented above, two projects are located in favorable ground conditions of the Jurong Formation and Fort Canning Boulder Bed with relatively high SPT values. The field permeability tests carried out during ground investigation state showed that in-situ permeability of soils ranges between 1x10^{-7}m/s to 1x10^{-8}m/s. In the Capitol project, it was observed that the mixed soil within the excavation zone above FCBB is fairly stiff and wall deformation recorded in inclinometers were minimal (only about 10mm to 20mm). Drop in the piezometer head outside the excavation is therefore considered purely due to undrained unloading responses of the ground. However, significant decrease pore pressure was registered in the case of Tanjong Pager. Although lateral movements of the wall recorded were about 15mm to 20mm, the reduction in horizontal stresses has led to a reduction in pore pressure. It was also noted that the reduction in pore pressure had a significant correlation with the trend of groundwater table drawdown measured in water standpipes. This can be attributed to the natural geological characteristic of the Jurong Formation and its behavior during excavation. Stress relief had caused the fissures and fractures to open and significantly increase its water permeability from its preconstruction state. Similar phenomena were noted elsewhere in other excavations in Jurong Formation, where estimated permeability's of 1.15x10^{-5}m/s was observed (Chen et al 2000). It took about two years after the base slab was cast for the pressure head to be restored to its original levels. In such cases, having a deeper penetration of retaining walls as a form of cut-off may not be a solution for preventing the draw-down. While pore pressure drops outside the excavation are undesirable, there was no evidence of subsequent large settlement in the surrounding region.

In Southbeach, the maximum water head drop recorded ranged from 6m to 9m. In this case, the water table drawdown measured in water standpipes and the stress reduction are considered as the main contributing factors for the large drop in pore pressure. The contingency proposal such as recharge wells or topping up the road surface were proposed and well planned prior to the onset of pore pressure disturbance. The pore pressure reductions in OA layers resulted in the drainage of the overlying soft clay layers, triggering widespread consolidation settlements of the ground. This consolidation settlements were even enhanced by the presence of Fluvial Sand layers which interchange horizontally in between the soft clays. There is a close relationship between piezometric head drop and settlement (Osborne et al, 2005). Although the ground settlements immediately responded to the pore pressure reduction, the increment continued in the same trend even after the piezometric drops stabilized with the casting of base slab. The pore pressure decrease in OA causes dissipation of pore pressure in the marine clay at the OA-Kallang interface. This resulted in the consolidation of the marine clay (which continued after casting of the base slab) and consequently ground settlements. (C.C.Chaing,et al. 2006). Although the drop in piezometric levels stabilized after casting the base slab, the increase in settlement continued to occur at the same trend as shown in Figure 17. It was learnt that the adequate recharge system activated prior to assist in maintaining the water table and helped control the ground settlements to adjacent sensitive properties, such as Esplanade MRT station in the west side of Zone 1 and NOC Club building in the north of Zone 1.

It is inevitable that piezometric drawdowns will occur at the locality of deep excavations. However, the magnitude of the drawdown is dependent on the factors discussed above and therefore may be different from case to case. From the first two case studies discussed above where excavation was carried out in stiff soil condition, the subsequent settlements recorded were minimal. Where compressible soils are present as in Case Study 3, drawdown will result in consolidation settlements and if left unchecked these settlements can be substantial. However, through prudent planning and design, the drawdown can be limited, thus maintaining settlement to acceptable levels. It is recommended that a risk approach be taken and stringent measures to control drawdown only be applied to where structures are liable to suffer damage from consolidation settlement, but less stringent measures can be considered if a greenfield environment abounds. It is also recommended that recharge systems are always installed prior to onset of construction and strict maintenance regimes are implemented to optimize the benefits of the system.

7 CONCLUSION

In this paper, field performance in terms of groundwater responses during basement constructions from three case studies were discussed with the comprehensive instrumentation data. Some observations and lessons worth noting about the site conditions and activities that may influence pore pressure behavior were discussed. It was learnt that not only the predicted wall deformation was important but we should also be mindful of the nature of subsoil stratigraphy and pore pressure responses in the prediction of settlement behaviors. The factors contributing to the pore pressure responses should also be properly considered and addressed in the design consideration. This is important especially if thick layers of soft marine clay is present and may pose a concern on long term settlement. This paper is intended to serve as a project reference useful for the similar future projects.

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