Increasing geotechnical challenges in the design and construction planning of the Third Phase Bangkok MRT Underground

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ABSTRACT: The 3rd phase MRT underground project of Bangkok that is being implemented has faced increased difficulties in design and construction owing to the congested corridor along the alignment and the phenomenon of groundwater rebound in Bangkok subsoils in the aftermath of the end of land subsidence era from deep well pumping. The rebound is more than 10 meters from the situation during the initial phase construction and thus it necessitates careful soil investigation and design approach to prevent instability of groundwater ingress and uplift in bore tunnelling and excavation of station boxes. The tight space along the MRT underground alignment resulted in the twin bored tunnels being placed at very small clearance in a few locations. It creates high risk in the shield driving, particularly in the saturated sand layer. The works also involve tunnel driving at close proximity or under a number of existing buildings or road viaducts. In such cases, various schemes of underpinning are employed, and one of them is strengthening of founding soil layers by cement jet grouting. An intensive instrumentation program according to the observational design method is employed to safe guard against occurrence of any undesirable instability in the excavations and impacts to the third party properties.

Keywords: MRT Underground, shield tunneling, deep excavation, underpinning, groundwater rebound, the observational method

1. INTRODUCTION

The 1st phase MRT underground project, i.e. Blue Line (Initial) was constructed in 1999-2003. It consisted of 18 stations and 22 km of twin bored tunnels running along inner city roads. The tunnel was successfully bored with 4 EPB shields and the station box was smoothly constructed using concrete diaphragm wall with the top down excavation method. For the 2nd phase project, Blue Line Extension, the works began in 2011 and completed 2017. The 5.4-km underground section consisting of 4 stations and 4.8-km-long twin bored tunnels were placed along narrow roads in the old city quarter and crossed the main river of the city. In this phase, the shield tunnelling and station excavation experienced incidents of severe difficulties associated with the impact of groundwater rebound in the subsoils of Bangkok. The rebound was the consequence of banning deep well pumping to solve land subsidence problem in Bangkok area. At a section of shield tunnelling, problematic groundwater ingress occurred through segmented lining just installed while driving through a mixed face of clay and sand layers; and in one of the station box excavation a hydraulic uplift of the excavation base occurred that resulted in flooding and significant complication in finishing the excavation and casting the concrete base slab.

Currently, the 3rd phase underground project, Orange Line East, is under construction. The underground section consists of 12.1 km of twin bored tunnels and 10 stations placed along two of the most traffic congested roads of the city where there exist elevated road and expressway viaducts along the alignment. The piled foundation of the viaduct complicates the alignment design of the MRT underground tunnel and positioning the station boxes that result in necessity in close proximity placement of the twin bored tunnels to each other and to neighbouring structures at numerous locations. In addition, the soil investigation reveals that the subsoil stratigraphy consists of sand layer of larger thickness and higher rebounded groundwater levels than that in the earlier phase in other zones of the city. The design approaches of the underground works of the project are outlined below. In addition, presentation is made on the experiences in the design and difficulties in dealing with the situation of risen groundwater pressure in the sand layers in the tunnelling and deep excavations just made in the 2nd phase – Blue Line Extension.

2. BANGKOK MRT SYSTEM

The MRT system of Bangkok, when fully implemented, is shown in Figure 1. As it has been planned, the underground portion constitutes about 60 km in combined length, of which about one half have been completed. Besides the underground works of the Orange Line East that are being constructed, the Orange Line West and Purple Line South projects, which are to start the implementation by the end of the year, have the remaining underground portion of the Bangkok MRT system. Works of the three underground MRT projects that are being or to be soon carried out are challenging because of the very tight work spaces and the risen groundwater head in the subsoils.

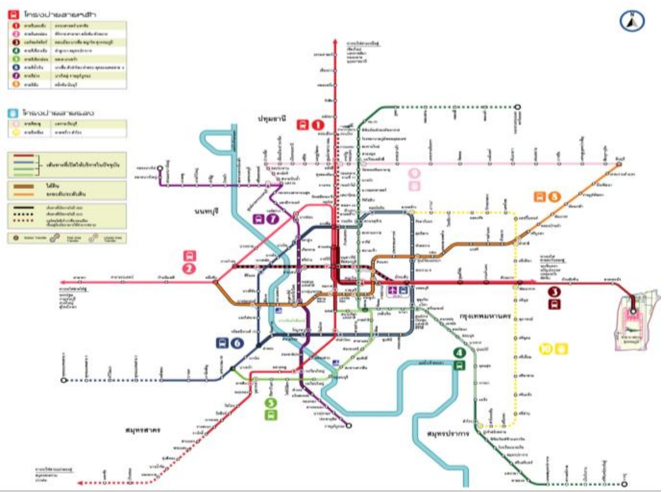


Figure 1 Bangkok MRT System

3. SUBSOILS AND GROUNDWATER

Bangkok is situated at the head of the Gulf of Thailand on the southern part of the low lying Chao Phraya plain. The vast flat plain is carpeted with a thick soft marine clay layer which overlies a very thick series of alluvial soils of alternating layers of stiff to hard clay and dense to very dense sands. In the upper 50 m zone within which most foundation and excavation works are placed, the subsoil profile is relatively uniform throughout the city area, which is an ideal condition for tunnelling. However, the soft soil condition necessitates the use of shield method for tunnel construction. The shallow zone consists of a 12-15 m thick layer of soft to medium stiff clay (Bangkok clay) that is followed by a layer of stiff to hard clay and a layer of sand (the First Stiff Clay and the First Sand). Below them, alternating layers of stiff to hard clay and dense to very dense sand exist to a great depth. The sand layers are aquifers of Bangkok groundwater system. The subsoil stratigraphy along the Orange Line East that is under construction is shown in Figure 2.

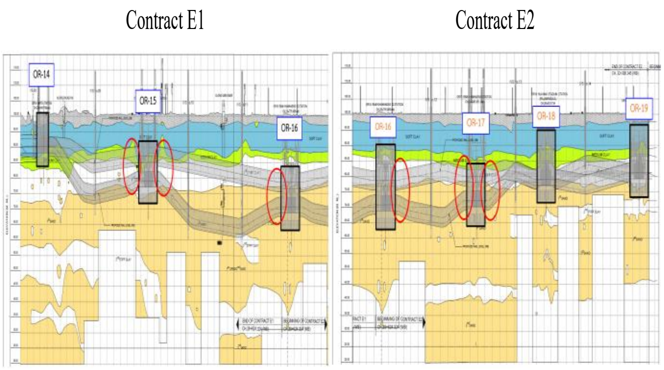


Figure 2 Soil profile along E1&E2 Contracts of Orange Line East

It is well known that Bangkok had suffered land subsidence problem owing to excessive deep well pumping from the sand aquifers for water supply. The decline in groundwater head in shallow aquifer layers was as much as 25-35 m from the ground surface during the peak of the crisis in early 1980s, when the subsidence occurred at a rate as high as 120 mm/year. After the surface water supply was made adequate in most of the city area around the turn of the century, the situation improved as deep well pumping diminished. Since then the rebound of groundwater started to develop and the subsidence ceased in most city areas. The rebound continues till present and the accumulative amount is about 10-12 m during the last 15 years. At present the piezometric water head in the 1st and 2nd sand layers of Bangkok where the MRT underground works are involved with is about 13-14 m depth below the ground surface that is 10-12 m higher than the condition when the 1st Phase MRT Underground line was constructed (Figure 3). It has been risen at a rate of approximately 0.8-1.0 m per year (Figure 4). Because the groundwater law that has been in effect since 2004 totally bans deep well pumping in the entire area of Bangkok and surrounding suburbs, it is foreseen that the ground water heads in the sand layers might rebound to the hydrostatic level in the near future.

In the past the subsidence situation created complication of long term settlement and differential settlement in the design of MRT underground structures. On the other hand, the condition of the lowered groundwater head in the sand layers resulted in ease in deep excavations for station construction and shield tunnelling. Incidents of hydraulic instability were rare in past deep excavations and tunnelling. However, at present the situation has changed since the groundwater head started to show significant rebound about 15 years ago. Difficult and problematic incidents were experienced in a number of recent deep excavations of building construction as well as in the works of the Blue Line Extension Project. In addition, the rise of groundwater head was linked to incidents of difficulties in EPB shield tunnelling in the city including a severe incident in the 2nd phase MRT Underground project that led to a six-month delay of the stall and repair of the TBM.

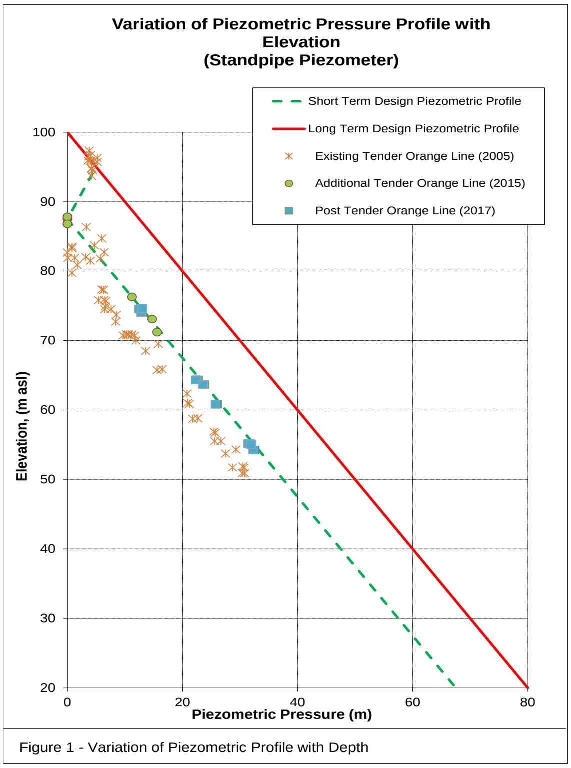


Figure 3 Piezometric pressure in the subsoils at different times

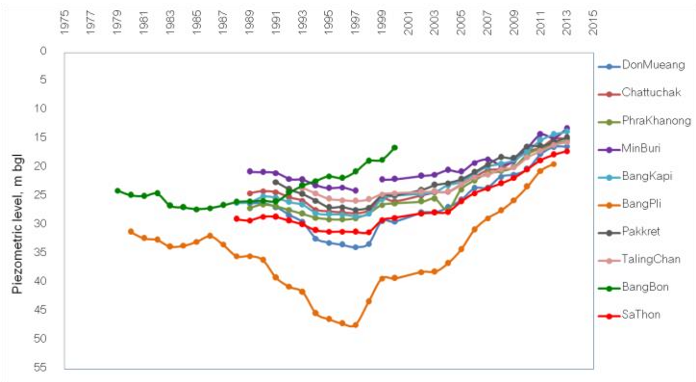


Figure 4 Changes in piezometric heads in Bangkok sand layers

4. INCIDENTS IN PAST MRT UNDERGROUND WORKS

Problematic incidents related to the impact of groundwater rebound in the underground works of the past MRT project are presented. The recently completed underground structures of the Blue Line Extension were placed in the stiff clay and sand layers as shown in Figure 5. The deepest bored tunnel section is at 37 m depth at tunnel invert at a location near the river crossing.

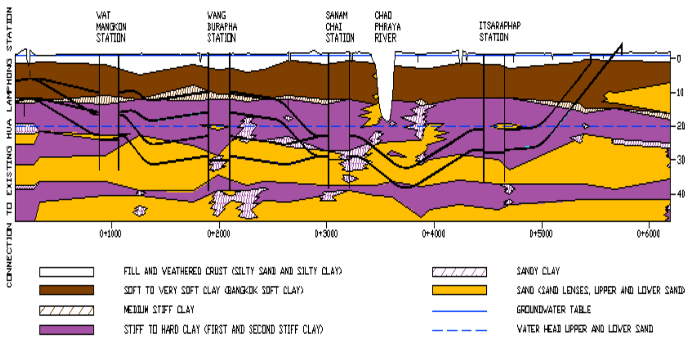


Figure 5 Soil Profile along Blue Line Extension Underground

Facing the situation of higher water pressure in the 1st and 2nd sand layers than that indicated by early investigation, the detailed design of two of the three station boxes that were placed at large depths needed to be modified from those planned in the pre-tendered design. This is to avoid problems of base instability during excavation from hydraulic uplift of the stiff clay at the excavation base. The excavation procedure adopted for the third station remained the normal approach as the designer considered it was still adequate. Unfortunately, after a portion of the base area was excavated and concrete casting, the hydraulic uplift occurred in the next stage of the base excavation that resulted in flooding and difficulty in control of water ingress in the remaining stages of the excavation. The work suffered almost 20 months delay in the construction to fight with the persistent problem of groundwater ingress despite extensive remedial grouting exercise. A brief outline on the situations of hydraulic instability involved with the base excavation design and construction of these stations is given in the following sections.

4.1 Difficulty and problem in station excavations

The first station excavation of BLE that needed modification in the design was Station BS12 which was 225 m long, 25.8 m wide and 32 m deep. The diaphragm wall was 1.2 m thick and 44.5 m deep. The cross section is shown in Figure 6. The detailed soil profile along the station excavation was derived from a comprehensive subsurface borehole investigation made after the award of the contract. A piezometric pressure was observed at approximately 15.0 m below the ground surface.

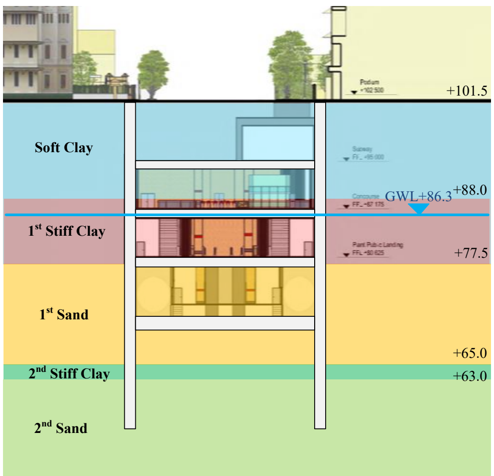


Figure 6 Cross section of BS12

Preliminary design analysis indicated that excavation to the base would experience hydraulic uplift failure owing to the high piezometric head in the 2nd sand layer underlying the thin 2nd stiff clay layer below the base of the excavation. Factor of safety against the failure considering the weight balance equilibrium was only 0.59 while the Outline Design Specification specified a minimum factor of 1.03. Thus the potential basal hydraulic uplift failure was the key factor in the design and construction and it necessitated an auxiliary measures. Among three possible measures, i.e. extending tips of the diaphragm wall to the 3rd stiff clay layer, the dewatering by deep well pumping and base grouting, the third method was adopted for reason of the most technical viable and cost effective one. The entire area of the excavation base was grouted. Two methods of grouting were adopted, i.e. chemical grouting by Tube-A-Manchette (TAM) and cement jet grouting. TAM grouting was chosen for the main part of station box excluding the north and south ends. Both methods were to create a watertight barrier in the 2nd sand to thicken an impervious plug below the excavation base to counteract the hydraulic uplift pressure as well as to improve side shear resistance of the soil with the side of the diaphragm wall. The U-shaped grouted zone as shown in Figure 7 was adopted. The grouting work was launched underneath the ground surface from a platform at the mid-height of the station height. The scheme was proven to be an effective method to prevent the hydraulic uplift failure of the excavation base and it allowed a normal construction process of the concrete base slab casting under a dry condition.

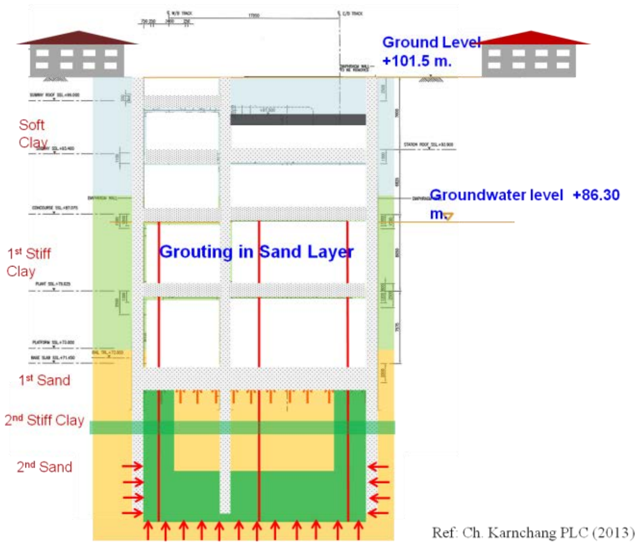


Figure 7 U-shaped base grouted zone of BS12

The second station was Station BS13 which was 181 m long, 23.4 m wide and 30 m deep. The diaphragm wall was 1.2 m thick and 40 m deep. Cross section and subsoil is shown Figure 9. The piezometric head was observed at approximately 16.0 m below the ground surface.

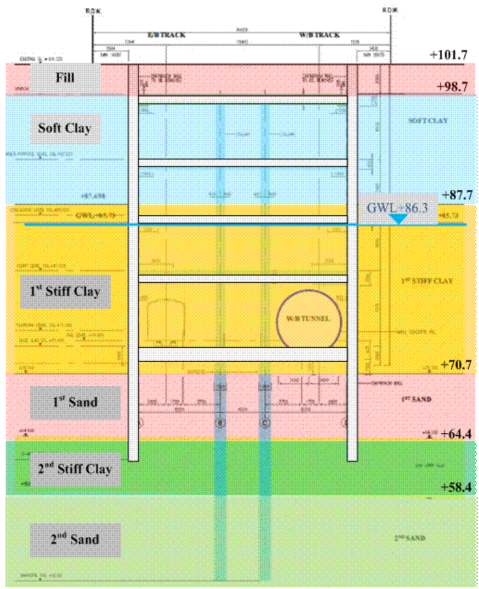


Figure 9 Cross section of BS13

Similar to the condition of BS12 excavation, potential base instability against hydraulic uplift was faced. However, the situation was not as critical. Factor of safety against the failure considering the weight balance equilibriumn was marginally at 0.96. In addition, the 2nd stiff clay layer at BS13 was not as thin as in the case of BS12, (i.e. 6 m). Therefore, a different measures was adopted. To minimize time and cost impacts on the construction work, the normal approach without implementation of auxiliary ground improvement measures was considered and finally adopted. The fundamental of the method is that if the base excavation is made in stages of a small sized excavation zone bounded by high soil surcharge + cast concrete slab around the perimeter of the excavation, the stability of the excavated base against the hydraulic uplift will be significantly assisted by the side shear resistance of the soil along the perimeter as compared with that of large excavation area. The Outline Design Specification stipulated at that in case shear strength of base soil was considered, a minimum factor of safety was required at 1.20. This requirement could be satisfied for the soil and groundwater condition below the excavation base of the station. The accurate determination of the density of the soil layers and the spatial variation of the thickness of the stiff clay layer were a must in the adaptation of this method. The staged excavation sequence adopted for BS13 excavation is shown in Figure 10. The work was successfully completed as planned.

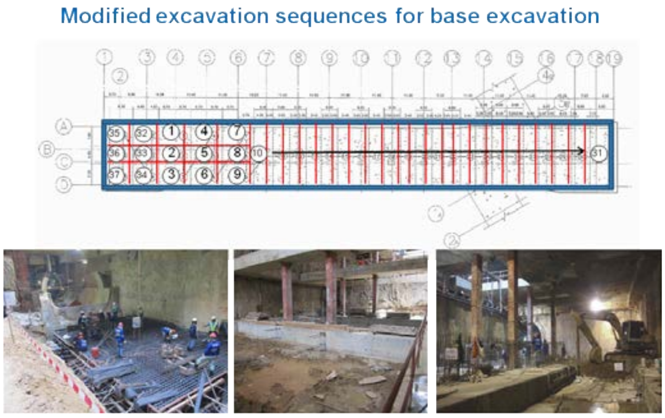


Fig. 10 Sequence of staged base excavation of BS13

Successful base excavation of the BLE Underground station was not the case at BS11 Station which was the deepest excavation among all MRT stations in Bangkok. The base of the excavation was at 32 m below the ground surface. Prior to the start of the excavation it was judged by the designer that the subsoil and piezometric head in the sand layers data indicated that the base excavation could be with adequate factor of safety against hydraulic uplift. Unfortunately, after the excavation was made to the lower level of the bracing of the D-wall and base excavation and concrete base slab casting was progressed from one end of the station box to about 15 percent of the length of station, significant water ingress occurred that seriously affected the work. The whole station was flooded by 2-m-depth of water within 12 hours. Extensive dewatering pump system was installed and a comprehensive program of chemical and cement grouting subsequently employed. The amount of ingress was as high as 1400 cubic meter/hour at one stage. Groundwater ingress repeatedly occurred in the subsequent stages of excavation for concrete casting that altogether delayed the work by 20 months. The investigation pointed that there might be a condition of lateral non-continuity of the 2nd stiff clay below the excavation base that in the initial design was considered sufficiently thick and covering the entire area of the excavation base. This incident of severe and prolonged groundwater ingress also led to subsidence problem of the surrounding ground and buildings that necessitated extensive stabilizing grouting works throughout the period.

4.2 Water ingress in EPB shield tunnelling

Prior to the tunnelling in the Blue Line Extension project, bored tunnelling by means of EPB shields and concrete segmented ring support had been commonly used with great success including those in the Blue Line Initial System project, in which the tunnelling was made deep in the sand layer at depth as much as 25 m below the ground surface. There had been no incidents of groundwater problem in EPB shield driving in Bangkok sands. However, in the BLE project that the EPB shield tunnelling had to be made through the sand layer as deep as 37 m below the ground surface with the piezometric head 10 m higher than that existed during the time of the first MRT shield tunnelling, an incident of serious water ingress occurred in August 2013 during the early drive of an EPB shield through the interface between the 1st stiff clay and the 1st sand layer. The consequence was flooding and severe damage to the TBM that led to an interruption of the work for over 7 months. The incident was an uncontrolled water ingress into the tunnel in the tail area of the EPB shield machine. Flooding and excessive movements of segmental rings in vicinity of the shield were experienced. It was reported the ingress initially occurred through a 20 mm wide gap formed at the circumferential joint of the key segment placed near the invert of the ring. The groundwater ingress from the 1st Sand layer shortly led to piping of fine sand into the tunnel, and subsequently excessive movements and distortion of several segmental rings behind the shield. During the incident various remedial grouting measures were urgently carried out to control the ingress and avoid excessive distortion of the concrete segmental rings that might ultimately result in a tunnel collapse.

It was obvious that the incident was caused by difficulty of the EPB shield in controlling stability of the excavated tunnel face while the tunnel was being excavated in the mixed face stiff clay and sand layers. At that position the groundwater head in the sand layer was 17 m. The sand was in the lower part of the tunnel and as the tunnel was progressively advanced, the portion of sand in face area progressively increased as the TBM drive pitched down. This condition was one of the most difficult tunnel grounds to deal with in soft ground tunnelling. It was apparently that with high ground water pressure in the sand layer in the excavated tunnel face, the shield machine experienced difficulty in maintaining adequate face pressures. Thus over excavation would significantly occur. After a careful examination on the TBM operation data it was believed that the triggering factor of the incident was an excessive displacement of the key segment of the segmental ring resulted from the over excavation of the mixed face soil by the TBM under the high groundwater condition.

5. ORANGE LINE EAST

The 3rd Phase MRT Underground work that is under construction at this stage is a part of the Orange Line East project (Cultural Center to Minburi) that is implemented as design-build contracts. The underground portion constitutes 12.1 km length out of the total 21.1 km length of the line. There are 10 underground stations along the underground section that the works are administered in 3 contracts (E1-E3). The underground line is mainly constructed by bored tunneling using 3 EPB shields except for a short cut-and-cover startup section. Owing to the tight space along the corridor of the alignment of the line that traverse along two most congested roads of the city that also have foundation of elevated roadway along the right or way, the MRT underground twin tunnels are mostly placed in the vertically stacked configurations and in many places they need to be placed at very close proximity to each other, particularly when leaving or entering the station boxes (Figures 2 and 11). The tunnels also pass very near to or cross foundations of existing buildings or other structures, some of which need underpinning works. The close proximity shield tunneling works for the deeper section of alignment in the sand layer of high groundwater pressure for the shallow section in soft clay layer necessitate special considerations in the design and construction planning to avert instability problems during construction and to ensure stability of the completed structures in long term. Because of the adoption of the vertically stacked configuration of the running tunnels most of the underground stations are excavated at large depth. And with the very extraordinary thickness of the 1st sand layer in the subsoils in the considerable part of the alignment, careful considerations are needed for the design and planning of the station excavation to avert potential basal instability problems as earlier experienced in the previous project.

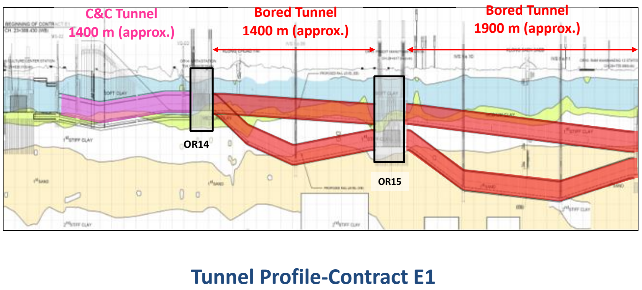


Figure 11 Vertically stacked alignment of tunnels in Contract E1

5.1 Close proximity tunnelling

The design of the bored tunnelling and concrete segment lining has to deal with the situation of very close proximity placement of the twin tunnels in various places. The clear distance between the tunnels is as close as 1.6 m which is only 25% of the tunnel diameter (e.g. Figure 12). For such conditions where the tunnels are to be excavated in the 1st sand layer, special measures are provided to avert potential instability problems to the lining of the firstly excavated tunnel from the TBM weight and any unexpected excessive over-excavation during the excavation of the second tunnel. The protective measures generally consist of ground improvement by jet cement grouting. Where the ground space does not permit the launching of ground improvement, other measures are used. They include installation of steel ring bracing inside the concrete segmental lining of the first driven tunnel, the careful driving of the shield, and the close monitoring of the movements of the lining and ground.

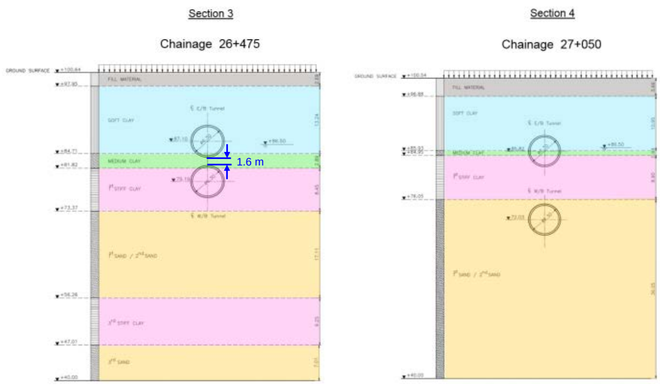
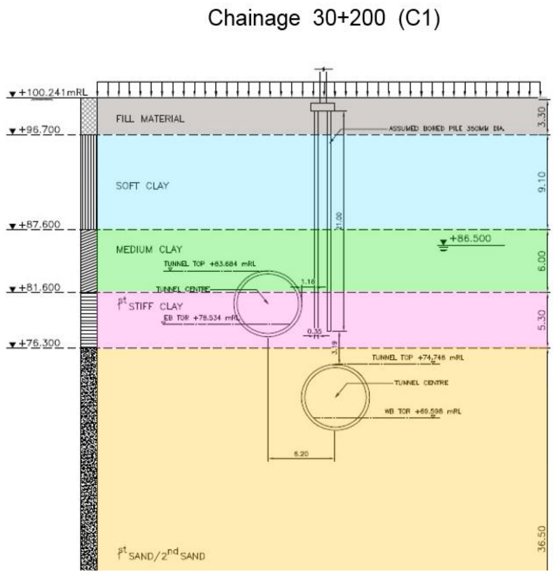


Figure 12 Example of close proximity shield tunnelling

Close proximity tunneling of the works also need to deal with the situation of potential impacts to the piled foundation of the existing structures and vice versa the impact of the load to the stability of the tunnel lining. Extensive numerical analyses are employed to check the situation of all cases in details and necessary preventive and corrective measures are provided in the design. In some cases, the excavation may be made without the need for auxiliary measures, but in some cases, ground improvement by jet cement grouting is adopted (Figure 13). In the worst case, underpinning with pile replacement needed adopted.

Besides the cases of close proximity tunneling, there is a case of the station box excavation placed very close to the piers of the elevated expressway at a clear distance between the diaphragm wall and the foundation piles of the pier as small as 2.7 m. In this case the impact assessment with the aid of rigorous numerical analysis (PLAXIS) suggests implementation of the mitigation measure to control the lateral movement of the viaduct pier and induced flexural stress in the piles within the allowable limits by using preinstalled struts for the station excavation at two levels by means of jet cement grouting (Figure 12). In addition, an extensive instrumentation program is employed to closely monitor the response of the viaducts and ground movements at all stages of the station excavation.



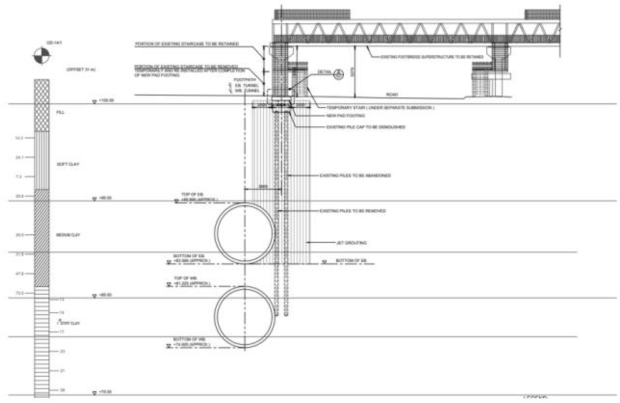


Figure 13 Examples of close proximity tunnel to existing piled foundation and underpinning by jet cement grouting

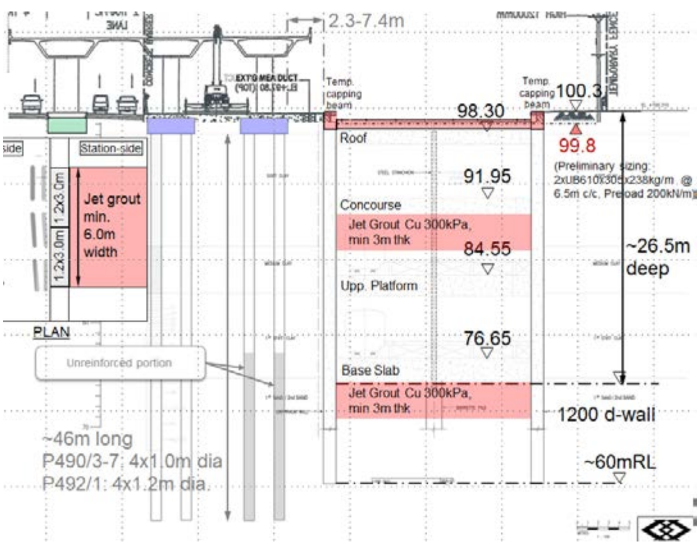


Figure 14 Pre-installed struts for station box excavation next to expressway viaducts

5.2 Underpinning work at crossing with existing Depot Access

A difficult and challenging underpinning work of the project is at the crossing of the cut-and-cover tunnel construction with the existing cut-cover-tunnels of the 1st Phase Blue Line Initial to the depot yard. The existing tunnels are at shallow depth and have piled mat foundation. Because the existing depot structure permits only 6 mm of movement of the rail structure, a careful consideration in the planning and design of the underpinning works in the limited access space is made. Methods of underpinning initially considered included the use of pipe roof jacking technique, jet cement grouting over the entire area, etc. However, due to the presence of many piles underneath the existing depot structure, the pipe jacking was not used. The underpinning scheme needs to employ combination of methods including barrette pile installation, soil improvement by jet grouting of large diameter columns and soldier pile wall excavation underneath the existing structures, etc. The section of the crossing is shown in Figure 15.

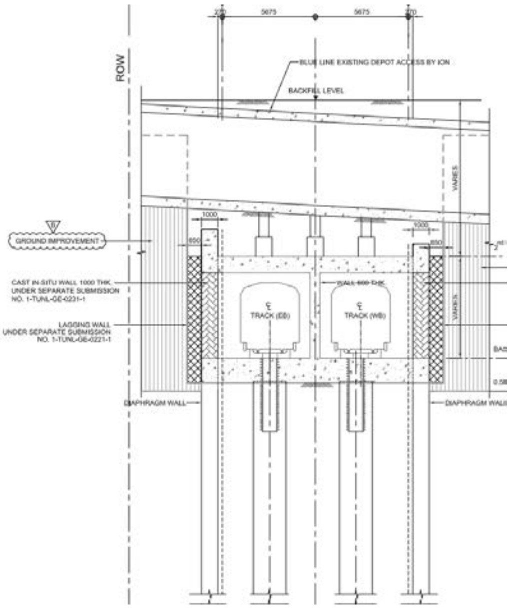


Figure 15 Underpining work at Depot Access crossing

5.3 Measures to deal with potential hydraulic uplift in excavation of station boxes

After thorough assessment on the alternative measures to deal with potential hydraulic uplift and water ingress problems at the base of the excavation of various deep station boxes of the project, it has been decided to adopt the option of positive cut-off of the diaphragm walls by extending the tips deeper below the base of the excavation to the 3rd stiff clay layer. Although this measures calls for a much larger depth of the diaphragm wall than the base grout plug alternative, it is judged that the ease and time duration of construction are more favorable. Figure 16 show examples of the soil conditions and design of the diaphragm wall tip for uplift prevention.

6. CONCLUSION

The MRT Underground works in Bangkok have faced increasing difficulties and challenges owing to the situation of rebound of groundwater level in the subsoils and the works in the remaining projects are situated in congested narrow roads of the city where there are numerous obstructions along the corridor. Therefore, the design and construction planning need to be carefully made to avert undesirable instability incidents in both deep station excavation and shield tunneling similar to that had recently occurred in the previous projects. publication in the SEAGS & AGSSEA Journal. We look forward to receiving your paper contribution.

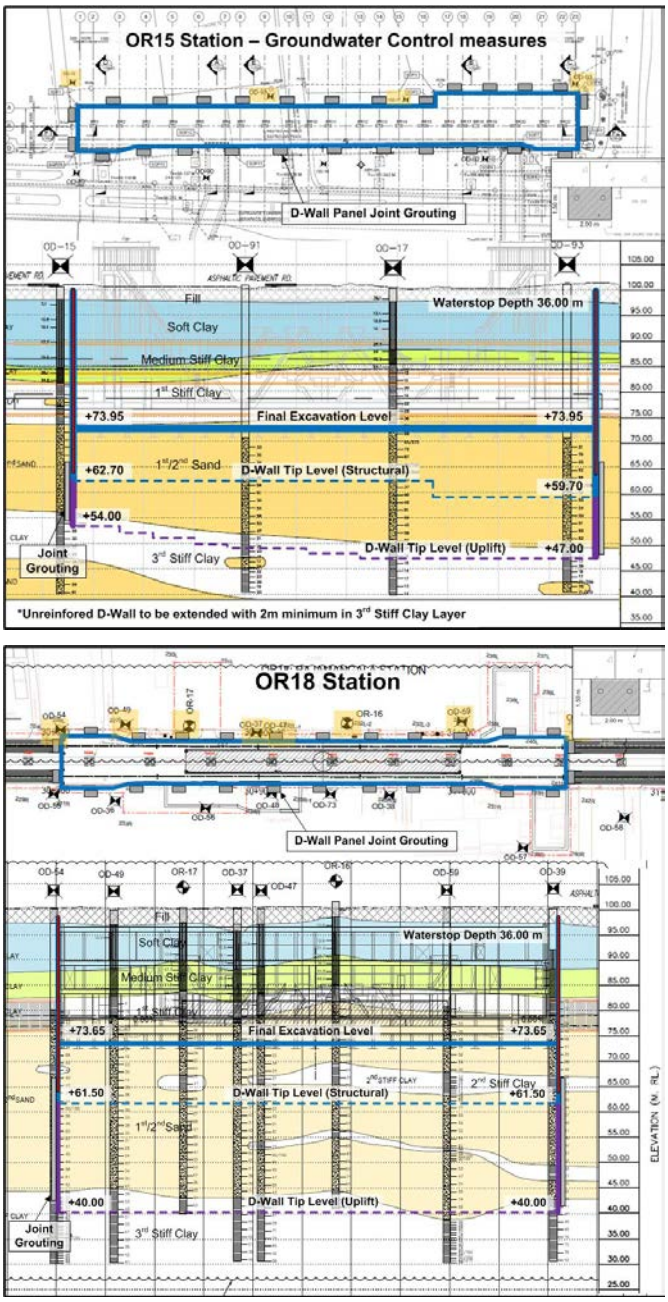


Figure 16 Examples of designed diaphragm wall tips for uplift prevention at two station boxes

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