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Assessing the Impact of Pipe Box Tunnel Installation with dual MTBM and Excavation Performance in Singapore's Soft Soil

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ABSTRACT: Trenchless construction methods using pipe box tunnels are often selected to construct underground structures located within a built-up matrix of infrastructures, utilities, and heavily trafficked roads. It could be disastrous if the displacement and stresses within the systems were not fully understood. This is especially true if the soil is varied and weak. This paper presents the challenges of the installation of trenchless pipe box tunnel using dual micro-tunnelling machines (MTBM) concurrently and the excavation with slope stabilising measures for the pedestrian underpass beneath Shenton Way of Singapore. The possible numerical approach to simulate the combined impact of MTBM installation and excavation will be discussed. The results will be presented together with the instrumentation monitoring data.

KEYWORDS: Trenchless construction method, pipe box tunnel, pipe roof, MTBM, slope stabilising measures, horizontal jet grouting

1 INTRODUCTION

Singapore's underground infrastructure has been growing with highly dense integrated network of Mass Rapid Transit (MRT) lines, roads, underground utilities, vehicular and pedestrian underpasses. As the city continues to expand, it is crucial to enhance the underground pedestrian connectivity, to reduce surface congestion and optimize land use. Often, these newly design underpass are located underground, so the pedestrian can move around seamlessly with all-weather comfort. The traditional cut-and-cover method is not feasible to construct underground structures in highly urbanized areas, where sensitive utilities and congested roads cannot be easily diverted or relocated due to the extensive time and cost associated with traffic management and mitigation measures. This challenge is further aggravated by complex geological conditions in shallow, varied, and weak soil conditions. Thus, trenchless construction methods using pipe box tunnel (PBT) is more viable, ensuring stability while minimizing surface disruption.

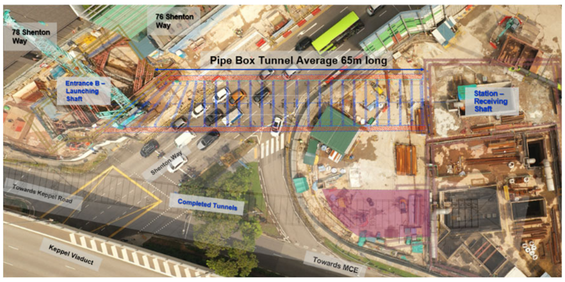

Figure 1. Pipe Box Tunnel Layout Plan at Shenton Way

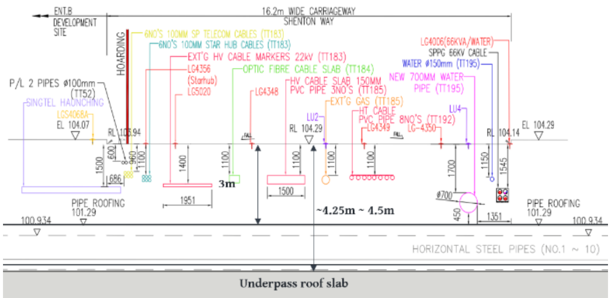

Figure 2. Existing Utilities within soil cover of Pipe Box Tunnel.

2 PIPE BOX TUNNEL AT SHENTON WAY

2.1 *Project Background*

An average of 68m long pedestrian underpass is to be constructed beneath Shenton Way, one of the major thoroughfares within the Central Business District (CBD) to connect the new MRT station to the commercial buildings. The underpass was designed with approximately 6.5m width and 9.4m height, with soil cover of 3m thickness. Figure 1 shows the pipe box tunnel layout underneath the Shenton Way.

Underneath the Shenton Way, numerous utilities was found including telco cables, gas pipes, water pipes and electric cables. It is not possible to relocate or divert all the utilities to install retaining wall vertically and commence cut-and-cover excavation. Hence, trenchless construction method using pipe box tunnel is adopted without disruption to traffic and various utilities. Figure 2 shows the elevation view of the high density of existing utilities within pipe box tunnel soil cover.

2.2 *Envisaged Geological Condition*

The geological soil profile along the pipe box tunnel is shown in Figure 3. Generally, the soil profiles of pipe box tunnel comprise 2m to 6m thick sandy silt Fill layer, followed by soft marine clay and estuarine clay, also classified as Kallang Formation varying between 4m to 9m thick. Underlying the Kallang Formation, we found completely weathered sedimentary rocks of Jurong Formation. This area was also once part of the shoreline of Singapore before the land was reclaimed in the year of 1932. It is noted that this area was man-made backfilled with waste materials.

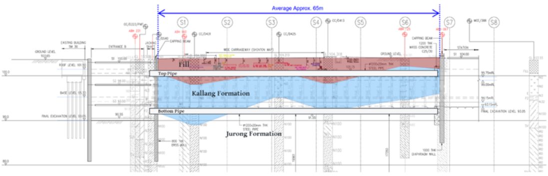

Figure 3. Envisaged Soil Profiles along Pipe Box Tunnel

3 PIPE BOX TUNNEL DESIGN

3.1 *Launching and Receiving Shaft*

PBT comprises a series of interlocking steel pipes installed using pipe jacking method, percussion method or MTBM to form a rectangular box-shaped structure. Upon completion of the pipe box, excavation works are carried out horizontally within the PBT to construct the permanent underground structures.

PBT installation requires launching and receiving shaft from both ends. Typically, launching shaft will be constructed with bottom-up excavation sequence using vertically installed retaining walls and lateral strutting system. As for receiving shaft, its construction sequence would have to be aligned with the overall main underground works for the purpose of site coordination and planning. Generally, ground improvement blocks are installed at both ends of the PBT to prevent soil and water ingress during the MTBM break-in and break-out. This also allows the MTBM commenced upon the completion of receiving shaft, with its cutter head socketed in the ground improvement blocks at the receiving shaft. In this project, mass concrete wall was installed to form a rigid interface that can effectively minimise the MTBM-induced ground settlement near to the receiving shaft. Figure 4 shows the sectional views of launching and receiving shaft for pipe box tunnel at Shenton Way.

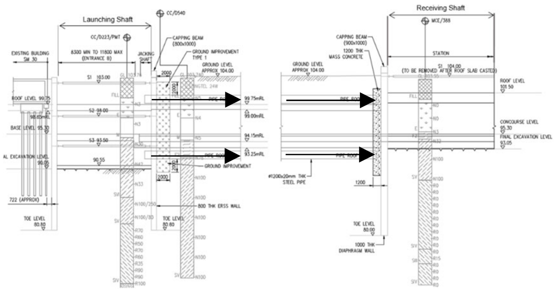

Figure 4. Sectional View of Launching and Receiving Shaft at Shenton Way

3.2 *Micro-Tunnelling Works*

MTBM shares similar key features with slurry TBM, where both are shielded with closed face system using bentonite to support the tunnel face against the earth and ground water pressures. Every MTBM-driven pipe installation induces ground movement, which are most effectively to install the pipes in top-down sequence. The first row of pipe has the lowest overburden, potentially inducing the most of total cumulative ground movement. The subsequent rows of MTBM-driven pipe installation induce smaller ground movement due to the beneficial effect of the structural confinement of first row. MTBM's key operation comprises the following:

- Cutterhead speed and torque controlled by operator to tackle the complex soil conditions.
- Thrust forces where hydraulic jacks push the MTBM advancement.
- Slurry suspension to maintain face stability in the closed mud chamber behind the cutterhead. This will form filter cake on excavation face that holds the loose soils and against the groundwater and earth loads. The operator adjusts the slurry pump rates so that the pressure remains within predetermined bounds.

In response to the anticipated geological uncertainties, the first MTBM machine was mobilised to install the steel pipe from No.1 to No.4 as a pilot run. This stage provided crucial performance data under actual ground condition. It also allowed MTBM operators to get hands-on experience with the actual site geological condition behaviour. In this project, a second MTBM machine was deployed to operate in parallel after the 4th pipe installed.

The clearance between the existing utilities and permanent roof slab is approximately 2.2m. After careful consideration of the space constraint and ground settlement that could occur during MTBM-driven pipe installation, a total of 32 numbers of 1.2m diameter steel pipes were designed to form the rectangular box-shaped pipe box tunnel. Figure 5 shows the cross-section view of the pipe installation.

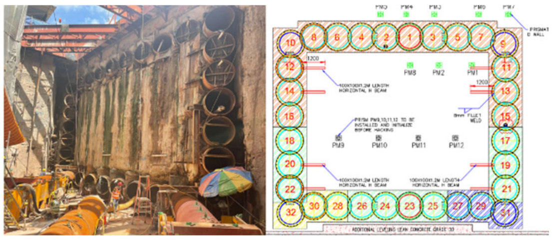
Figure 5. Cross Section of Pipe Installation using Dual MTBM

The receiving shaft excavation had not been completed at the time of pipe box installation, thus, retractable MTBM with outer diameter of 1.24m was adopted to install all the interlocking steel pipes. The MTBM is equipped with two foldable arms and a winch system, allowing the machine to retract and reinsert itself during the pipe installation. The steel pipes are interlocked using C-T joint mechanism with the purpose to prevent water/soil ingress and maintain the pipe alignment to avoid declutching. Foam fillers were applied within the voids of the "C" in the C-T joints. Figure 6 shows the cutter head of MTBM and the C-T joint interlocking mechanism of the pipes.

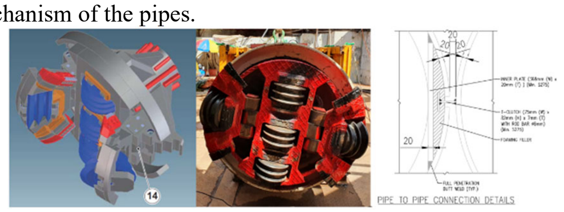

Figure 6. Retractable MTBM machine and C-T Joint Interlocking Mechanism

3.3 *Pipe Installation Sequence*

The MTBM-driven pipe installation sequence started with installation of slurry plant for soil conditioning and spoil removal. At the launching shaft, the retaining wall was systematically hacked to facilitate the fixing of steel entrance ring, which is embedded into the ground improvement block. For the reaction wall, reinforced concrete was constructed against the launching shaft temporary retaining wall. After completing the reaction wall, temporary working platform with a hydraulic jacking frame was erected on the reaction walls. MTBM gate frame was also installed to ensure dimensional control and pipe alignment.

After the first pipe reached the designed length and was socketed into the mass concrete wall, the MTBM machine was retracted with the outer shield left-in. Each steel pipe was equipped with grout outlet nozzles, for the purpose of annulus grouting to seal the overcut gaps. The annulus grouting was to be carried out progressively at every section of the steel pipes.

All the steel pipes were also infilled with concrete grade C32/40 for the robustness prior to commencing excavation. The infilling was carried out progressively, to minimise the immediate settlement induced by the self-weight of infilling all pipes at the same time. The first stage of concrete infill was carried out for pipe no.1 to 4 after MTBM works and annulus grouting up to pipe no. 8 as illustrated in Figure 5. The second stage of concrete infill was carried out for pipe no. 7 to 8 and 11 to 14 after MTBM works and annulus grouting up to pipe no. 16. The remaining pipes were infilled with concrete after completed all the MTBM works and annulus grouting.

3.4 *Excavation within Pipe Box Tunnel*

The excavation commenced upon the completion of the pipes installation and ground improvement, where required to strengthen the weak soil area. . During the excavation stages, internal slope needed to be as steep as possible to reduce the unsupported span of the pipe box tunnel while steel frames were installed at intervals. To ensure the slope is maintained safely, slope strengthening measures were required. The permanent structures was constructed in sequential manner before the steel frames support was removed at intervals. This process was repeated until the full length of PBT was excavated. For the pipe box tunnel of Shenton Way, throughout the 68m average length, 26 nos of steel frames were installed at 2.5m spacing. Figure 7 shows the slope gradient and the supporting steel frame within PBT in Shenton Way.

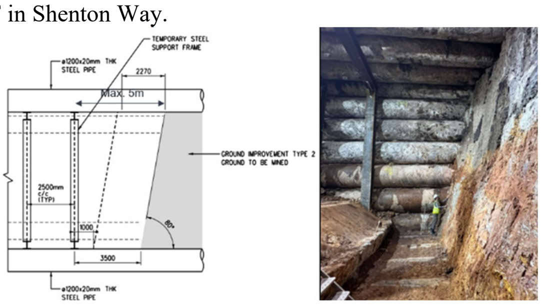

Figure 7. Slope Design within PBT

3.5 *Slope Strengthening Design*

As Kallang Formation was found within the PBT in the site investigation, slope strengthening measures using ground improvement was designed. Due to site constraints, ground improvement works for the PBT were carried out vertically for the areas near to both shafts, while horizontal grouting was applied at the center area of PBT. Horizontal grouting was selected due to the extensive services underlying the road, rendering it highly challenging to carry out surface grouting. The horizontal grouting zone is approximately 35 m in length, installed in two stages from the launching shaft using double tubed jet grouting system. The design strength requirements were Cu = 300 kPa and E’ = 150 MPa for all the ground improvement work. Permeability of the grouting area is designed as minimum k = 1x10-7 m/s. Trial test on the vertical grouted area was performed before commencing the excavation works.

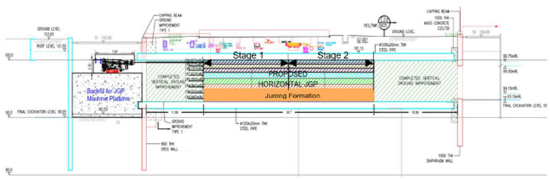

Figure 8. Proposed Ground Improvement Sequences

4 NUMERICAL APPROACH

Finite Element Method (FEM) is used to simulate the sequence of installing the pipe box tunnel using MTBM. The impact of the installation to the surrounding ground as well as to the existing utilities could be analysed via FEM. FE software used in this project was PLAXIS that allows various construction sequences to be modelled, thus allowing a complete understanding on ground movements, pipe box deflections, bending moments and shear forces developed at each stage of construction. The software is commercially available and has been well tested.

4.1 *Analysis Consideration and Parameters*

All soils were modelled using elastic perfectly plastic soil material satisfying the Mohr-Coulomb failure criteria. Fill and Jurong Formation (sedimentary rocks with completed fractured condition) were defined as drained materials while marine clay (M), estuarine clay (E) and fluvial clay (F2) were defined as undrained materials. Effective stress parameters were used to define the stiffness and strength of Fill and Jurong Formation, whereas undrained shear strength and stiffness adopted for soft clay layers.. Traffic loading of 25kPa and construction loading of 32kPa were considered in the analysis. Summary of soil parameters used in the FE analysis are shown in Table 1.Table 1. Summary of soil parameters.

Soil Type	Unit Weight	Strength and Modulus Parameter			
		Undrained Shear Strength, C_u	Cohesion, c	Friction Angle, ϕ’	Effective Elastic Modulus, E’
	kN/m^3	kPa	kPa	deg	MPa
Fill	20	-	2	30	10
M	16	15	0	22	261C_u
F2	19	20	0	26	261C_u
E	15	1	0	24	261C_u
SVI	20	5N	5	30	1.74N
SV SPT N ≤ 50	21	5N ≤ 250	5	33	1.74N
SV SPT N > 50	21	5N ≤ 250	10	34	1.74N

4.2 *Cross-Section Analysis for PBT*

To evaluate the combined effects of ground movement induced by MTBM works and the subsequent excavations in stages, first, a transverse (cross-section) analysis simulated the soil response and immediate ground movements induced by the MTBM-driven pipe box tunnel installation.

As a first-degree estimation for the ground movement, the potential volume loss (%) and settlements can be estimated based on the overcut closure. In this case, for the installation of one steel pipe at the elevation of first row of pipe, max. 35mm and 7% were calculated. For total cumulative effects of the MTBM-driven pipe box tunnel induced settlement, numerical analysis was adopted to simulate the installation sequence with dual MTBM for all 32 pipes. Three modelling approaches were studied using Plaxis 2D including stress reduction, line contraction and grout pressure to simulate the MTBM-induced ground movements.

- The stress reduction method simulates excavation by releasing in-situ soil stresses around the MTBM, allowing the surrounding soil to deform. This method is also named as the β-method as described by Panel and Guenot (1982). Its accuracy relies on geological experience to determine an appropriate stress release percentage.
- The line contraction method, as described by Vermeer and Brinkgreve (1993), involves deactivating the soil cluster within the pipe and simultaneously activating the tunnel lining. A contraction strain is applied to the lining, representing the estimated volume loss induced by MTBM excavation. For PBT with 32 pipes, it may be too conservative to assume every individual MTBM-driven

pipes experiencing 7% volume loss based on the overcut gaps.

- The grout pressure method, based on the work of Möller and Vermeer (2008), deactivates the soil cluster within the pipe and applies a prescribed face pressure to simulate slurry support during tunnelling. This method requires the establishment of the target operating pressure using chart-based method prior to prescribing the face pressure for each row of MTBM-driven pipes.

The grout pressure method was adopted in this project to simulate MTBM operation pressure and its cumulative impact onto the surrounding soil movement. Figure 9 shows the cross-section analyses done for the design stage.

The grout pressure to be applied for MTBM was established using chart-based approaches for effective stress calculation (Anagnostou & Kovari, 1996), and total stress calculation (Kimura & Mair, 1981). The controlled operation pressures are in the range from 0.4bar to 0.9bar for the 1st row of steel pipes and 1.2 bar to 2.1bar for the last row of steel pipe. Figure 10 shows the modelling phases carried out to capture the concurrent MTBM and concrete infilling works.

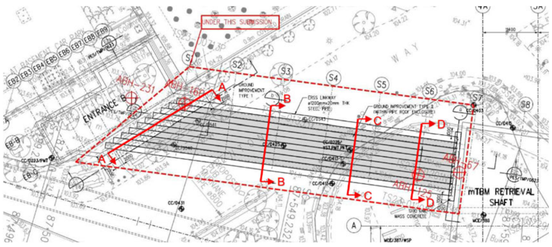

Figure 9. Design Cross Sections Along PBT at Shenton Way

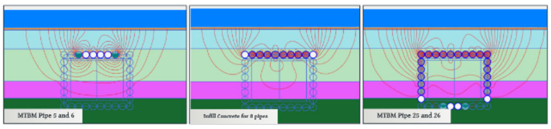

Figure 10. Typical Cross-Section Analysis for concurrent MTBM and concrete infilling works.

4.3 *Longitudinal Analysis for PBT*

To assess the steel pipe design and ground response induced by PBT excavation, a longitudinal 2D model was developed to simulate the varying soil profiles and complex construction stages. A total of 25 sequential excavation stages were simulated, with each subsequent excavation stage supported by a steel frame installation. The steel frames were numbered in a ascending manner from the receiving shaft (No.1) to the launching shaft (No. 26). Prior to excavation commencement, horizontal grouting works were simulated, enabling the excavated slope to form 80-degree gradient. The model computed the cumulative steel pipe settlement resulting from the sequential excavation and steel frame support installation phases, thus, prompting design refinement and migration measures.

The model setup assumed both the launching and receiving shafts have been excavated to the final MTBM working levels. The soil profile and mesh were generated to reflect the soil actual stratigraphy. The steel frame support was modelled as node-to-node elements, and the steel pipes was modelled as plate elements. The excavation geometry was drawn as per the design slope gradient. The analysis phase of excavation and steel frame support installation were captured in separate phases to simulate the actual site construction stages. Figure 11 shows the longitudinal analysis model adopted in the design.

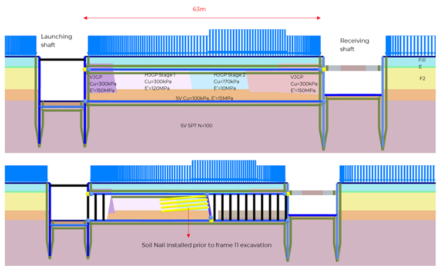

Figure 11. Longitudinal Analysis Model

The construction sequences were modelled as follows:

- Phase 0: Wish-in-place modelling of the shafts
- Phase 1: Remove portal walls at both shafts
- Phase 2: Install steel frames No. 1 and No. 26
- Phase 3: Two-way excavation within the vertical grout zone, advancing to frames No. 2 and No. 25
- Phase 4: Install frames No. 2 and No. 25
- Phase 5: Continue alternating excavation and frame installation—launching toward No. 3, receiving toward No. 24—throughout frames 3–24
- Phase 6: Cast permanent underpass span (frames 24–26) in parallel with excavation to frame No. 4 at the receiving shaft
- Phase 7: Advance receiving-shaft excavation to frame No. 23
- Phase 8: Complete underpass concrete works and remove central support.

4.4 *Combined Impact of the MTBM Operation and Excavation*

With both transverse and longitudinal analyses models, we can decouple and quantify independently, the ground settlement resulting by the dual MTBM pipe installation and excavation-induced impact, yielding more precise predictions of total ground movement. The dual MTBM-induced impact onto the surrounding ground was assessed using grout pressure method, from which the maximum settlement was predicted. A subsequent PBT excavation-induced impact was analysed using longitudinal analysis. The resulting settlement profile indicates that the max. ground settlement occurs at the interface between horizontal grouting stage 2 and vertical grouting, near to steel frame No. 10. This finding concurs with the max. settlement identified in the cross-section analysis at the same location.

The max. predicted ground settlement in longitudinal analysis was used as the basis for cross section analysis to simulate the excavation effect with stress reduction method. The simulation of the combined effects of PBT installation and excavation activity using cross-section analysis comprised the following analysis steps:

- All 32 pipes installation effects completed.
- Deactivate the soil within PBT, adjust the M_{stage} to 0.2 (20% of soil stress relaxation), representing the internal support pressure(βP_k) acting on unsupported perimeter (1- $\beta 0 P_k$), considering the 3D slope effects in the 2D plain strain analysis.
- Compare the ground settlement against the predicted ground in settlement longitudinal analysis. If the predicted ground settlements are consistent, move on to next step. Otherwise, reiterate to adjust the M_{stage} to achieve consistent ground settlement.
- Steel frame modelled as plate elements are activated, remove internal support pressure and this plate element will take remaining load βP_k (80% of the stress acting on lining).

5 CHALLENGES ENCOUNTERED

5.1 *MTBM: Geological Risk and Operational Constraint*

MTBM operations in the first row faced several overlapping challenges. Shallow overburden with 3m thick fill layer above the pipe box elevates geological risk. MTBM operation is to be carried out with dense network of existing utilities immediately above the pipe box, while maintaining uninterrupted live traffic. Under these conditions, the TBM operator must precisely regulate cutterhead speed and torque, and simultaneously monitor slurry pressure to prevent excessive ground settlement or heave.

For the first row of MTBM works, slurry was found to have leaked into a nearby existing drain, and this has induced 12mm settlement. As a remedial measure, the existing drain was cleaned and inspected with the supervision team. All the leakage points were sealed with cementitious grout and provided with temporary internal supports. During the first row of MTBM, we also found the slurry circuit experienced blockages several times. It was observed that the obstructions included timber fragments, scrap metal plates, and rebars, that choked the line. To clear these blockages during MTBM works, operators incrementally increased cutterhead torque and rotational speed, which resulted in the increase of slurry pressure and caused a heave of 30mm recorded. as the exact locations of these obstructions could not be determined, the target slurry pressure was allowed to be lowered down to the ultimate limit-state (ULS) value of 0.4 bar, giving MTBM operators additional flexibility to manage potential blockages. Figure 12 shows the blockage encountered during MTBM works and reinforcement and timber fragments identified.

Figure 12. MTBM blockage and obstruction


Figure 13. Cutter head Condition after 1st row of MTBM.

The inherent geological risk and unforeseeable obstruction have caused the MTBM advancement to be highly erratic. To manage this, experienced operators must continuously monitor the slurry pressure, advancement rate and torque, for any abnormal spikes that indicate a blockage. At any sign of anomaly, site team was required to engage the project design team to evaluate the subsurface conditions and decide the appropriate remedial measures. In this project, several road repair works were carried out due to the observed cracks. Figure 13 shows the wear and tear of the cuter disc up to 5mm after the installation of pipe no. 10.

During the MTBM works of pipe No. 11 and 13, the adjacent inclinometer reflected lateral soil movement. The maximum recorded lateral movement reached 15.6 mm, moving away from the PBT alignment. This change in soil movement is attributed to the heterogeneous fill such as construction debris, timber fragments and tree roots, encountered. Figure 14 plots the inclinometer changes for Pipes 11 and 13 alongside the corresponding slurry-pressure record for Pipe 13, which remained within the prescribed target range. The data indicate that, even under controlled slurry-pressure conditions, debris-laden strata can impose unanticipated lateral stresses on the surrounding ground, displacing it away from the advancing TBM.

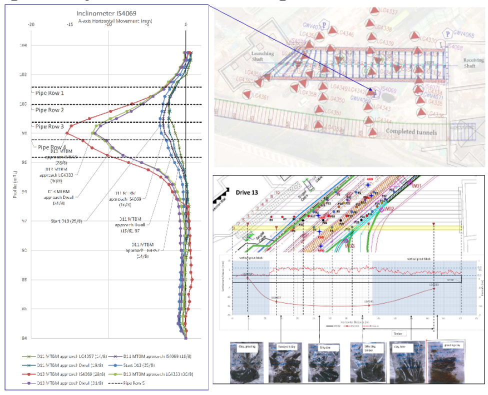

Figure 14. Measure Inclinometer Reading - MTBM Pipe No.11 & 13.

5.2 *Exposed Grouting Quality During Excavation*

During the excavation work, the exposed grouting within the Stage 2 horizontal grouting did not meet the expected quality. Figure 15 shows the exposed grouted surface of the slope near to the steel frame support No. 10. The area of grouted coverage and strength did not satisfy the design requirements. Field observations by geologists showed that only approximately 50% of the targeted soft clay was treated. Remnants of Bakau Piles were also observed on the slope surface. It was also informed that dead coral reef, debris and construction waste were found, and they had choked the slurry tubes during horizontal grouting works as shown in Figure 16. These unseen obstructions could cause the drill rods to be deflected arbitrarily; hence, the design interlocking of the horizontal grouting columns could not be formed. Field observation found that lower half of the slope was underlain by completely weathered sedimentary rocks, also named as Jurong Formation SV in Singapore.

Figure 15. Exposed Grouted Surface on the Slope at Frame 10.


Figure 16. Slurry returns during horizontal grouting works.

To verify the in-situ strength of these poorly grouted clay areas, additional coring was performed. Figure 17 shows all the coring test results for Stage 1 and Stage 2 horizontal grouting. [Need to mention the strength did not achieve target, before continuing with next sentence]. Advancing the excavation could result in the slope not being trimmed to the design angle and hence, soil displacement control could be compromised.

Owing to the construction space constraint, fiber glass soil nails were installed to enhance the slope stability before advance to the subsequent excavation. Re-analysis was carried out to simulate the fiber glass soil nails installation and reduced grouting strength parameters.

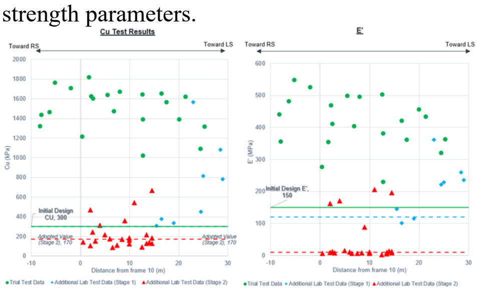

Figure 17. Coring Test Results of the Horizontal Grouting Area

6 RESULTS AND DISCUSSION

Prior to the MTBM works, ground settlement markers, utilities markers, piezometers and inclinometer in soil were installed to monitor the impact of the works. Figure 18 shows the measured settlements that occurred after all 32 pipes were installed and infilled with concrete. These field measurements, equivalent to 3.9% volume loss and max. 68mm settlement, correspond closely with the predicted settlement profile, considering the slurry-pressure adjustment implemented following the incident during the 1st row of pipe installation (Section 5.1).

During PBT excavation, the respective settlement points recorded a max. of 80 mm, equivalent to 4.9 % volume loss. The observed settlement trend and max. values agree closely with the predicted profile for Dual-MTBM tunnelling using grout-pressure control, followed by a stress reduction approach (Mstage = 0.27, corresponding to 27 % soil relaxation).

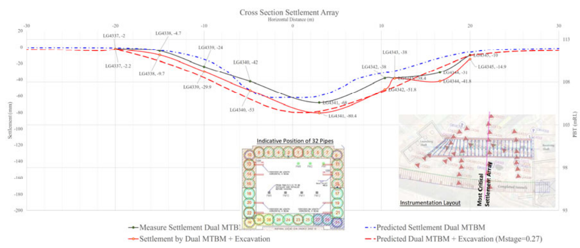

Figure 18. Predicted Cumulative Settlement and Measure Settlement (Cross Section)

Upon completion of the 32 pipes installation, vertical displacement of the steel pipe is monitored using real-time horizontal in-placed inclinometer (HIPI). The HIPI was installed at the top row, comprises 23 sensors at 3m spacing c/c. The HIPI is activated prior to commencement of excavation works. As the PBT will be excavated from both directions, 3D prisms are installed at the launching shaft to verify the fixity of the datum point adopted for the HIPI. Field observation confirmed that the excavation remained effectively dry with no significant water ingress. The piezometer readings fluctuated within the acceptable tolerance of ±2m. Figure 19 shows the measured HIPI settlement trend of the top row of PBT at all excavation stages. The max. recorded settlement is 15mm, occurring near Frame No. 12 position. The results are closely aligned with the predicted max. settlement of 23mm. The settlement profile remained well controlled throughout the excavation within vertical grouting area at both shafts. With the lessons learned for the heterogeneity of the fill, estuarine clay, marine clays, and construction debris encountered during MTBM operations, the construction sequence was optimised to include the casting of mass concrete base at each stage of steel frame erection. This measure established a stable working platform for the excavators and enhanced post-installation of steel frame ground stability, therefore, minimising the induced ground settlement.

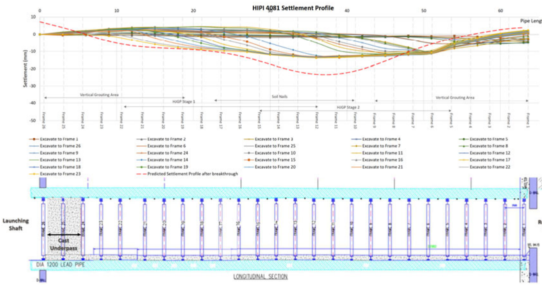

Figure 19. HIPI Settlement Profiles at all critical excavation stages

7 CONCLUSION

This paper discussed the design and construction of PBT for the underpass beneath Shenton Way, dealing with inherent fill heterogeneity comprising residual construction debris, timber, tree roots and dead coral. During the execution of MTBM and excavation works, mitigation measures implemented include dynamic slurry pressure control for the face stability, horizontal jet grouting to strengthen weak and varied soil and intermittent steel frame supports, later supplemented with fiberglass soil nail when exposed grout performance fell short.

For the ground response estimation, the grout pressure method emulated concurrent dual MTBM tunnelling sequences, while a staged stress reduction approach was used to simulate the excavation-induced soil relaxation. The field measured ground settlement and pipe box vertical movements are in close agreement with the design predictions, confirming the numerical approach reliability. This provides a validated template for future urban underpass projects faced with similar complex geological conditions and site constraints.

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