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Alternative approach for tunnelling face pressure

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ABSTRACT: With denser underground Mass Rapid Transit (MRT) network, some new MRT tunnels are much deeper than those in the completed MRT lines. For such deeper tunnels, the design approach for the tunnel face pressure should take into consideration both adequate face pressure to limit ground settlement and anticipated number of cutterhead interventions due to more wear and tear of cutter tools in hard ground. For deeper tunnels in competent and low permeable ground, there is merit to optimise tunnelling face pressure with appropriate mitigation measures. This approach could result in the lesser wear and tear of the Tunnel Boring Machine (TBM), as well as higher tunnelling progress, thus, promoting productivity while enhancing tunnelling safety. However, with lower face pressure, ground settlement risks will increase should unforeseen ground condition be encountered. If these risks are not properly mitigated, excessive ground settlement or damage to the adjacent building may occur. In this paper, a case study will be presented to demonstrate how observational method can be used for adoption of lower tunnel face pressure with appropriate and adequate measures to mitigate ground settlement risk.

KEYWORDS: Tunnel face pressure, observational method, bored tunnelling, case study, productivity.

1 INTRODUCTION

Bored tunnelling in Singapore's urban environment presents unique challenges due to the proximity of existing buildings and infrastructure and the presence of soft soils. Building and Construction Authority (BCA 2017, 2020, 2024) has put in place a robust tunnelling control framework with various risk mitigation measures to ensure the safety of bored tunnelling works in Singapore. Completed bored tunnelling projects in Singapore have predominantly adopted tunnel face pressure equal to or exceeding the full hydrostatic pressure, aligning with international design guidelines (e.g. DAUB 2016, GEO 2014).

As Singapore's underground rail and road network expands, new tunnels are being constructed at greater depths than previous lines. For deeper tunnels in competent and low-permeable ground, there is merit to optimise tunnelling face pressure which could result in the lesser wear and tear of the TBM, hence reducing number of cutterhead intervention, as well as higher tunnelling progress. This will promote productivity while enhancing tunnelling safety.

However, with lower face pressure, ground settlement risks will increase should unforeseen ground conditions be encountered. If these risks are not properly mitigated, excessive ground settlement or damage to the adjacent building may occur.

This paper presents an alternative approach for optimising tunnelling face pressure in competent ground with low permeability while maintaining safety standards. A case study is presented to demonstrate how the Observational Method can be effectively implemented to allow lower tunnel face pressures while incorporating comprehensive risk mitigation measures. The approach balances the competing demands of ground stability, construction efficiency, and safety requirements.

2 DEEMED-TO-SATISFY APPROACH FOR FACE PRESSURE

Designing appropriate face pressure is fundamental to safe TBM operation. Inadequate face pressure may lead to excessive ground movement and even instability of the tunnel face.

The face pressure must balance the water pressure, soil pressure and surcharge load. As illustrated in Figure 1, the face pressure should be set higher than hydrostatic pressure to limit the ground deformation and to keep the behaviour of the ground elastic. This aligns with the recommendation in various design guidelines (e.g. DAUB 2016, GEO 2014).

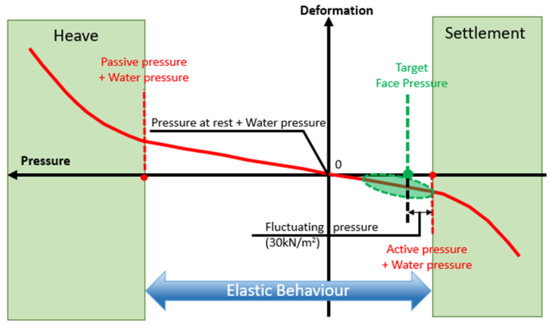

Figure 1. Schematic diagram of face pressure and ground deformation.

Singapore geology is complex despite its small area. Major geology groups include Jurong Group, Sentosa Group, Bukit Timah Centre, various stiff and soft soils formations. For shallower tunnels, soft soils or a mixture of stiff and soft soils are often present along the tunnel alignment. Adopting tunnel face pressure equal to or exceeding the hydrostatic groundwater pressure prevents groundwater flow towards the tunnel face. In turn, it minimises ground settlements due to groundwater lowering or decrease in pore-water pressure, especially for ground containing soft soils or highly permeable soils. This approach is suitable for most tunnelling projects in Singapore where the geological formation often varies along the tunnel alignment. It is known as “Deemed-to-Satisfy” approach in the tunnelling industry.

This approach has served well for past tunnelling in Singapore, particularly in built-up area where limited ground movement is desired.

3 ALTERNATIVE APPROACH FOR TUNNEL FACE PRESSURE VIA OBSERVATIONAL METHOD

Adequate face pressure is critical to maintain tunnel face stability and to control ground movement. However, higher face pressure will cause more wear and tear of the TBM cutter tools, particularly in hard ground. This will increase the number of cutterhead interventions (CHI), a recognized high-risk activity (Shirlaw et al. 2003). Furthermore, requiring high face pressure during CHI may lead to work safety issues for the construction staff. For tunnelling in hard soils, an optimal face pressure should balance between maintaining tunnel face stability and preventing excessive wear and tear of the cutter tools.

For new tunnels become deeper (>30m) in hard soils with low permeability, it may not be the best solution to adopt the Deemed-to-Satisfy approach, which involves designing the tunnel face pressure for full hydrostatic pressure.

The Alternative Approach presented here employs the Observational Method to enable safe tunnelling with face pressures below hydrostatic levels. This approach is particularly relevant for deep tunnels (>30m) in competent ground with low permeability, where the conventional Deemed-to-Satisfy approach may be conservative for such ground.

The key elements of this Alternative Approach include:

1. Comprehensive 3D numerical analysis incorporating coupled hydraulic-mechanical behaviour.
2. Advanced instrumentation and monitoring systems.
3. Protective measures to prevent damage of adjacent buildings.
4. Contingency plan.

3.1 *3D Numerical Analysis of Tunnel Face Stability*

Where the tunnel face pressure is less than hydrostatic pressure, there will be groundwater seepage towards the tunnel face. Designing tunnel face pressure in such situation requires a fully coupled hydraulic-mechanical stress analysis using 3D geotechnical numerical model. The analysis shall include detailed steps covering the tunnelling activities such as ring-by-ring tunnel construction, CHI and TBM stoppage or unforeseen condition. The analysis shall also employ advanced soil models to capture realistic ground behaviour. As full step-by-step 3D tunnelling analysis is time consuming, where appropriate, a simplified "Pseudo 3D" may be adopted, where the steps of TBM advance is not modelled (DAUB, 2016).

From the 3D analysis, the designer determines the allowable stand-up time to limit the ground behaviour within the elastic range. If the allowable stand-up time is shorter than the worst tunnelling rate, the designer should increase the target face pressure.

The designer should obtain the water ingress rate during CHI from the 3D numerical analysis and during construction, should verify the water ingress rate is within the allowable limit.

3.2 *Observation method and risk-mitigation measures*

Adopting face pressure lower than hydrostatic pressure involves some potential risks, such as risk of over-excavation due to seepage gradient, dewatering or lowering of pore-water pressure, and risk of unforeseen ground condition. The risk of over-excavation is mitigated via verification of excavation volume and action to be taken when there is over-excavation stipulated in BCA's tunnelling framework (BCA 2017).

Following the principles of Observational Method (OM) (EC7-1 Clause 2.7), comprehensive instrumentation and monitoring shall be provided to observe the actual performance, and a contingency plan shall be devised, to be adopted if actual performance is outside acceptable limits.

In addition to the conventional instrumentation and monitoring plan, the following additional monitoring is to be provided:

- Geophysical detection of tunnelling-induced voids. Modern TBMs can be equipped with sensors utilizing seismic scattering or other geophysical detection methods. Such sensors can provide an early warning of tunnelling-induced voids.
- Piezometers at the border of the determined tunnel influence zone, to verify there is no drawdown beyond this perimeter as assumed in the design.
- Adequate recharge wells that served as protective measures are pre-installed for existing buildings / structures located within the tunnelling influence zone. These recharge wells shall be activated when the pre-determined Alert Level of the piezometer is breached.

As a contingency plan, the operating face pressure shall revert to minimum hydrostatic pressure for the remaining of the tunnelling works if damage to adjacent structures/buildings occurred due to tunnelling works or building settlement markers breached the pre-determined Work Suspension Level.

4 CASE STUDY

A case study is carried out for bored tunnelling with TBM face pressures adopting the Observation Method (Alternative Approach) for three tunnel drives launching from the same shaft.

4.1 *Tunnel/ project description*

The project consists of bored tunnels of 7m outer diameter for a MRT project in Singapore, in reclamation area underlain with thick layers of soft ground. The depths of the bored tunnels varied from 20m to 40m. Ground improvement (GI) works had been carried out around the tunnels and along the tunnel drives to meet the required long-term settlement limits for the tunnel structures. The ground improvement was extended from the bottom of the tunnels to hard/dense layers with SPT N-values of greater than 30.

As the proposed tunnels are located within the ground improvement block, it satisfies the criteria of competent ground with low permeability and hence is allowed to adopt the Alternative Approach or the Observational Method. Studies on the design and analyses as well as the performance of the bored tunnelling works adopting Alternative Approach with optimised tunnel face pressure are carried out for three tunnel drives launching from the same launch shaft. The tunnel lengths are approximately 1.2km for each tunnel, with total tunnel length of about 3.6km.

4.2 *Geological condition and challenges*

The geological formations encountered along the tunnel alignments are as follows:

a) Reclamation Fill (Sandfill)
b) Kallang Group (Recent Deposits) consisting of Marine Clay (M), Estuarine Clay (E), Fluvial Sand (F1) & Fluvial Clay (F2)
c) Bedok Formation (Old Alluvium)

The reclamation fill layers were found covering the site with thickness typically ranging from 15m to 20m deep. The Kallang Group layers were encountered underneath the reclamation fill with thickness varying from 15m to 50m. The Bedok Formation was encountered below the Kallang Group layers.

<u>Tunnel 1</u>: The depths from the ground surface to the axis level of Tunnel 1 are about 25m at the launch shaft, increasing to a maximum depth of about 40m approximately at 400m from the launch shaft and reducing to about 20m at receiving shaft. Tunnel 1 was driving at a depth of more than 30m for a distance of about 600m between CH200 and CH800. The tunnel was mainly embedded in the Kallang Group layers with partially in the Bedok Formation for a short stretch and partially in the

reclaimed fill near the receiving shaft where the tunnel is at shallower depths.

Tunnel 2 and Tunnel 3: The depths to the tunnel axis for the tunnel drives vary from 20m to 28m below the ground surface. The tunnel drives are fully embedded in the Kallang Group layers.

Geological profiles along the tunnel alignments are shown in Figure 2 to Figure 4 for Tunnel 1 to Tunnel 3, respectively.

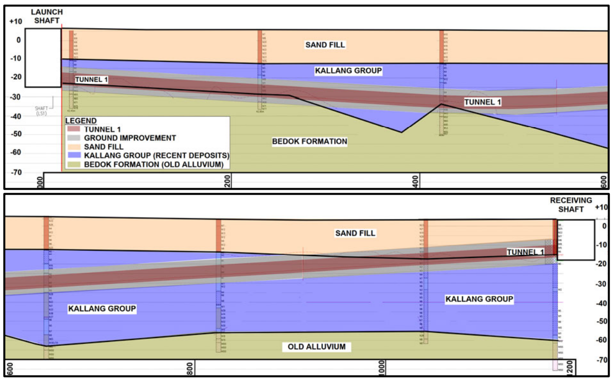

Figure 2. Geological profile along Tunnel 1 alignment.

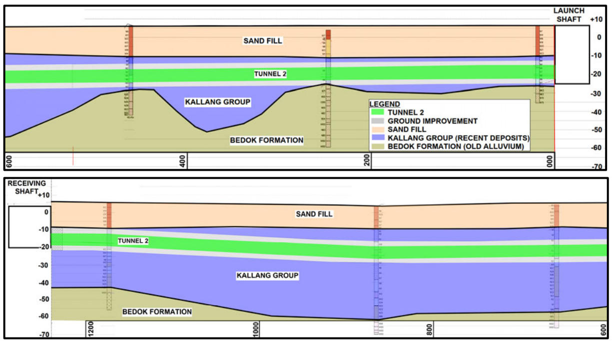

Figure 3. Geological profile along Tunnel 2 alignment.

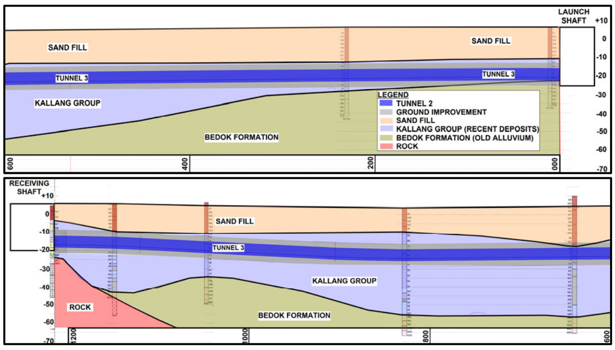

Figure 4. Geological profile along Tunnel 3 alignment.

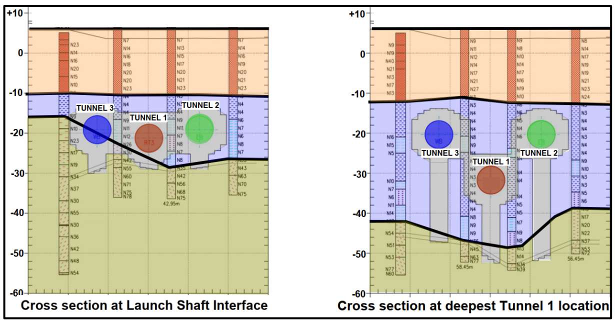

Figure 5. Typical cross section views of tunnels and GI works.

The presence of thick layers of soft soils in the reclamation area posed difficulties for the tunnel drives and induce excessive settlements in long term due to settling ground conditions. As such, extensive ground improvement (GI) works have been carried out to control the long-term settlements of the permanent tunnels. The GI will also enable better control of the surrounding ground settlements induced by the tunnelling works. Cross section views showing the typical GI around and below the tunnels are shown in Figure 5.

The GI works using the deep soil mixing (DSM) method had been carried out to achieve the following design parameters for the above-mentioned purposes:

- Undrained shear strength, Cu: 700 kPa
- Young's modulus, E: 500 MPa
- Coefficient of permeability, k: 1.0E-8 m/s or lower

Verification tests of the design parameters have been carried out extensively. The results of the verification tests showed that the characteristic values obtained are significantly higher than the proposed design parameters of the GI.

4.3 *Adopted face pressure*

Analyses of tunnel stability, groundwater and surrounding ground settlement: Adopting the Alternative Approach, 3D finite element (FE) analyses on the tunnel stability have been carried out to determine the TBM face pressures for the tunnel drives. From the geological profiles, the possible scenarios for the tunnelling works are summarized as follows:

Scenario 1: Tunnel fully covered within GI

Scenario 2: Tunnel partially covered within GI and partially exposed in OA

Scenario 3: Tunnel partially covered within GI and partially exposed to F1 layer.

All the above possible scenarios were expected for Tunnel 1 drive as shown in Figure 6, whereas Scenario 1 was expected for Tunnel 2 and Tunnel 3 drives. As such, the analyses for various possible scenarios were carried out for Tunnel 1. The studies were carried out for the various cases as summarized in Table 1.

Based on the 3D FE analyses carried out for the three possible scenarios, the results are presented in Figure 7 to Figure 9 for Scenarios 1 to 3, respectively, and summarized in Tables 2 and 3. From the results, it is shown that the factors of safety (FOS) for the stability of the tunnel drive in various scenarios are well above the allowable limit, indicating that the tunnels are stable with the applied face pressures for the various scenarios.

The assessment on the groundwater ingress and surrounding ground settlements showed that Scenario 3 (tunnel partially in F1) is not suitable for carrying out the cutterhead intervention (CHI) without further improvement. For the tunnel drives, as the TBMs are operated in closed mode conditions and very short term, the groundwater ingress during the progress of tunnelling will be insignificant.

Adopted Tunnel Face Pressures: The TBM face pressures for Tunnel 1 have been proposed based on the tunnelling scenarios shown in Figure 6. The proposed face pressures are summarized as follows:

Case 1 (Tunnel fully in GI): (a) Minimum FOS of 2 based on stability analyses; (b) Minimum face pressure of 1.5 bar; (c) Target face pressure of minimum face pressure + 0.3 bar; (d) Maximum face pressure of 0.9 overburden pressure.

Case 2 (Tunnel partially in OA) and **Case 3** (Tunnel partially in F1): (a) Minimum FOS of 2 based on stability analyses; (b) Minimum face pressure of 0.75 hydrostatic pressure or 2.5 bar whichever is lower; (c) Minimum face pressures of higher than 0.6 hydrostatic pressure; (d) Target face pressure of minimum face pressure + 0.3 bar; (e) Maximum face pressure of 0.9 overburden pressure.

Figure 10 presents the proposed TBM face pressures for Tunnel 1 drive.

The adopted TBM face pressures for Tunnels 2 and 3 drives followed the same criteria as stated for Tunnel 1 drive.

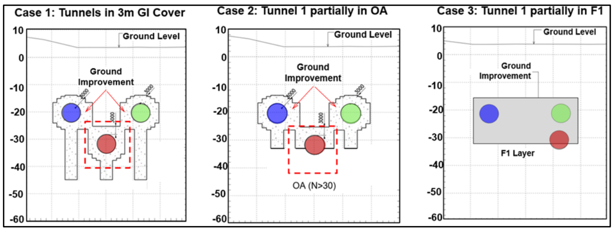

Figure 6. Possible tunnelling scenarios for Tunnel 1.

Table 1. Summary of analysis cases.

Scenario	Case	Applied Face Pressure (bar)	Remark
1: Full GI	1 1a	0	Design k k = 10 x Design k
2: Partial OA	2 2a	0.9	Design k k = 10 x Design k
3: Partial F1	3 3a	1.5	Design k k = 10 x Design k

Note: k denotes coefficient of permeability

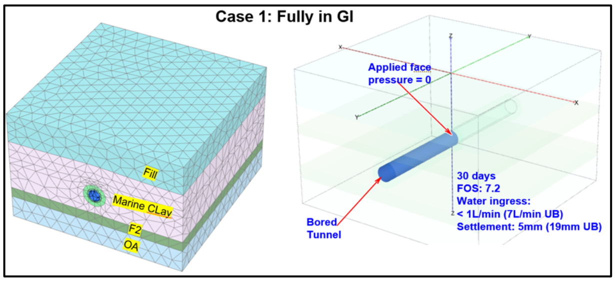

Figure 7. 3D FEA model and results for Case 1.

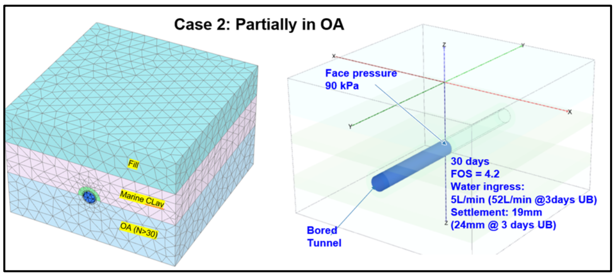

Figure 8. 3D FEA model and results for Case 2.

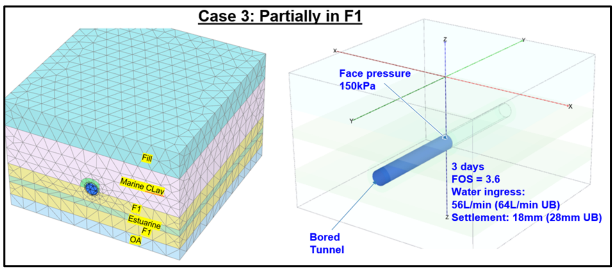

Figure 9. 3D FEA model and results for Case 3.

Table 2. Summary of stability analyses for Tunnel 1.

Case	Time (hours)	Face Pressure (bars)	FOS
Case 1:	720	0	7.20
Full GI (3m)			
Case 2: Partial OA	720	0.9	4.19
Case 3: Partial F1	72	1.5	3.58

Table 3. Summary of water ingress and ground settlements.

Scenario	Case	Applied Pressure	Ground Settlement (mm)	Water Ingress (L/min)
1: Full GI	1	0 bar	5 @ 720 hrs.	0.8
			19 @ 720 hrs.	7.2
2: Partial OA	2	0.9 bar	19 @ 720 hrs.	5.0
	2a		24 @ 72 hrs.	52.3
3: Partial F1	3	1.5 bar	18 @ 72 hrs.	56.4
	3a		28 @ 72 hrs.	64

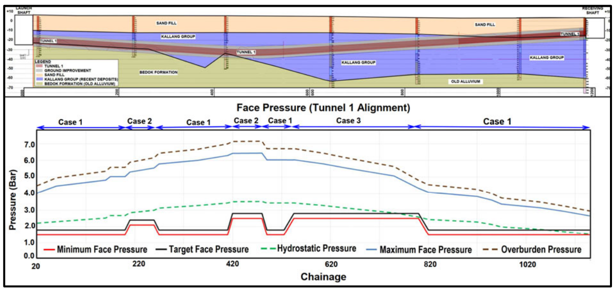

Figure 10. Adopted TBM face pressures for Tunnel 1.

4.4 *Risk mitigation measures*

Proposed Instrumentation and Mitigation Measures: Extensive instrumentation and monitoring have been carried out to monitor the tunnelling performance and the surrounding ground. Figure 11 presents section views of the typical monitoring arrays. Additional piezometers were also installed just outside the influence zone of the tunnelling works at Array F3 to monitor groundwater behavior due to TBM operating at reduced face pressure below hydrostatic pressure.

The proposed mitigation measures for tunnelling works are summarized as follows:

a) Settlement of surrounding ground and structures: When the proposed Alert Level is exceeded, the tunnel face pressure shall be increased to hydrostatic pressure.
b) Piezometers outside the tunnelling influence zones: When lowering of the piezometric heads exceeded the Alert Level, the tunnel face pressure shall be increased to hydrostatic pressure. Recharging wells shall be installed near the existing structures and activated when the proposed action levels are breached.

The CHIs have been proposed to be carried out with atmospheric air pressure for Scenario 1 (tunnel fully covered with GI) and compressed air pressure of 0.9 bar for Scenario 2 (tunnel partially in OA). The CHIs are only allowed with the Cu of GI greater than 700kPa and water ingress of not more than 5L/min. The proposed mitigation measures for CHIs are as follows:

a) When water ingress exceeded 5 L/min: (i) step up compressed air pressure until water ingress is within 5 L/min; or (ii) abandon CHI and TBM progress further to the next location for CHI.
b) When Cu of GI is < 700kPa, abandon CHI and progress to the next location for CHI.

Tunnelling Performance under Reduced Face Pressures: Figures 12 and 13 showed the plan view and instrument layout of Array F3 for monitoring of the tunnelling performance for Scenario 2, where Tunnel 1 is partially embedded in OA. The results of the monitoring are summarized as follows:

- Piezometers within influence zones: -10 to +12 kPa
- Piezometers outside influence zones: +1 to +6 kPa
- Inclinometers: <5mm
- Magnetic extensometers: -5 to +4 mm
- Rod extensometers: -2 to -4 mm
- Ground settlement markers: 0 to -9mm

The above instrumentation results showed that the measured values of the instruments were well below the estimated values, indicating the tunnelling performing well with the TBM operating under reduced face pressures.

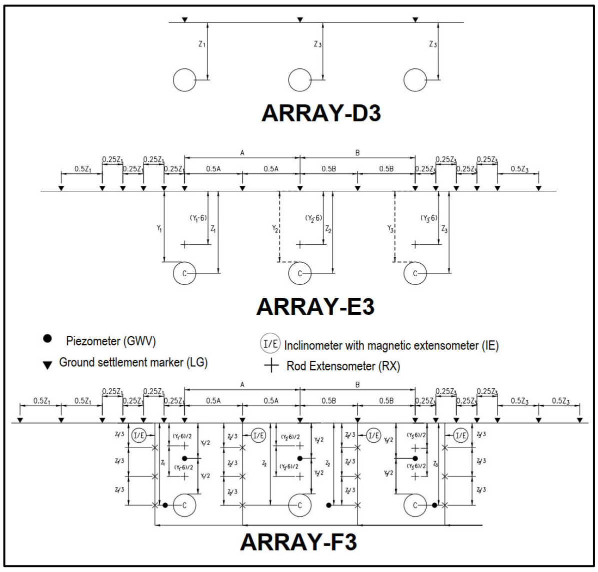

Figure 11. Typical monitoring arrays for tunnelling works.

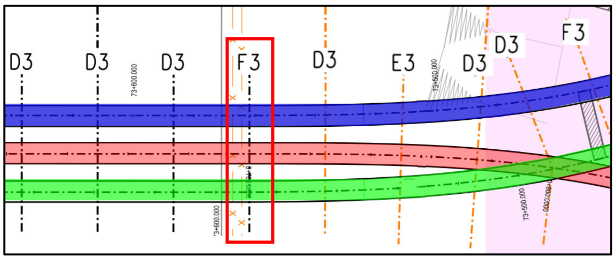

Figure 12. Plan view of monitoring Array F3 for Case 2 of Tunnel 1.

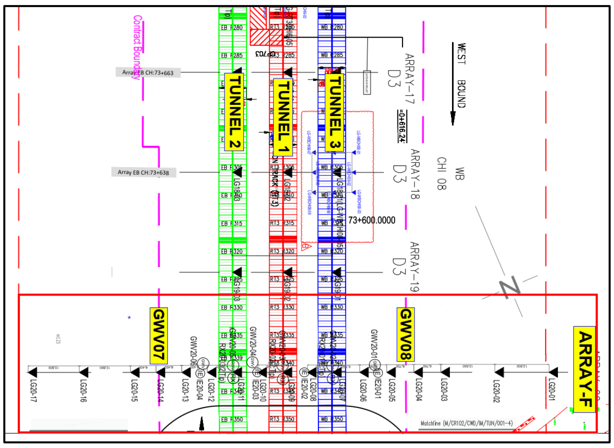

Figure 13. Detailed Instrument Locations for Array F3.

4.5 *Benefit*

The progress of the tunnelling works had been greatly improved with the reduced in the TBM operating face pressures. The time saving was due to increases in tunnelling rate as well as reduction in the number of CHI carried out. For the tunnelling rate, the estimated rates varied from 4 to 5m per day with face pressure equivalent to the hydrostatic pressure, compared to 7.2m to 8.6m per day with TBM operating under the adopted face pressure.

Reduction in the number of CHIs with the adopted face pressure is also the main contributor to the time saving. The CHIs were carried out at typically every 250m of tunnelling advance or typically 5 CHIs for each tunnel drive with the adopted face pressure. The reduction in number of CHIs significantly reduced the time and the cost of the tunnelling works.

The estimated time savings for each tunnel drive are summarized in Tables 4 to 6. The cost savings from the reduced time, manpower and cutter head tool replacements are estimated to be in the order of S$ 0.5 million for each tunnel drive.

Table 4. Estimated time saving for Tunnel 1.

"Deemed-to-Satisfy" Tunnelling Face Pressure (DTS)
• Estimated Tunnelling Rate: **4.4 m/day (263 days)** including main drive conversion and Y-switch installation • Estimated total number of **CHI 24 nos** for 1156m (**72 days**) • **Estimated total time: 335 days**
"Alternative Approach" Tunnelling Face Pressure (ALT)
• Actual Tunnelling Rate: **7.2 m/day (160 days)** • Estimated total number of CHI 5 nos. for 1156m (**15 days**) • Main Drive Conversion: **40 days** • Y-switch installation: **7 days** • Total time: **220 days** Estimated time savings: **115 days**

Table 5. Estimated time saving for Tunnel 2.

"Deemed-to-Satisfy" Tunnelling Face Pressure (DTS)
• Estimated Tunnelling Rate: **5 m/day (245 days)** including main drive conversion and Y-switch installation • Estimated total number of **CHI 25 nos** for 1225m (**75 days**) • **Estimated total time: 320 days**
"Alternative Approach" Tunnelling Face Pressure (ALT)
• Actual Tunnelling Rate: **7.5 m/day (163 days)** • Estimated total number of CHI 6 nos. for 1225m (**18 days**) • Main Drive Conversion: **43 days** • Y-switch installation: **7 days** • Total time: **231 days** Estimated time savings: **89 days**

Table 6. Estimated Time Saving for Tunnel 3

"Deemed-to-Satisfy" Tunnelling Face Pressure (DTS)
• Estimated Tunnelling Rate: **5 m/day (242 days)** including main drive conversion and Y-switch installation • Estimated total number of **CHI 25 nos** for 1211m (**75 days**) • **Estimated total time: 317 days**
"Alternative Approach" Tunnelling Face Pressure (ALT)
• Actual Tunnelling Rate: **8.6 m/day (140 days)** • Estimated total number of CHI 5 nos. for 1211m (**15 days**) • Main Drive Conversion: **33 days** • Y-switch installation: **25 days** • Total time: **213 days** Estimated time savings: **104 days**

5 CONCLUSIONS

In Singapore, in view of complex geological conditions where the geological soil layer often varied along the tunnel alignment, the Deemed-to-Satisfy approach for design of tunnel face pressure adopts full hydrostatic water pressure plus active soil pressure. This paper presented a case study of a successful implementation of an alternative approach which adopts observation method to optimise tunnel face pressure for a MRT tunnel project in Singapore.

In this case study, the ground improvement block surrounding the tunnel alignment with its base extending to hard ground provides a competent, low-permeable ground conditions for the adoption of Alternative Approach. Under this approach, the face pressure below hydrostatic levels can be safely adopted when supported by comprehensive analysis and monitoring. The case study demonstrated that tunnel face pressure lower than the hydrostatic pressure could be successfully implemented in the improved ground areas.

The application of the Observational Method, combined with detailed 3D numerical analysis, provided a robust framework for managing the associated risks. The monitoring results showed that ground movements and piezometric responses remained well within predicted ranges, validating the design approach.

The economic benefits of this approach were significant. The reduction in face pressure resulted in cost savings of approximately S\$0.5 million per tunnel drive, resulting in a total saving of about S\$1.5 million for the three tunnel drives.

With appropriate analysis, monitoring, and risk mitigation measures, tunnel face pressure can be optimised without compromising safety. The methodology presented here could be applicable to future deep tunnelling projects in similar ground conditions, particularly where reduced face pressure could offer significant construction benefits.

It should be noted that the successful implementation of reduced face pressures requires careful consideration of site-specific conditions and should only be undertaken with appropriate expertise and monitoring systems in place.

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