TBM Face Pressure Calculation – A Review of the Industry's Design Approach in Singapore's Thomson East-Coast Line

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ABSTRACT:

Correct design and application of face support pressure is crucial in ensuring the safety and limit the impact of TBM tunnel construction to be controlled within the allowable limit. Continuous improvement in the tunnelling technology has been observed which improves the safety of tunnelling construction in general. Nevertheless, this does not warrant for engineers to be negligent in the design and execution of the tunnel construction. Through learning from the past projects, continuous improvement in the design and construction approaches were made, resulting in the overall benefit to the industry through improved safety, productivity and efficiency in tunnel construction.

In this paper, review of TBM face pressure design approach in ten (10) of Thomson East Coast Line (TEL) tunnelling contracts were carried out. The face pressure design of the ten contracts were reviewed as part of the arrangement where the permanent work designer need to review and submit the face pressure calculation which was carried out by the temporary works designer. As a result of the review of numerous tunnelling contracts carried out by different designers, insights into the current industry's approach to the design of face pressure were made. Findings and lessons learned gained from the review process are discussed in this paper, in order to provide further insights and considerations for future tunnel designs in Singapore, hopefully resulting in further continuous improvement on the safety of the tunnel design in the future.

1 INTRODUCTION

Tunnel construction using pressurized Tunnel Boring Machine (TBM), whether Earth Pressure Balance (EPB) or slurry TBM has become one of the main construction methodologies for tunnels in Singapore. Since the early days of tunneling in Singapore where different tunnelling methods are employed using Greathead shield, drum digger and NATM (Shirlaw and Doran, 1988), continuous improvement in the tunneling technology has been seen, which results in better ground control during the tunnel construction.

In the construction of North East Line, most of the construction was carried out using EPB TBM, and settlements were generally well controlled (Shirlaw et.al., 2003). Nevertheless, some large settlements and sinkholes were recorded, and lessons were learned with recommendations given for further improvement in future tunneling projects.

By the time of Singapore's 5th MRT line Downtown Line, it was shown that the ground movement caused by tunnelling can be controlled very well, causing no significant settlement on the building, even in close proximity to the foundations provided proper tunnelling controls are applied (Goh K.H., et.al., 2016).

Unfortunately, accidents during tunneling does happen (BCA, 2023), showing the importance of appropriate planning, selection of tunneling method, as well as design and control of the face pressure during tunnelling. For the success of tunnelling, it is very important that the required face pressure is calculated appropriately and sufficient measures implemented to control the pressure during construction.

Often times, the engineer designing the TBM face pressure has to balance the risk of ground settlement (due to insufficient face pressure), as well as heave/blow-out and excessive cutter tool wear (due to too high pressure). The engineer will also have to balance the risks of tunnelling accordingly depending on various factors such as the expected ground condition, surrounding structures, depth of tunnel, etc. These considerations are largely learned from experiences in the past projects, by making sure mistakes are not repeated, and good initiatives employed where required.

This paper describes the review of design approaches that have been adopted in some of the more recent tunneling projects in Singapore on the Thomson East-Coast Line and provides suggestion on the recommended approach in designing the TBM face pressure, as well as any considerations for some of the conditions that are commonly found in Singapore.

1.1 Background

Figure 1 - Thomson East-Coast Line tunnels reviewed in this paper.

As part of the BCA requirement as the A/E designer for Thomson East Coast Line (TEL) contract C2102, C2105 and E1002, the permanent works designer was required to review and submit the face pressure calculation which was carried out by the temporary works designer. This unique arrangement results in the review of 10 tunneling contracts in the TEL (see Figure 1), with over 20km total length of tunnel, spread across all the major geological conditions in Singapore carried out by 7 different engineering consultancy firms. The TEL contracts which were reviewed are summarized in Table 1.

Table 1 - Summary of TEL contracts reviewed in this paper.

Contract	**Geological Condition at Tunnel Face**	**Approximate Length of Tunnel (km)**	**TBM Type**
T202	Fill/KF/BTG	2.0	Slurry
T206	Fill/BTG	6.3	Slurry/EPB

Contract	Geological Condition at Tunnel Face	Approximate Length of Tunnel (km)	TBM Type
T222	JF	2.3	EPB
T225	JF/KF/FCBB/OA	2.2	EPB
T227	KF/GI	1.5	EPB
T228	KF/GI	1.3	EPB
T307	KF	1.0	EPB
T308	KF	2.7	EPB
T310	KF/OA	3.5	EPB
T311	OA	0.5	EPB
Note for geological conditions: KF – Kallang Formation BTG – Bukit Timah Granite JF – Jurong Formation FCBB – Fort Canning Boulder Bed OA – Old Alluvium GI – Ground Improvement			

The arrangement provides the rare opportunity to compare the different approaches adopted in the industry when designing TBM face pressure across the different conditions that are found in Singapore. Despite the many tunnelling projects already done in Singapore, there isn't a specific guide for designing the TBM face pressure in Singapore. Through this review, it is envisaged that future engineers will be able to make better judgments by reviewing some of the considerations studied in this paper. Ultimately, it is hoped that this review will pave the way for the creation of specific design guide for tunnelling in Singapore.

1.2 Scope of Discussion

In order to limit the scope of discussion in this paper, the paper will focus solely on the design aspect of the TBM face support pressure. Other considerations to maintain the control of ground settlement such as to name a few, control during TBM stoppage, soil conditioning, control of face support pressure, muck reconciliation, design of TBM cutter and support systems, etc. are not discussed here to limit the scope of discussion as these topics deserve in depth discussions on their own.

It shall be noted that success in tunnelling does not solely depends on the determination of the face pressure. Other factors such as selection of correct tunneling method, workmanship and TBM design to name a few are critical if not more critical than the design aspect. Design of the face support pressure is one part of the whole that ensures success in tunnelling work, where all party needs to work together to ensure the success of tunnelling work.

It is also worth to highlight that to come up with a good design, the designer should also be familiar and experienced with the other considerations in controlling the ground movement. As can be seen in the subsequent sections, while the design guides provide a good reference for the design, design judgments will need to be made to suit the site specific conditions.

2 REVIEW OF EXISTING DESIGN GUIDES

While there isn't a specific design guide in Singapore for the design of TBM support pressure, there are two international design guides which were widely adopted internationally. Brief overviews of the two design guides are presented below.

2.1 Hongkong GEO Guide No. 249 (2009) and 298 (2014)

Hongkong Geotechnical Engineering Office (GEO) released two numbers of design guide, namely guide no. 249 (2009) for slurry TBM tunnelling and guide no. 298 (2014) for EPB TBM tunnelling.

While there are slight differences in the application of the pressure for slurry and EPB TBM, the required support pressure are calculated in a very similar manner in both design guides, and therefore the calculation is discussed together in this section. For simplicity, the guides are referred to as GEO Guide in the subsequent sections.

The design guide provides design methodology for required minimum face support pressure in both effective and total stress condition depending on the ground condition. In addition, both ULS and SLS condition are checked for failure and ground settlement respectively. Generally, ULS condition reflects the failure or instability of the tunnel face, while SLS condition reflects the amount of pressure required to maintain the ground settlement to be within the prescribed limit.

2.1.1 Effective Stress (Drained) Condition

For the effective stress calculation, the required face support pressure is calculated as follows:

$$P_F = P_E + P_W$$

Where P_F is the required face support pressure, P_E is the required effective pressure due to soil and P_W is the required pressure to stabilize the water pressure.

For the ULS condition, the required effective pressure due to soil can be calculated based on truncated formula from Anagnostou and Kovari (1996) shown below:

$$P_E = F_0\gamma' D - F_1 c'$$

Where F_0 and F_1 are non-dimensional factors described in Anagnostou and Kovari (1996), γ' is the effective unit weight and c' is the effective cohesion. It shall be noted that there are two additional terms in the original paper by Anagnostou and Kovari (1996) relating to groundwater drawdown which is ignored in the design guide. It shall be noted that based on this calculation, as there is no groundwater drawdown is considered, the resulting face pressure in this condition will always be higher than the hydrostatic pressure as the P_W component is simply hydrostatic.

For the SLS condition, the required effective pressure due to soil can be calculated based on truncated formula from Proctor and White (1977) shown below:

$$P_E = F\gamma' D$$

Where F ranges between 0.2 to 0.55 depending on the SPT-N of the ground. As can be seen in the equation, there is no correlation for the ground settlement or volume loss in the calculation by Proctor and White, and the equation was developed to derive the pressure exerted by the soil if limited movement is allowed.

2.1.2 Total Stress (Undrained) Condition

In the undrained condition, the minimum face pressure required to stabilize the face is calculated as follows.

$$P_F = (\gamma \cdot z_0 + q) - N \cdot c_u$$

Where, P_F is the required face support pressure, γ is the total unit weight of the soil, z_0 is the depth of the soil layer, q is the surcharge loading applied at the top of the soil layer, N is non-dimensional stability number and c_u is the undrained shear strength of the soil layer.

In the ULS case, the value of N is simply taken as N_{TC}, where N_{TC} is the stability number at collapse and suggested to be obtained from the chart in Kimura & Mair (1981), which depends on the ratio of C/D and P/D. C represents the depth of the soil layer above the tunnel crown, D represents the excavated diameter of the tunnel and P represents the unsupported length of the tunnel. For ULS calculation, the guide suggests that the value of P can be taken as 0, assuming that the ground is allowed to move and converge to the TBM shield in the ULS case.

In the SLS case, the value of N is taken as

$$N = LF \cdot N_{TC}$$

Where LF is a load factor that is dependent on the targeted volume loss, and is suggested to be obtained from a chart in Kimura and Mair (1981) or equation in Dimmock and Mair (2007). Furthermore, in determining the NTC, the value of P is suggested to be taken between 0 to L, depending on whether there is slurry injection around the shield. For tunnelling in urban condition where strict volume loss control is required, this calculation typically governs the undrained calculation.

2.2 German DAUB Recommendation (2016)

In the publication by DAUB "Recommendations for Face Support Pressure Calculations for Shield Tunnelling in Soft Ground" (2016), three general methods of calculating the required face support pressure is recommended. The calculation methods are briefly summarized below, and the readers are highly suggested to refer to the publication for complete discussion of each calculation method. For simplicity, the guides are referred to as DAUB Recommendation in the subsequent sections.

It should also be noted that due to the nature of the calculation, the recommendation suggests the first two calculation method (limit equilibrium and stability ratio) to be mostly utilized for ULS calculation, and SLS condition is recommended to be checked using numerical analysis.

2.2.1 Limit Equilibrium Method

In this calculation method, the stability of the face is determined based on the stability of an assumed failure mechanism, for example a wedge failure block shown in Figure 2. In the calculation, the required pressure is determined based on the required pressure to balance the driving forces of the overburden above the wedge block and self-weight of the wedge. Typically, the critical support pressure is then obtained by varying the angle of the wedge to obtain the largest support pressure required. In the recommendation, it is highlighted that this calculation should be calculated using the drained soil parameter, and undrained parameters are not recommended for this calculation. The calculation however, is suitable for mix of alternating cohesive and non-cohesive soil in front or above the tunnel face as the varying soil can be modelled in the failure wedge.

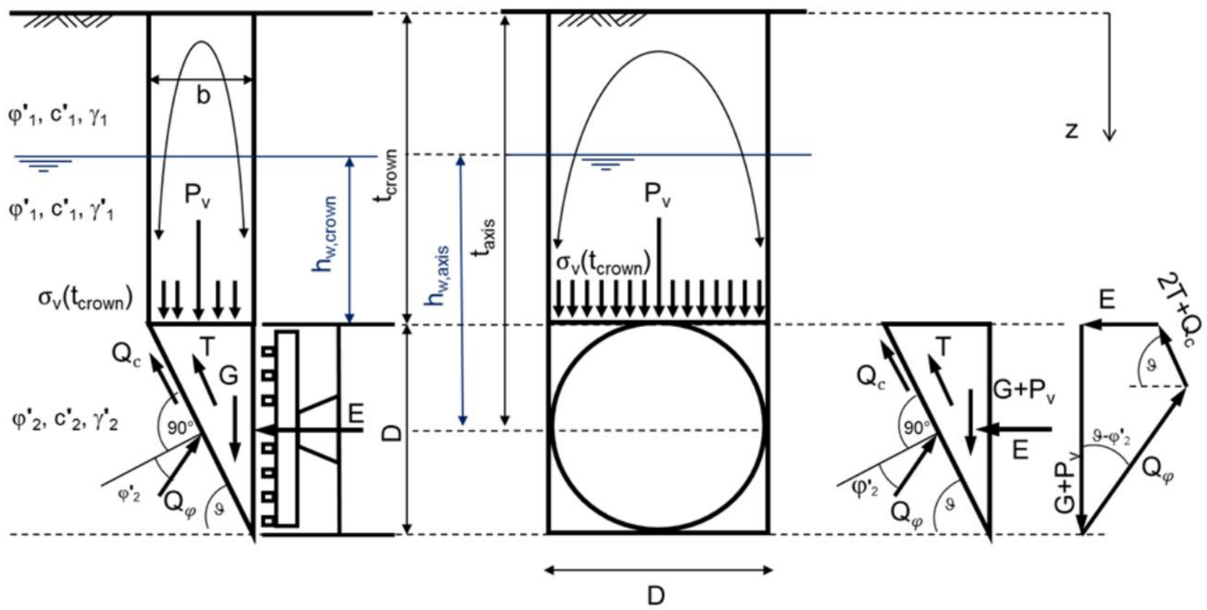

Figure 2 - Sketch of an example of assumed wedge failure mechanism from DAUB (2016)

Multiple approaches to calculate the stability of the wedge are discussed in the recommendation based on different literatures, including on variation to calculate the weight of the overburden above the wedge, stabilizing lateral earth pressure coefficient, soil friction on the soil wedge, to name a few. Although the recommendation gives a recommended approach for each item, the design engineer is expected to understand the implications of each approach in their design to suit their project specific conditions.

2.2.2 Stability Ratio Method

The stability ratio method described in the DAUB recommendation is essentially the same family of method described in Hongkong GEO calculation for the undrained condition discussed in section 2.1.2. The main difference between the two methods is in the determination of the stability number N. In DAUB recommendation, four different literatures are presented, and generally acceptable range of stability number based on Leca & New (2007) is given. These range of values are generally used as a sense check for the designer to ensure that the design is sufficiently safe.

2.2.3 Numerical Method

For tunnels close to sensitive structures, the DAUB recommendation suggests that numerical analysis to be carried out. While it is noted that 3D step-by-step analysis would be best to simulate the 3D stress profile of the TBM excavation, it is also mentioned that this method is very time consuming and therefore usually simpler 2D analysis approach is adopted.

However, as the 2D approach does not capture the 3 dimensional stress of the tunnel excavation, it is unclear whether this simplified approach will be more or less conservative compared to the 3D model. As a middle ground, a simplified “Pseudo 3D” approach is described, where the steps of TBM advance is not modelled. This “Pseudo 3D” analysis is mentioned to be beneficial for short TBM drives under critical areas.

3 REVIEW OF FACE PRESSURE APPROACHES IN TEL CONTRACTS

Based on the review of the design approaches in the 10 tunnelling contracts shown in Table 1, it is seen that all of the contracts based their analysis largely based on the methodology described in Hongkong GEO guides. Nevertheless, some modifications from the guide were observed in all contracts to suit the local ground conditions, constraints and site-specific conditions.

In this section, the approaches in the different contracts are categorized based on the different geological conditions. Considerations for other conditions are discussed in section 4.

3.1 Face Pressure in Rock

This section summarizes the review on approaches of face pressure design when the tunnel is situated fully within either BTG of grade G(III) or better, JF grade S(III) or better and FCBB. Face pressure design in contracts T202, T206, T222 and T225 where rock is encountered are reviewed and summarized below.

Technically, for tunneling in intact rock, no face pressure will be required. The intact rock face is generally stable on its own, as generally is shown with undrained calculation (see section 2.1.2), with c_u taken generally as half of Unconfined Compressive Strength (UCS) of the rock mass. Intact rocks generally have very low permeability and therefore drained analysis are not carried out. Nevertheless, typically minimum face pressure is applied to ensure the chamber is fully filled. This is carried out mainly to ensure the pressure in the chamber can be adjusted quickly in case unexpected condition such as e.g., faults/large joint is encountered. In slurry TBM, this is also to prevent damage to suction pump due to fluctuation in slurry level.

When the rock is highly fractured, generally the permeability of the rock mass becomes much higher, and the effect of seepage into the tunnel will need to be considered. While the rock mass itself is generally very stiff and won’t deform significantly, the movement of the soft soil layer above the rock mass due to the lowering of pore water pressure need to be assessed, especially if there’s any critical structure in the area. In the reviewed calculations, face pressure is mainly set to be at least hydrostatic pressure where there is a risk of groundwater seepage into the tunnel.

Figure 3 - Photo of free air intervention in intact BTG rock, showing stable and dry condition at the face.

3.2 Face Pressure in Competent Soil

This section summarizes the review on approaches of face pressure design when the tunnel is situated fully within residual soils of BTG, JF or OA(C) or better with SPT-N generally higher than 30. It should be noted that the previous definition shouldn't be taken as a hard and fast rule, and definition may differ depending on the tunnel depth and other conditions such as the ground cementation and permeability to name a few. Generally competent ground is defined as ground condition where the ground itself provides significant support to the stability of the face, as compared to soft soil where the strength of the ground itself is very small or at times negligible. Face pressure design in contracts T202, T206, T222, T225, T310 and T311 where competent ground is encountered are reviewed and summarized below.

Generally, the calculations in these ground conditions are governed by the drained condition, as the high undrained shear strength of the ground renders only minimal or at times no face pressure is required in the undrained condition. Often times, the design in these ground conditions revolves on strategy of applying face pressure on whether the drained condition is encountered.

As outlined in DAUB Recommendation (2016) based on Anagnostou & Kovari (1994), undrained behavior can be expected for TBM advance of 1.7-16.7mm/min for soil permeability of lower than 10^{-7} - 10^{-6} m/s. For some competent soil where the soil permeability can be shown to be consistently lower than 10^{-7} m/s (for this review e.g., the OA found at the eastern Singapore), the soil can be safely assumed to behave in undrained condition. In such cases, drained condition calculation is typically carried out as contingency measure to allow the TBM face pressure to be raised in case any sandy/permeable layer is encountered unexpectedly.

On the other hand, when the ground is known to be permeable or sandy, drained condition calculation must be carried out and considered accordingly. While most contracts apply the approach from GEO Guide for the ULS calculation, in some contracts some alternative approach are adopted.

One of the variation in the approach is the adoption of wedge failure check similar to limit equilibrium method described in section 2.2.1. The approach is deemed appropriate, as the competent soil in the contract is overlain by thick, soft soil layer, and the approach can adequately account for the different strength of the different soil layers.

As can be seen in 2.1.1 and 2.2.1, it can also be seen that the analytical calculation methods for drained condition do not typically correlate to the ground movement adequately. In addition, the drained SLS calculation in GEO Guide by Proctor and White is largely correlated to Hongkong's ground condition. As such, variations in the drained SLS calculation are also observed, with some contracts applying an additional check on the drained SLS condition, based on load factor method. The method is done by comparing the mobilized shear strength compared to unfactored ULS method calculated based on Anagnostou & Kovari (1996). The fraction of mobilized shear strength is limited to a value based on past literature or past project experience in similar ground.

For some of the contracts where the tunnel is deep, the face stability is also calculated taking into account of seepage due to the high pressure required to balance the hydrostatic. In such cases, full equation by Anagnostou & Kovari (1996) is used instead of the truncated equation in the GEO Guide. The truncated part of the equation details the pressure required to stabilize the face due to the seepage.

Nevertheless, when the tunnel is in close proximity to any sensitive structure and/or there's indication of soft compressible soil layer near the tunnel, the impact of pore pressure reduction to the ground settlement needs to be adequately considered. As highlighted in DAUB recommendation, the only reliable method of quantifying ground settlement in drained condition is to carry out complex 3D analysis, which is very time consuming. As such, for all the contracts reviewed, face pressure is maintained above hydrostatic when the tunnels are in close proximity/cross under buildings or structures. Full discussion on considerations when tunnelling near to buildings/structures are described in section 4.1.

Figure 4 - Photo of free air CHI in competent ground (OA).

3.3 Face Pressure in Soft Soil

This section summarizes the review on approaches of face pressure design when the tunnel is situated fully within KF or OA(D) or worse with SPT-N generally lower than 30. It should be noted that the previous definition shouldn't be taken as a hard and fast rule, and definition may differ depending on the tunnel depth and other conditions such as the ground cementation and permeability to name a few.

Generally soft soil is defined as ground condition where the ground itself provides little to no support. Significant face pressure is generally required to maintain the face stability, and because generally the soil is compressible, even higher pressure is typically required to maintain the ground surface settlement. Face pressure design in contracts T225, T227, T228, T307 and T308 where soft soil is encountered are reviewed and summarized below.

When the TBM is tunnelling in soft soil, generally face pressure calculation is governed by the undrained condition, with undrained SLS calculation often being the governing calculation to meet the stringent volume loss control in Singapore's urban condition. Furthermore, due to the compressible nature of the soil, applying face pressure below the hydrostatic pressure is not advisable, unless very detailed analysis and risk assessment has been carried out. Especially in soft clayey material, the effect of consolidation settlement will need to be adequately accounted for. In all the contracts reviewed, all contracts have applied pressure higher than hydrostatic when tunnelling in soft soil.

Especially when the TBM is driven under Kallang Formation's Marine Clay, generally the face pressure needs to be applied very close to the overburden pressure when calculated using the undrained SLS formula discussed in section 2.1.2 due to the very low undrained shear strength of the soil. This is in line with site observation from Shirlaw (2003) in the construction of North East Line shown in Figure 5, where it can be seen that for volume loss to be reliably controlled below 3%, face pressure needs to be more than 80% of the overburden pressure. In some of the contracts, this correlation between the normalized face pressure and the volume loss by Shirlaw (2003) is applied as additional check to determine the face pressure required to obtain the required volume loss. As the resulting pressure from the empirical approach generally agrees well with the required pressure from the undrained SLS formula, this approach generally is useful as a sense check if the calculated pressure is reasonable.

With such high pressure nearing the overburden, often times the pressure fluctuation during the TBM operation (generally EPB is used in soft soil condition) need to be considered carefully as the applied face pressure may end up being higher than the overburden and risks causing heaving/blow-out instead. Full considerations for prevention of blow-out/heave is discussed in section 4.2.

In addition, while most of the contracts obtained the N_{TC} value based on the chart by Kimura and Mair (1981) as recommended by the GEO Guide, some variation in the approach were observed where N_{TC} value limit of 6 is used, based on Broms and Bennermark (1967), which is one of the listed reference in DAUB recommendations. Generally the different methods of deriving N_{TC} is observed to not affect the resulting face pressure value significantly in soft ground condition, due to the range of N_{TC} generally being similar, and the low value of undrained shear strength.

As drained condition generally does not govern the calculation, it is technically possible to apply lower face pressure if the face is situated fully in sandy highly permeable material which is not going to behave in undrained condition. Nevertheless, this condition is rarely found as the most commonly found Fluvial Sand layer in Kallang Formation, is typically interbedded with the clay layer and full face sand is very rarely encountered. Out of all the contracts reviewed, no such condition is found, and therefore not discussed further.

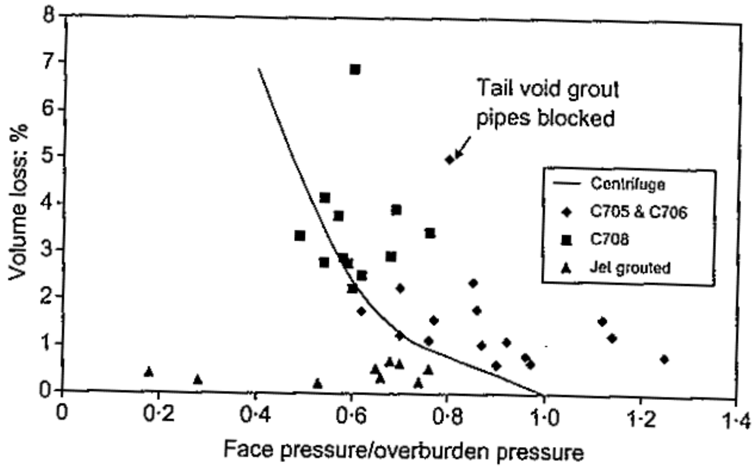

Figure 5 - Observed volume loss for tunnelling in Kallang Formation plotted against normallised face pressure, by Shirlaw (2003).


Figure 6 - Sample of Marine Clay of Kallang Formation prepared for triaxial test, showing main representative of soft soil found in Singapore.

3.4 Face Pressure in Mixed Face Condition

While generally there is a wide definition of mixed face condition in tunnelling, for the purpose of the TBM face pressure design, definition from the GEO Guide is adopted, where the tunnel face is considered to be mixed face if there are 2 or more geological units (soils or rocks) at the tunnel face that have very different nature. By this definition, the mixed face condition reviewed in the TEL contracts are, interface between rock and residual soil of BTG or Jurong Formation, interface between OA and KF, interface between KF and any rock, and presence of thick fluvial sand in Marine Clay of KF. The author views this definition is more applicable in Singapore context compared to DAUB recommendations which only considers the interface of hard rock and soft soil as mixed face.

Based on the GEO Guide, two (2) different cases are described. In the first case, the tunnel is driven with significantly weaker layer above the tunnel. In this case, it is recommended that the significantly weaker layer to be treated as load, and cover over tunnel (C) is taken as the depth of the stiffer layer. In the second case, where there are multiple geological units at or close to the face of the tunnel, face pressure is applied to stabilize the weakest unit. It is worth noting that the second approach is also similarly suggested in DAUB recommendations. Based on the review of TEL contracts, the second approach is typically adopted, although in some cases the first approach is adopted based on wedge stability analysis similar to limit equilibrium method described in 2.2.1.

As shown in BCA (2023), mixed face condition is one of the most critical stretch in TBM tunnelling where accidents mostly happen, especially at the interface of BTG rock and soil or due to presence of boulder at the tunnel face. While generally the face pressure is designed to maintain the face stability of the weaker material, accidents generally happen due to difficulty in maintaining the pressure and difficulty advancing the TBM due to the difference in stiffness causing overexcavation. Generally control of the TBM parameter is key in this instance, with reduction of cutter rotation speed, advance rate and correct soil conditioning/slurry properties being some of the key aspects to pay attention to in these cases. Detail of the construction considerations is outside the scope of this paper and can be referred to other publications.

Another key interface worth highlighting in the TEL contracts are the presence of thick loose fluvial sand (F1) layer in Marine Clay of KF. From design perspective, the application of pressure from undrained calculation as described in section 3.3 is generally sufficient to maintain the stability of the tunnel face. Nevertheless, due to the significantly higher permeability of F1 layer compared to Marine Clay layer renders risk of pressure loss when suddenly encountering the mixed face to deserve special attention. Based on the reviewed TEL contracts, special attention was given during design review to ensure both contractor and contractor's designer to understand the criticality of these interfaces. Trial of soil conditioning application (EPB TBM is used in all contract with Kallang Formation) to ensure that face pressure can be controlled even when encountering large deposit of loose sand layer. Based on the reviewed contracts, application of clay shock and/or polymer additive is found to produce good results.

4 OTHER CONSIDERATIONS

Apart from the geology, other factors also affect the design of the TBM face pressure, which is further discussed in the sections below.

4.1 Proximity to Buildings/Structures

When the tunnels are in close proximity to buildings/structures, control of ground movement becomes much more critical as excessive ground movement can cause damage to the buildings/structures. As highlighted in DAUB Recommendation, the ground movement assessment in drained condition is difficult to be quantified using analytical models, especially if seepage effect is to be considered. As discussed in the recommendation, the only real way to quantify the ground movement is to run a complex 3D numerical analysis which is resource intensive.

Due to this complexity, in all the contracts reviewed, face pressure is applied higher than hydrostatic when the tunnel is in close proximity to the buildings, which also ensures impact of long term settlement due to pore pressure reduction is mitigated sufficiently. Especially when the tunnel is driven under live railway tunnel in parallel arrangement for more than 300m in contract T222 (Velu, 2019), design of TBM face pressure, additional control measures such as recharge wells were employed to ensure controlled movement of sensitive structure even during the TBM stoppage or unforeseen condition is encountered.

Nevertheless, it shall be noted that applying face pressure above the hydrostatic might become increasingly costly and unfeasible as the tunnel depth increases, and balance of risks may need to be considered, which is discussed in section 4.4.

4.2 Prevention of Blow-out/Heave

While face pressure is required mainly to control ground surface settlement, application of high face pressure may inversely cause the ground to heave instead, or the tunnelling medium (muck or slurry) might flow to the surface, causing what is known as blow-out.

Generally, most of the reviewed TEL contracts limit the face pressure to below the ground overburden, which is generally sufficient to prevent heaving or blow-out in general conditions. Nevertheless, in cases where there is higher risk of blow-out such as tunnelling near boreholes or fractured rock, where there is potential pathway(s) available for the tunnelling medium to travel to the surface, further limiting the face pressure to the unit weight of tunnelling medium multiplied by the depth of tunnel might be appropriate. This ensures that it is physically impossible for the tunnelling medium to flow above the ground surface, avoiding disruption on the ground surface.

In addition, due to the complex mechanism of seepage of tunnelling medium and limitation of ground information during design, surface watchmen deployment above the TBM is found to be useful to spot any signs of blow-out as soon as possible. As observed in one of the contract, small amount of foam was spotted by the surface watchman, which prompts the face pressure in the TBM to be reviewed and adjusted accordingly, preventing any further issue during the TBM advance.

In another case, when the tunnel is driven in very soft soil with low strength, often times the required pressure to maintain the ground movement to be within allowable limit is close to the overburden pressure. Further considering the pressure fluctuations that occur during the TBM drive, the range of operating pressure may be very close to the ground overburden. In such cases, the designer may either reduce the allowable pressure variation where possible (subject to constructability review), or consider the contribution of ground shear strength in the calculation of heaving, assuming failure of the soil in the reverse direction of settlement. However, generally this approach is to be adopted with caution, with additional safety factors typically applied.

4.3 Tunnelling under Water Bodies

Among the reviewed TEL contracts, only a single contract (T228) is found to encounter this particular condition, and therefore this review is limited in scope to the particular condition. In this contract, the tunnels cross under the Marina Channel under soft ground condition of Marine Clay. Due to the soft clay, the required pressure calculated is found to be close to the overburden pressure. Considering the lack of sensitive structure above the tunnel when under water, it was agreed that prevention of blow-out which may pollute the water body is much more critical than ground movement control. As such, the maximum pressure is ensured to be lower than the ground overburden to ensure the risk of blow-out is minimized as much as possible.

4.4 Deep Tunnels

For deep tunnels, generally there will be less impact to the ground surface settlement, as can be seen in the formula in the maximum ground settlement described in LTA (2019),

$$S_{max} = \frac{0.0031\, V\, D^2}{K\, z_0}$$

Where V is the volume loss expressed as a percentage, D is the excavated diameter of the tunnel, K is the non-dimensional trough width parameter and z_0 is the depth to the centre of the tunnel. As the maximum surface settlement is inversely correlated to the depth of the tunnel, deeper tunnels generally generate flatter ground movement for the same volume loss, resulting in generally less impact to the surrounding structures.

Furthermore, application of high face pressure results in high cutter tool wear, resulting in more frequent stoppage required. Compressed air stoppage under high pressure if hydrostatic pressure need to be balanced in deep tunnel is generally considered as a high risk activity due to the risk of pressure loss to the ground which may result in injury to workers. Furthermore, with the drive to raise the productivity of the construction, there is plenty of driver to optimize the application of face pressure in deep tunnels.

Within the reviewed TEL contracts, several relatively deeper tunnel stretches of about 40m deep were observed to adopt face pressure lower than hydrostatic pressure when the tunnels are in greenfield conditions to optimize the applied pressure. In order to ensure the ground movement is well controlled, the ground movement is monitored, and pressure is prescribed to be raised accordingly in case any adverse reading is observed.

5 CONCLUSION

Based on the review of design approaches, it can be concluded that while the available design guides provide good guidance for the design of TBM face pressure, certain modifications and judgments need to be applied in the design to suit the design of the TBM face pressure. A good understanding of typical constraints encountered along the tunnel drive and construction considerations is required to ensure the designed face pressure is safe, efficient and most importantly constructible.

While it is observed that none of the contracts have applied 3D numerical modelling in their analysis of TBM face pressure, as described in DAUB Recommendation, this method although tedious, remains the only real way to predict the ground movement reliably in the drained condition, especially if seepage into the tunnel is present when the face pressure is applied below the hydrostatic pressure. As the future tunnels need to go deeper due to increasing constraint underground, it is expected that a more exact method of analysis will be required, to further optimize the applied pressure which will increase the overall safety and efficiency of the tunnel construction.

By publishing the findings in the review of a number of design approaches in different ground conditions of Singapore, it is hoped that the lessons learned in these projects will provide better insight for future designers, resulting in increased safety of tunnel constructions in Singapore as a whole. In addition, the author also hoped that this might provide a good building block for future Singapore specific design guide, which will provide even more comprehensive guidance to the future tunnel designers in Singapore.

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