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Stability analysis when tunnelling in soft soil condition in Hochiminh city



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ABSTRACT

The rapid development in the economy and the urban population in cities in Vietnam in recent years leads to dramatic growth in traffic congestion, noise and air pollution, and environmental degradation. Urban underground infrastructure thus becomes an inevitable solution since the surface space becomes too expensive and restricted in spite of the high cost of tunnel construction. However, tunnelling in deltaic urban areas, in particular in the cases of Mekong Delta and Red River Delta areas, often faces to difficulties of soft soil conditions and the existence of important historical buildings on the surface. The first metro line constructed in Vietnam from Ben Thanh to Suoi Tien, which is located under crowded areas of Hochiminh city, was tunnelled in such conditions. This paper presents a back analysis for the stability of the tunnelling process in the project. The minimum support pressures is estimated by using the wedge model proposed by Jancsecz, S., & Steiner, W. (1994). Meanwhile, the maximum support pressure is estimated by the blow-out model proposed by Vu et al. (2016). Based on these results, the range of support pressures recommended for the Earth Pressure Balance Tunnel Boring Machine in the Hochiminh Metro Line No.1 project soft soil conditions is derived and compared to in situ support pressures obtained from the monitoring data in this tunnelling project. The study result shows a good agreement between the support pressures obtained from stability analysis models and the monitoring data from the construction site. Based on this back analysis, some recommendations are proposed with the purpose of successful construction in the next tunnelling projects in Hochiminh city.

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1. Introduction

The economic development and the urban population in cities in Vietnam have grown rapidly in recent years. This issue leads to rapid growth in traffic congestion, noise and air pollution, environmental degradation, etc. Therefore, infrastructure development is an essential demand today. Although the cost of tunnel construction is high in comparison with the construction cost of surface buildings, urban underground infrastructure is an inevitable development since the surface space becomes too expensive and restricted.

There are eight metro lines that will be built both in Hanoi and Hochiminh city. The Hochiminh Metroline No.1 construction was completed in 2020. It is considered as a pilot metro line in Hochiminh city from the Ba Son station to Suoi Tien park with difficult soft soil conditions. With a length of 2.6km underground construction, the monitoring data obtained in the tunnelling process is valuable data and information for next tunnel projects with better design and management.

One of the most important tasks of tunnelling design with TBMs in cities is to maintain the stability of nearby buildings. When tunnelling in soft soil conditions in urban areas and especially under historical areas, there might be a risk of damage to buildings from the collapse of the tunnelling face and the subsequent surface settlement. In the case of Hochiminh Metroline No.1, the tunnel alignment is designed under historical buildings and density areas with a soft soil condition of silty and clay soil layers. Therefore, controlling the support pressures at the tunnelling face, around the TBM and at the tail is extremely essential to avoid unexpected displacements in the surrounding ground and damages to existing buildings.

In the case of tunnelling in soft soil conditions, when the support pressures at the tunnelling face are too small, the tunnelling face might collapse and the soil moves towards the TBM, thus uncontrolled settlement might appear on the surface. The minimum support pressure derived by this condition was presented in Anagnostou and Kovári (1994), Jancsecz and Steiner (1994), Broere(2001) and Vu et al. (2015).

In tunnelling, the soil volume ahead of the tunnelling face and/or above the TBM machine tail might be pushed upward in the case of too high support pressure pumped. As the result, the support fluid flows out the soil medium and the support pressure applied at the locations of the tunnelling face and/or the TBM tail dramatically reduces. In this case, the tunnelling face might collapse and the tunnelling process can be stopped. This situation is the blow-out in tunnelling (Vu et al., 2015). Some consequences of the blow-out situation are the standstill of the TBM machine, risks for people when maintaining, the surface of existing building collapse, etc. Especially, when shallow tunnelling, blow-out should be carefully taken into account. In fact, there are two cases of a blow-out in the tunnelling process including the Old Elbe Tunnel in the year 1909 and the Second Heinenoord Tunnel in the year 1997. This situation determines the maximum allowable support pressure applied in the tunnelling process. In tunnelling design, some below blow-out models proposed by Balthaus (1991), Broere (2001) and Vu et al. (2015) can be considered.

This paper presents a back analysis of the support pressures applying on the tunnelling face in the tunnel alignment in the Hochiminh Metroline No.1 project. The minimum supporting pressure is determined based on the wedge models and the maximum supporting pressures are derived from recent blow-out models. From this, the range of supporting pressures applied at the tunnelling face of the TBM is derived. The analysis results are compared with the actual supporting pressures obtained from the monitoring data of the project. A case of blow-out that appeared in the Hochiminh Metro Line No.1 project is also analyzed in this paper.

2. Face support pressure calculation

2.1. Minimum support pressure models

The minimum support pressure is determined from the lower boundary condition of too small support pressures applied at the tunnelling face, the tunnelling face will collapse and the soil will move towards the TBM. The minimum support pressure calculation was indicated in Jancsecz and Steiner (1994), Broere

(2001). In estimating the minimum support pressures in tunnelling design, the wedge stability model proposed by Jancsecz & Steiner (1994) as can be seen in Figure 1 are widely used. In this study, this model is also applied for the back analysis of the support pressure applied at the tunnelling face.

The minimum support pressure can be calculated as given by:

$$s_{min} = \sigma'_v K_{A3D} \eta_{eff} + p \quad (1)$$

Where: $\eta_{eff} = 1.3 \div 1.5$ - a safety factor; p - pore pressure.

The three-dimensional earth pressure coefficient K_{A3D} is estimated as follows:

$$K_{A3D} = \frac{\sin\beta\cos\beta - \cos^2\beta\tan\varphi - \frac{K\alpha}{1.5}\cos\beta\tan\varphi}{\sin\beta\cos\beta + \tan\varphi\sin\beta} \quad (2)$$

Where: β - determined as in Jancsecz & Steiner (1994).

The vertical soil pressure in this model is calculated as follows:

$$\sigma'_{v,a} = \frac{\gamma'z - c'}{K\tan\varphi'} \left(1 - e^{-K\tan\varphi' \frac{z}{a}} \right) + q_0 e^{-K\tan\varphi' \frac{z}{a}} \quad (3)$$

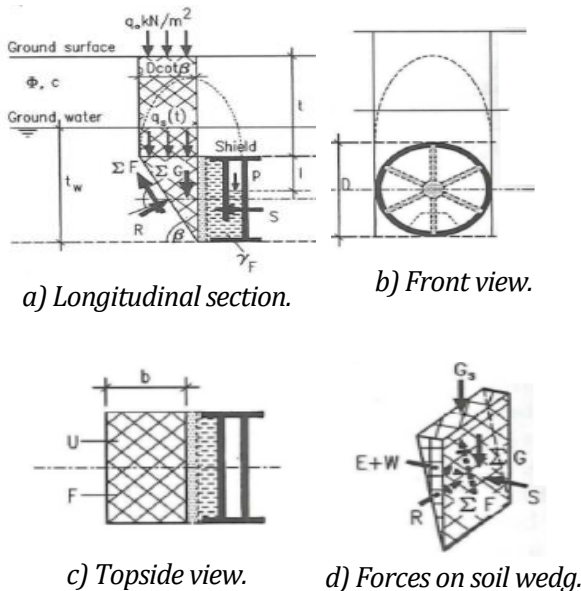


Figure 1. Wedge stability model (Jancsecz & Steiner, 1994)

2.2. Maximum support pressure models

To calculate the maximum support pressure, the blow-out condition discussed above in the case of applying too high support pressure at the tunnelling face is used. In tunnelling design and this study, blow-out models proposed by Balthaus (1991), Broere (2001) and Vu et al. (2015) can be used to obtain the maximum support pressure.

A blow-out model proposed by Balthaus (1991) is presented in Figure 2. A wedge shape soil volume pushed upward by high support pressure at the tunnelling face is assumed for calculating the maximum support pressure according to the case of blow-out occurrence. By balancing the forces of the wedge soil weight G and the support force S at the tunnelling face, the maximum support pressure in this case with safety indexes can be estimated as follows:

$$\eta = \frac{G}{S} > \eta_1 = \frac{\gamma C (B' + C \cot(45^\circ + \frac{\varphi}{2}))}{B' s(z_t)} > \eta_2 = \frac{\gamma C}{s(z_t)} \quad (4)$$

Where: C - the depth of the overburden, φ is the friction angle of the soil, γ is the volumetric weight of soil, and s is the supporting pressure applied at the tunnelling face.

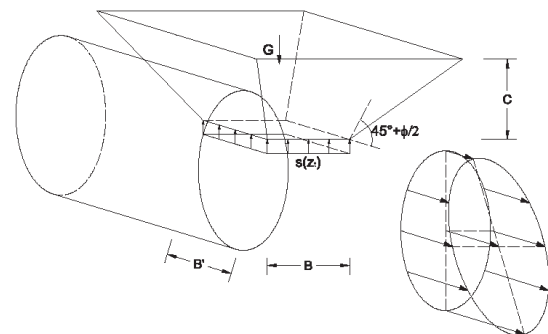


Figure 2. Calculation model of Balthaus for the safety against blow-out (Balthaus, 1991).

The other blow-out presented by Broere (2001) is shown in Figure 3. In this model, when the blowout occurs, the soil body above the tunnel is pushed upward, the shear stress at the interface between the soil column and the surrounding ground is taken into account. By balancing the equilibrium of forces on the soil body above the tunnel, the maximum support pressure applied at the tunnelling face and/or the tail can be calculated as follows:

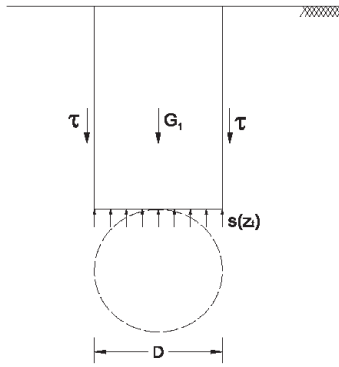


Figure 3. Blow-out model including frictions at soil column boundaries (Broere, 2001).

$$\eta s_{max} = C \left[\gamma + \frac{2c + CK_y \gamma \tan \varphi}{D} \right] \quad (5)$$

Where: c - the cohesion of the soil; K_y - the coefficient of horizontal effective stress.

In the comparison of the blow-out analysis to experimental and monitoring data in Vu et al. (2016), the maximum support pressures derived from the blow-out model proposed by Vu et al. (2016) are more accurate than the results derived from models of Balhaus (1991), Broere (2001). In this back analysis of support pressure applied in the Hochiminh Metroline No.1 project, the blow-out model proposed by Vu et al. (2016) (see Figure 4) is applied for calculating the maximum support pressure.

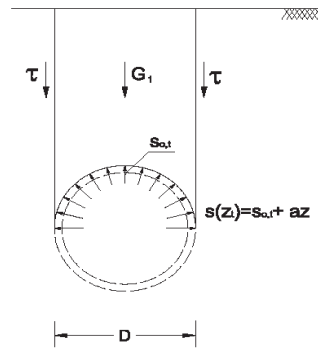
In this model, the maximum support pressure applied at the top part of the tunnelling face $s_{0,t,max}$ is calculated as follows:

$$s_{0,t,max} = \gamma \left(H - \frac{\pi}{8} D \right) + 2 \frac{H}{D} (c + HK_y \gamma \tan \varphi) - \frac{aD}{4} \quad (6)$$

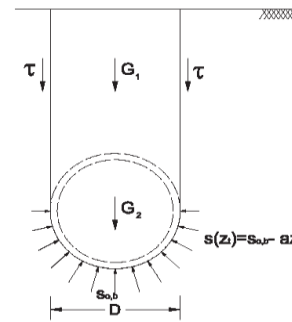
The maximum support pressure applied at the bottom part of the tunnelling face $s_{0,b,max}$ is calculated as follows:

$$s_{0,b,max} = \gamma \left(H - \frac{\pi}{8} D \right) + 2 \frac{H}{D} (c + HK_y \gamma \tan \varphi) + \gamma_T \pi d + \frac{aD}{4} \quad (7)$$

Where: H - the tunnel depth; γ_T - the tunnel unit weight; a - the grout vertical gradient.



a) Upper part.



b) Lower part.

Figure 4. Blow-out model with vertical support pressure gradient a (Vu et al., 2016).

3. The stability of tunnelling process in Hochiminh Metro Line No.1

Metroline No.1 in Hochiminh city is a pilot metro line built in Hochiminh city, Vietnam with a length of 19.7 km including 2.6 km underground under density areas of Ba Son shipyard, the Saigon Municipal Opera House and the Saigon river (see Figure 5). The soil condition along the tunnel alignment is soft soil condition with soft clay layers and silty sand layers.

Fourteen stations are located along this metro line alignment from the Ben Thanh station to the Long Binh deport. Although the Hochiminh Metroline No.1 project was launched in the year 2012, the underground work was completed in 2020.

The tunnel alignment is located under important historical buildings and density areas. Therefore, the requirements of allowable settlements and deformations of existing buildings on the surface are very strict.

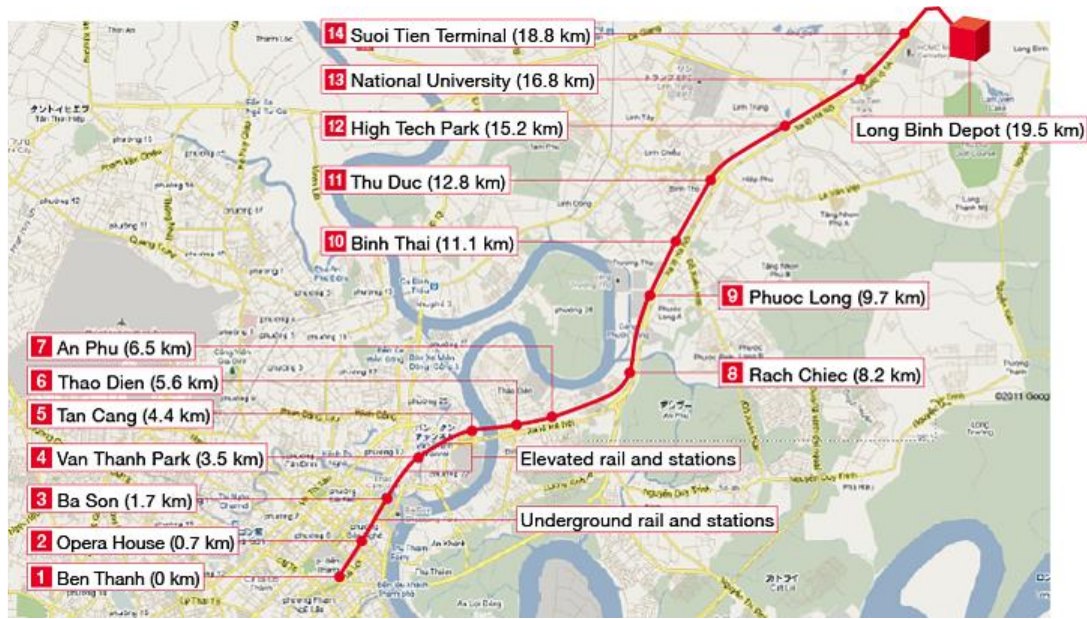


Figure 5. Hochiminh Metroline No.1 alignment in Vietnam (Vu and Le, 2020).

The tunnel segment is designed with an outer diameter of 6.65 m and the inner diameter of 6.05 m.

In order to minimize surrounding soil displacement, an Earth Pressure Balance Tunnel Boring Machine (EPB TBM) was applied for 2 tunnel lines with depths ranging from 11÷30 m. The TBMs used in this project include a cutter head with a diameter of 6.82 m and a shield with a diameter of 6.79 m as presented in Figure 6. There are 8 injection pipes on the cutter head. The support pressures applied at the tunnelling face are controlled by the operator in the control room.

The geo-condition and soil parameters used in this calculation for Hochiminh Metroline 1 are shown in Table 1. There are three soil materials including Fill layer (F) at the top, Alluvium layers and Diluvium materials at lower depths. In detail, the Fill layer is about 2 meters at the top. Alluvium layers are of about 30 meters in depth comprising Soft Clayey Silt (Ac2 and Ac3), Silty Fine Sand Layer 1 (As1) and Sand layer 2 (As2). Diluvium layers shown in Table 1 include Diluvium clayey silt (Dc) and silty sand layer (Ds).

In this study, the support pressures are calculated with some cross-sections along the excavated length of 100 m from chainage Km1+000 to Km1+1100 with the case of West Tunnel as can be seen in Figure 7.

The geocondition of the calculated tunnel alignment is shown in Table 2.

Table 1. Soil parameters applied in Hochiminh Metroline 1 Project (GIR, 2015).

Layer	γ (kN/m ³)	c (kPa)	ϕ (deg.)	K
F	19	0	28	0.6÷0.5
Ac2	16.5	14	0	0.6÷0.5
As1	19.5	0	31	0.6÷0.5
As2	19.5	0	31	0.5
Dc	21	22	0	0.5

F - Fill layer; Ac2 - Alluvium Clay Layer 2; As1:Alluvium Silty Fine Sand Layer 1; As2 - Alluvium Sand Layer 2; Dc - Hard Clay Silt; γ - unit weight; c - cohesion; ϕ - friction angle; K - Coefficient of Lateral Pressure.

Table 2. Geo conditions from Km1+000 to Km1 +1100 in Hochiminh Metroline No.1 (GIR, 2015).

Chainage	Depth of the cover (m)	Depth of soil layers (m)		
		F	Ac1	As1
Km1+000	12.07	1.16	2.96	10.68
Km1+020	12.215	1.5	2.86	11.1
Km1+040	12.115	1.61	2.52	11.66
Km1+060	12.115	1.67	2.46	12.13
Km1+080	11.905	1.6	2.54	12.38
Km1+100	11.525	1.3	2.55	12.55

For the minimum support pressures, Equations 1,2,3 is applied with the model in

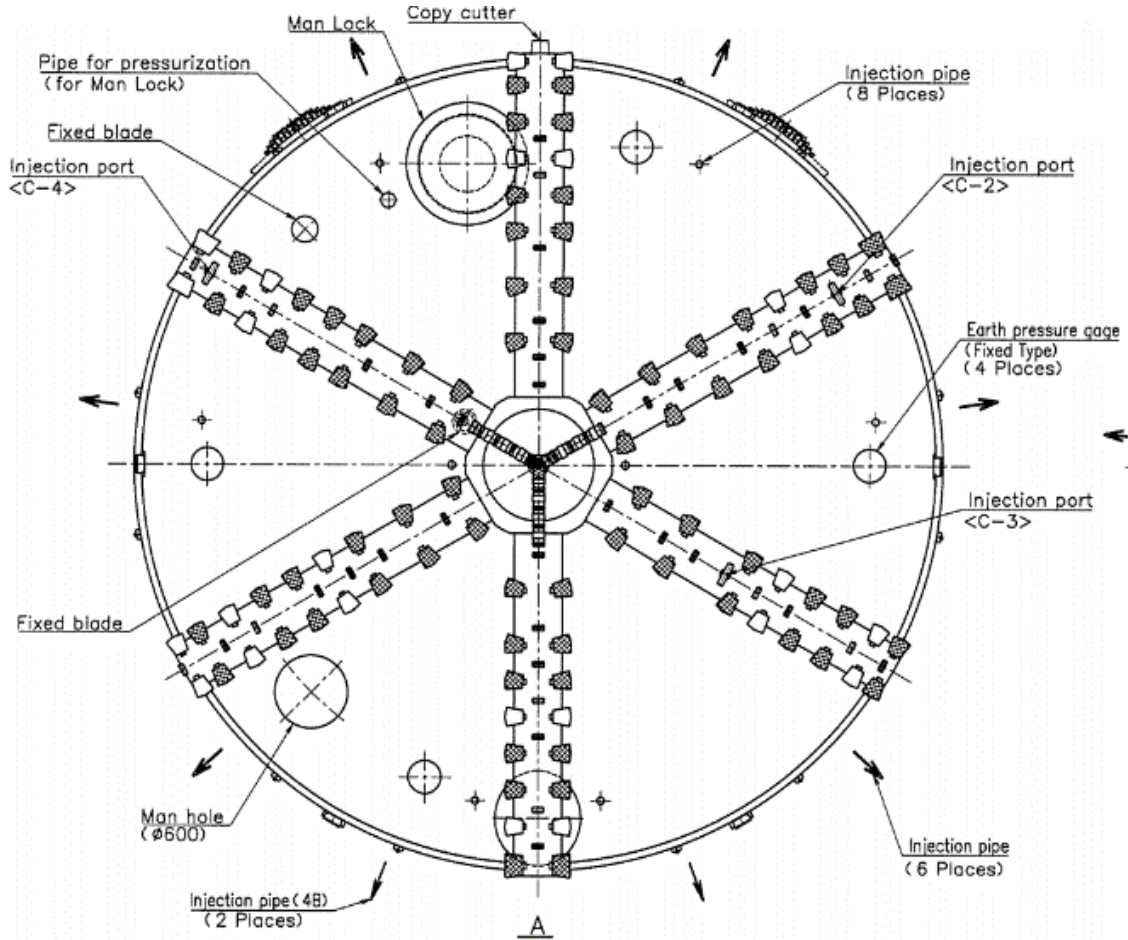


Figure 6. Excavation face of the EPB TBM in Ho Chi Minh Metroline No.1 (TDRSM, 2016).

Figure 1. The analysis of minimum support pressures are shown in Table 3.

Table 3. Minimum and maximum support pressures in Hochiminh Metro line No.1 from the chainage of Km1+000 to the chainage Km1+100.

Chainage	Supporting pressure at the tunnel face (kPa)			
	site	Analysis		
		S _{min}	S _{max}	S _{op}
Km1+000	195	113	259	163
Km1+020	175	112	260	162
Km1+040	175	111	259	161
Km1+060	190	111	259	161
Km1+080	250	109	254	159
Km1+100	240	109	249	159

The recommended operation support pressure (S_{op}) is estimated as:

$$S_{op} = S_{min} + 50kPa \quad (8)$$

The pressure of 50 kPa is the safety margin as indicated in Kanayasu et al. (1995) allowing tunnelling with shallow cover.

The maximum support pressures are derived from Equation 7 with the blow-out models shown in Figure 4. In this calculation, the “a” value as the vertical gradient of the grout can be equal to 7kN/m³ (Bezuijen and Talmon, 2008).

Site support pressures are obtained from the monitor of the TBM when tunnelling as the average value of two injection pressures at two active injection pipes on the tunnelling face.

Table 3 shows the analysis results for minimum, operation and maximum support pressures at the tunnel cross-sections at the chainages Km 1+000, Km 1+020, Km 1+040, Km 1+060, Km 1+080 and Km1+100.

Figure 8 compares the analysis results and the site support pressures. This figure shows that the operator in the project used to apply a higher support pressure than the recommended

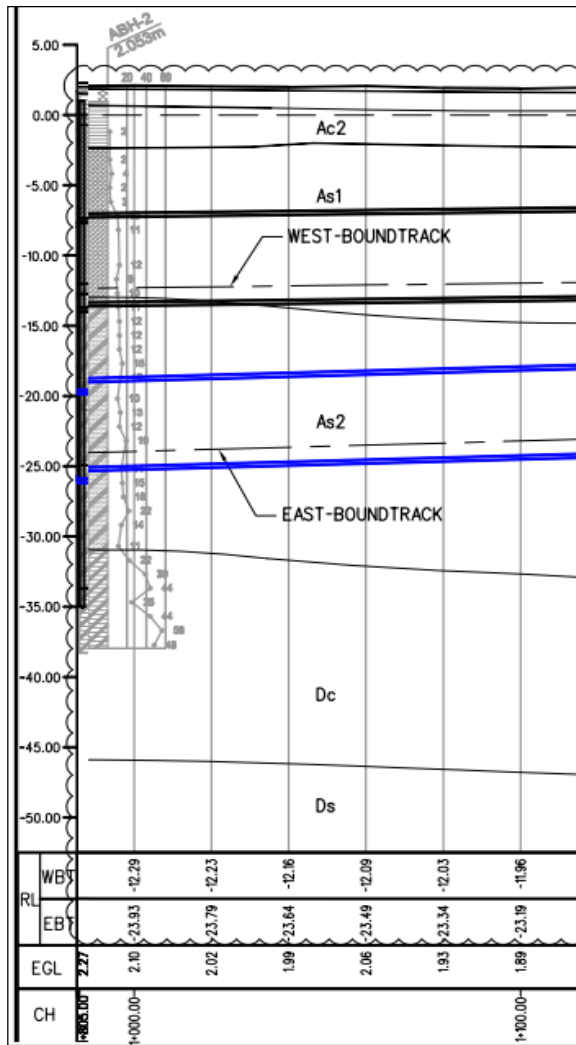


Figure 7. Geo-conditions of the tunnel alignment from chainage Km1+000 to Km1+100 in Hochiminh Metroline No.1 project (GIR, 2015).

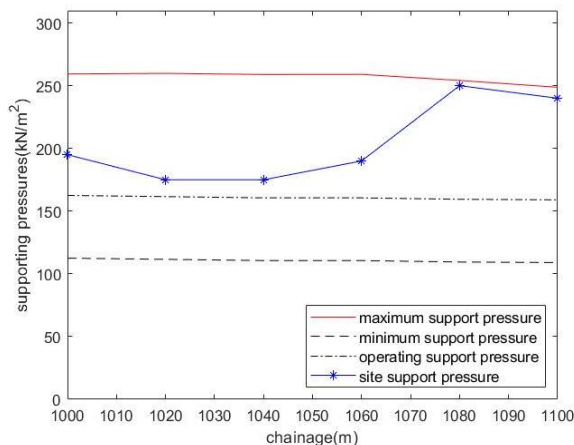


Figure 8. Back analysis for supporting pressures in Hochiminh Metroline No.1 from the chainage of Km1+000 to the chainage Km1+1100.

operating pressures when tunnelling. Especially, at the chainage of Km1+080, the applied support pressures at the EPB machine is closed to the maximum values. It could lead to the risk of a blow-out. Actually, there was a case of blow-out that appeared in this project as discussed below.

4. Blow-out case in Hochiminh Metroline No.1

In the tunnelling process in Hochiminh Metroline No.1 project, a blow-out occurrence was recorded on the date of 23rd April 2018, at the chainage of Km1+500 of the West Line as shown in Figure 9. The location of the tunnel is at the depth of -11.67m.



Figure 9. Blow-out at the chainage Km1+500 of the West Line in Hochiminh Metroline No.1.

Table 4. Soil parameters at the Saigon Municipal Opera House, Hochiminh city, Vietnam (GIR, 2015).

Layer	t, (m)	γ , (kN/m ³)	c, (kPa)	ϕ , (deg.)	E_0 , (kN/m ²)
F	1.1	18.0	0	28	2500
Ac2 & Ac3	1.7	16.0	14	0	10000
As1	13.9	19.5	0	31	16000
As2	17.0	19.5	0	31	35000
Dc	15.6	21.0	22	0	101000
Ds	-	21.0	0	34	77500

t - thickness; γ - unit weight; c - cohesion; ϕ - friction angle; E_0 - Young modulus.

At that moment, the additive polymer support fluid applied at the tunnelling face of the TBM was flowed on the road above the tunnel lining. As the observed data at the site, the support pressure at the tunnelling face at that moment was at 380 kPa. This value is the maximum polymer injection pressures pumped from line 1 and line 2 injection pipes in the TBM excavation chamber. In this case, there were 5

lines in the TBM but only 2 lines were used at that moment. Figure 9 shows the location of the blow-out occurrence. This situation is also back analyzed in this study.

The geo-condition at the Saigon Municipal Opera House location (closed to the blow-out location) is shown in Table 4.

The back analysis results for the blow-out issue are carried out from the models shown in Figure 4 and Equations 6 and 7. With the aim of investigating the effect of the cover on the stability of the tunnel, the calculation with various cover-to-diameter C/D ratios in this geo-condition is carried out in order to obtain the variation of maximum support pressures with different locations of the tunnel. When the blow-out occurs, the support pressure was observed at 380kPa. In Figure 10, the star legend (*) presents the monitoring data in this case study of blow-out corresponding to the C/D ratio of 1.76.

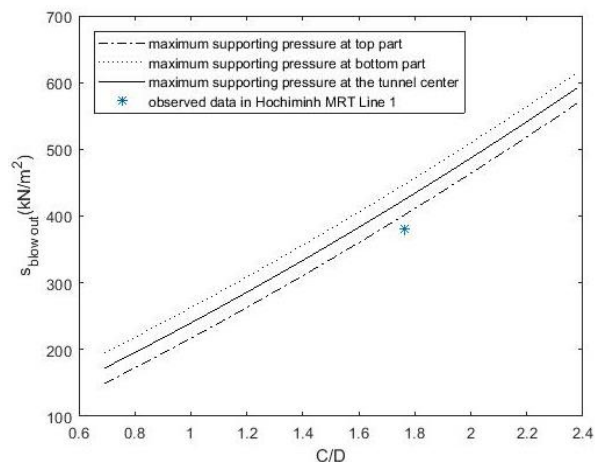


Figure 10. Back analysis for the blowout in Hochiminh Metroline No.1.

Based on the comparison in Figure 10 between the analysis data and the observed data on the site, it is clear that the blow-out pressure observed at the site has a good agreement with the blow-out pressures determined from the blow-out model proposed by Vu et al. (2016) at the tunnel center from Equations 6 and 7. The deviation in this case is only of 7%.

5. Conclusions

Tunnelling in soft soils conditions in the cities faces many challenges such as instability in the tunnelling process and large settlements on the

surface. In order to minimize the influence of the tunnelling on the surrounding soil medium and existing buildings on the surface, the selection of support pressures has an essential role in keeping the stability of the TBM. A back analysis for calculating support pressures at the tunnelling face at the Hochiminh Metroline No.1 project is carried out in this paper based on recent models of estimating maximum and minimum support pressures. A comparison between the analysis results and the site values in the Hochiminh Metroline No.1 project shows that recent models can predict accurately the minimum and maximum support pressures applied to the tunnelling process. A back analysis for the blowout case also shows a good agreement between analysis results and monitoring data. Based on the analysis, stability models indicated in the study should be applied for estimating the supporting pressures for future tunnelling projects in Vietnam.

Author contributions

The article is carried out completely by the author.

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