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Estimating the Maximum Support Pressure When Shallow Tunnelling in Soft Soils in Hanoi and Ho Chi Minh City

Minh Ngan Vu and Wout Broere

Abstract The growing economy and increasing urban population create a high demand for transportation infrastructure. When surface space becomes limited and costly, underground space becomes a crucial solution to create new infrastructure. The increasing number of tunneling projects in soft soil conditions poses challenges regarding the risks of instability during the tunneling process. The research paper presents a novel compact blowout model developed by Vu and Broere (Tunn Undergr Space Technol 138, 2023) and evaluates the maximum support pressure for two specific cases: Ho Chi Minh City (HCMC) Metro Line No.1 and Hanoi Metro Line No.3. Assessing the support pressures applied at the HCMC Metro Line No.1 site, combined with field observations on stability, results in recommendations for similar projects in Hanoi and HCMC.

Keywords Tunnelling · Soft soils · Support pressure · Metro line

1 Introduction

The demand for transportation infrastructure in urban areas is rapidly increasing due to the ongoing development of the economy and the ongoing increase in urban population. Already, more than half of the world's population lives in cities and the projected growth of the world population until 2100 by another 2.5 billion people is expected to be concentrated in urban areas, and primarily those in coastal areas. This creates challenges, as these growing cities need more space for living as well as infrastructure for transport and distribution of food, water and energy [6]. As surface

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space is often limited, use of underground space becomes essential to keep cities liveable and functioning.

As such, there is an increase in tunnelling projects in challenging locations with soft soils, high water tables, and limited right or way or cover. Tunnelling in such conditions has to deal with the challenges of keeping the tunnelling process stable, to limit the surface deformations and subsequent effects on nearby existing buildings. Especially when tunnelling with shallow overburden, these risks have to be taken into account, and the consequences of face instabilities can become significant. The precise determination of allowable support pressures, applied at the tunnelling face and in the tail void, plays an important role in ensuring the stability of the tunnelling process and minimizing the unexpected settlements at the surface. In these challenging conditions with limited cover, often the margin between the minimum support pressure, to prevent face collapse, and the maximum support pressure to prevent blowout, becomes small and difficult to handle during actual tunnelling. Therefore, a more exact determination of these limits and the margin between them becomes important to determine the possible alignment for shallow tunnels.

In limit analysis, the maximum support pressure is typically determined by considering the upper boundary condition. Leca and Dormieux [7] conducted a limit analysis for passive failure using elliptic cones originating from the tunnel face, and their findings indicated that this method yields very high allowable maximum support pressure. Similarly, Mollon et al. [10] employed a similar approach and obtained comparable high support pressures when studying solid passive failure mechanisms. Li et al. [8] and Liu et al. (2021) also reported similar results using this method. However, it is worth noting that the theoretical maximum allowable support pressure, also known as blowout pressure, estimated using the upper boundary condition in limit analysis approaches, often exceeds what has been observed in real-world field conditions and experiments. Using numerical and analytical simulation methods, Verruijt and Booker (1998) developed a 2D stability analysis, incorporating blowout. Wong and Subrin [14] and Mollon et al. (2010, 2013) introduced models that combined numerical simulation with limit analysis, encompassing 3D failure mechanisms. These models demonstrated a satisfactory agreement between the limit analysis and numerical models. However, they still tended to yield higher estimates for allowable pressures compared with real-world field observations. In a study on passive failure, Liu et al. [9] conducted one of the few available model tests in the literature. Their findings indicated a more confined area of failure than what was predicted by conical limit analysis models. Similarly, Li et al. (2009) argued that, for blowout or passive failure, it is necessary to consider a partial face mechanism rather than a full face mechanism, as the latter tends to overestimate the resistance of the soil body.

Experimental investigations in blowout have been performed. Berthoz et al. [3] conducted a series of tests using a unique reduced-scale model of an Earth Pressure Balance (EPB) machine in stratified soils to investigate both face collapse and blowout phenomena. The recorded blowout pressures from these tests showed significant differences when compared with the calculated pressures derived from the limit analysis models proposed by Subrin and Wong [11] and Wong and Subrin [14].

Despite suggestions that blowouts occur more frequently, only a limited number of blowout cases have been reported in the literature for recent tunneling projects. However, there is a notable and well-documented case of blowout that took place during the Second Heinenoord Tunnel project in the Netherlands, as reported by Bezuijen and Brassinga [4] and Broere [5].

The other popular method is limit equilibrium method for estimating the maximum support pressure when tunnelling with the studies of Balthaus [2], Broere [5], Vu et al. [12], and Vu and Broere [13]. Most of the limit equilibrium models are based on the equilibrium condition of support force from the TBM and the forces of the pushed upward soil mass above the tunnel.

This paper focuses on the state-of-the-art models for estimating the maximum allowable support pressures, both applied at the tunnelling face and the tail void [13] with limit equilibrium method and compares these with field observations.

A back analysis is performed using the predicted and measured support pressures for the HCMC Metro Line No.1 project. Combined with site observations this paper shows that lower support pressures should have been applied at the tunnelling face during boring in order to prevent the occurrence of a blowout. The allowable range of support pressures for the Hanoi Metro Line No.3 project is also calculated based on these models and recommendations for the upcoming tunnelling works are made.

2 State-of-the-Art Blowout Model for Estimating Maximum Support Pressure When Shallow Tunnelling in Soft Soils

A popular assumption in limit equilibrium models of blowout is that a soil volume acting as a rigid soil body above the TBM is pushed upward when blowout occurs. For instance, the model proposed by Balthaus (1991) assumes that the pushed-up soil volume has the shape of a truncated obelisk. Figure 1 shows the model proposed by Balthaus (1991).

The wedge soil body weight G can be estimated as follows:

$$G = \gamma C\left[AB + AC\,\mathrm{cotan}\left(45^\circ + \frac{\varphi}{2}\right) + BC\,\mathrm{cotan}\left(45^\circ + \frac{\varphi}{2}\right) + \frac{4}{3}C^2\mathrm{cotan}^2\left(45^\circ + \frac{\varphi}{2}\right)\right]. \quad (1)$$

The total support force at the top of the tunnelling face can subsequently be estimated as follows:

$$P = A.B.s, \quad (2)$$

where s is the support pressure.

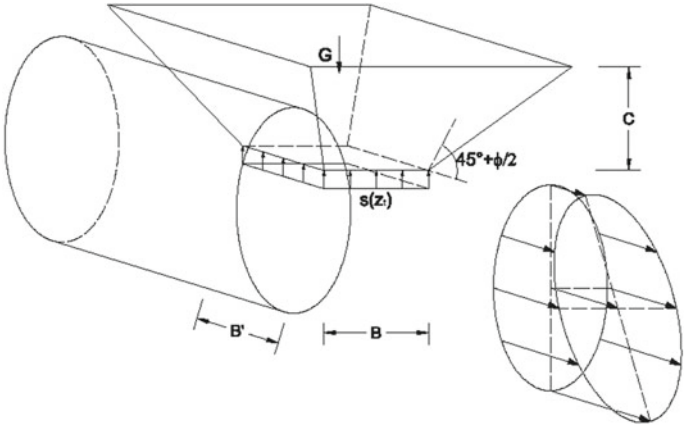

Fig. 1 Calculation model of Balthaus for the safety against blowout (Balthaus 1991)

After comparing the extent of the blowout soil volume in Balthaus (1991) to the results from numerical analysis of Soubra (2000), Li et al. (2009), Zhang et al. (Zhang et al. 2015), and experimental data in the study of Berthoz et al. [3], Vu and Broere [13] proposed a state-of-the-art model with a blowout volume that has been reduced, as can be seen in Fig. 2.

The dimensions A' and B' in this model are changed as follows:

$$A' = A + C\ \text{cotan}\left(45° + \frac{\varphi}{2}\right), \tag{3}$$

$$B' = B + C\ \text{cotan}\left(45° + \frac{\varphi}{2}\right). \tag{4}$$

Thus, the weight of soil body in Eq. (1) is then shown to be equal to:

$$G = \gamma C\left[AB + \frac{1}{2}BC\ \text{cotan}\left(45° + \frac{\varphi}{2}\right) + \frac{1}{2}AC\ \text{cotan}\left(45° + \frac{\varphi}{2}\right) + \frac{1}{3}C^2\text{cotan}^2\left(45° + \frac{\varphi}{2}\right)\right]. \tag{5}$$

The actual effective support pressure length at the tunnel roof B is influenced by various factors such as the C/D ratio (depth to diameter ratio) of the tunnel, the encountered soil types, the type of support fluids employed, and the penetration mechanism of the support fluid. Determining this specific length of B in advance is challenging, as it relies on the complex interaction between the soil conditions and the support fluid applied during the tunnelling process. Similar to Balthaus (Balthaus 1991), the soil body's weight can be expressed as a function of the effective support pressure length at the tunnel roof B, using the following equation:

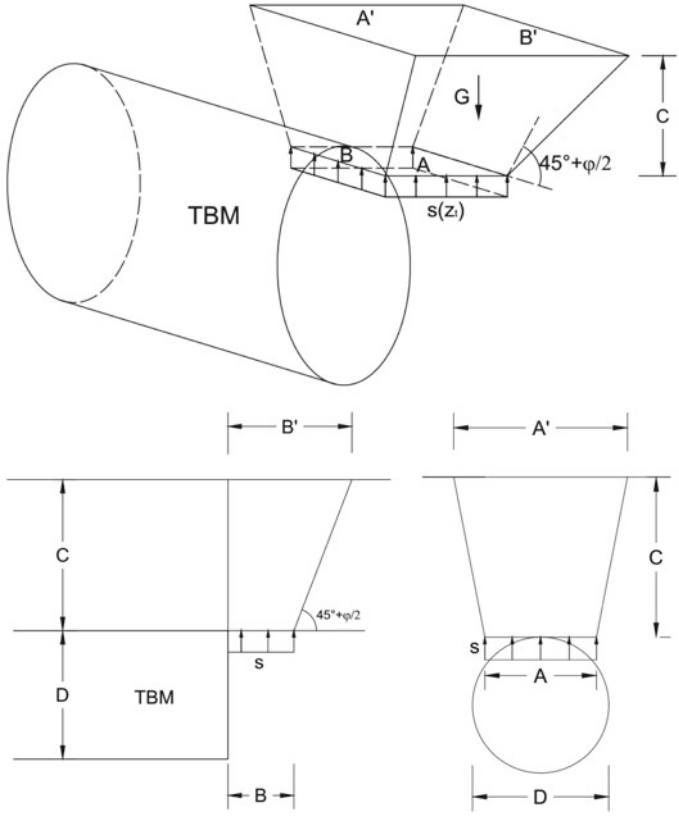

Fig. 2 Compact blowout model [13]

$$G = \alpha + \beta B. \tag{6}$$

The indexes α and β are determined as follows:

$$\alpha = \gamma C^2 \text{cotan}\left(45^\circ + \frac{\varphi}{2}\right)$$

$$\left[\frac{A}{2} + \frac{C}{3}\text{cotan}\left(45^\circ + \frac{\varphi}{2}\right)\right], \tag{7}$$

$$\beta = \gamma C\left[A + \frac{1}{2}C\,\text{cotan}\left(45^\circ + \frac{\varphi}{2}\right)\right], \tag{8}$$

and the maximum support pressure can be determined by establishing the equilibrium condition between the total support force P and the weight of the soil body G. Taking into account the safety factors, in accordance with Balthaus (1991), the maximum support pressure is given by:

$$\eta = \frac{\beta}{A.s} + \frac{\alpha}{A.B.s} > \eta_1 = \frac{\beta}{A.s} > \eta_2 = \frac{\gamma C}{s} \tag{9}$$

where η, η_1, and η_2 are the safety factors. Here, safety factor η_1 is used in case of shallow and moderate tunnels. The value of $\eta_1 = 1.1$ as indicated in Vu and Broere [13].

The validation of this model with the results from three centrifuge tests in GeoDelft and five reduced scale experiments in Berthoz et al. [3] showed a good agreement.

3 Maximum Support Pressure Estimation for HCMC Metro Line No.1 Project

The first metro line in HCMC City, Vietnam, known as Metro Line No.1 HCMC from Ben Thanh to Suoi Tien Park, is a pilot railway system constructed in the city. This metro line has a length of 19.7 km and includes a 2.6 km underground section passing through densely populated areas such as Ba Son shipyard, the Saigon Municipal Opera House, and the Saigon River (Fig. 3). The soil composition along the tunnel route consists of soft soil, including layers of soft clay and silty sand.

The metro line alignment in HCMC city comprises fourteen stations, stretching from Ben Thanh station to Long Binh depot. Although the project for HCMC Metro Line No.1 was initiated in 2012, the underground construction was finally completed in 2020. The tunnel route passes beneath significant historical structures and densely populated areas, resulting in strict requirements for allowable settlements and deformations of existing surface buildings.

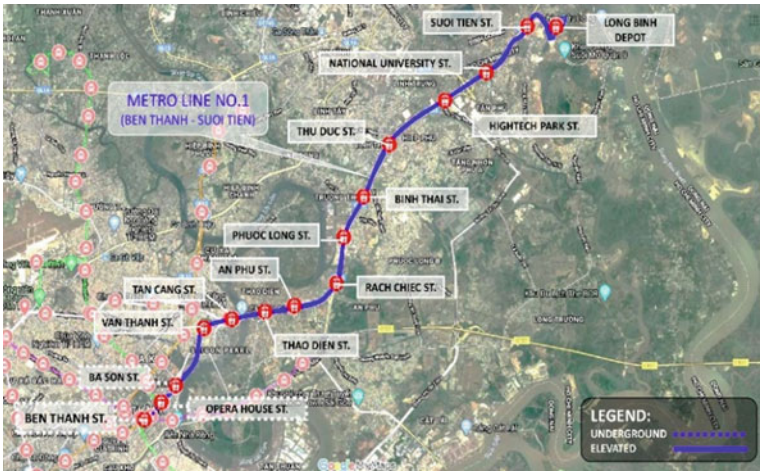

Fig. 3 HCMC metro line no.1 alignment in Vietnam

The tunnel segment has an outer diameter of 6.65 m and an inner diameter of 6.05 m. To minimize soil displacement in the surrounding area, an Earth Pressure Balance Tunnel Boring Machine (EPB TBM) was employed for two tunnel lines. These tunnels varied in depth from 11 to 30 m. The TBM used had a cutter head diameter of 6.82 m and a shield diameter of 6.79 m. The cutter head featured eight injection pipes, and the support pressures at the tunnelling face were regulated by an operator in the control room.

The geological conditions and soil parameters in HCMC Metro Line 1 outlined in Table 1 were utilized. The soil composition consists of three main soil types: a fill layer (F) at the top, alluvium layers, and diluvium materials at greater depths. Specifically, the fill layer is approximately 2 m thick, followed by alluvium layers reaching a depth of about 30 m. These alluvium layers comprise soft clayey silt (Ac2 and Ac3), silty fine sand layer 1 (As1), and sand layer 2 (As2). The diluvium layers mentioned in Table 1 consist of diluvium clayey silt (Dc) and a silty sand layer (Ds).

Figure 4 shows the designed tunnel alignment from the chainage Km 1 + 100 to Km 1 + 200. In this study, only the western line tunnel with shallow overburden is taken into account.

Figure 5 presents the analysis results of maximum support pressure from the chainage Km 1 + 100 to Km 1 + 200. A comparison to recorded support pressure at the site is also carried out. It can be seen that the site support pressures in this project tended to be larger than the predicted blowout pressures derived from the blowout model. This suggests that the operators of the Tunnel Boring Machine (TBM) had a tendency to operate it with a higher support pressure. The purpose behind this was to minimize excessive excavation of the soil in front of the TBM and enhance the stability of the tunnel face. However, this approach also posed a heightened risk of blowout occurrence. Furthermore, as depicted in the diagram, the measured blowout pressure does not correspond to the maximum applied support pressure during the excavation phase of tunnel construction. This can be attributed to the existence of additional structures like nearby buildings and road surfaces, which are not considered in the existing models due to the complexity of the calculations involved. Consequently, the true blowout pressure, accounting for the influence of surrounding structures, could potentially exceed the blowout pressure experienced in

Table 1 Soil parameters applied in HCMC metro line 1 project (based on UMRTL1-CP1b-TBMS-CGE-RPT-00073-C report)

No	Layer	γ (kN/m^3)	c (kPa)	φ (°)	K
1	F	19	0	28	0.6–0.5
2	Ac2	16.5	14	0	0.6–0.5
3	As1	19.5	0	31	0.6–0.5
4	As2	19.5	0	31	0.5
5	Dc	21	22	0	0.5

F Fill layer; *Ac* Alluvium Clay Layer; *As* Alluvium Sand Layer; *Dc* Hard Clay Silt γ Unit weight; *c* Cohesion; φ Friction angle; *K* Coefficient of lateral

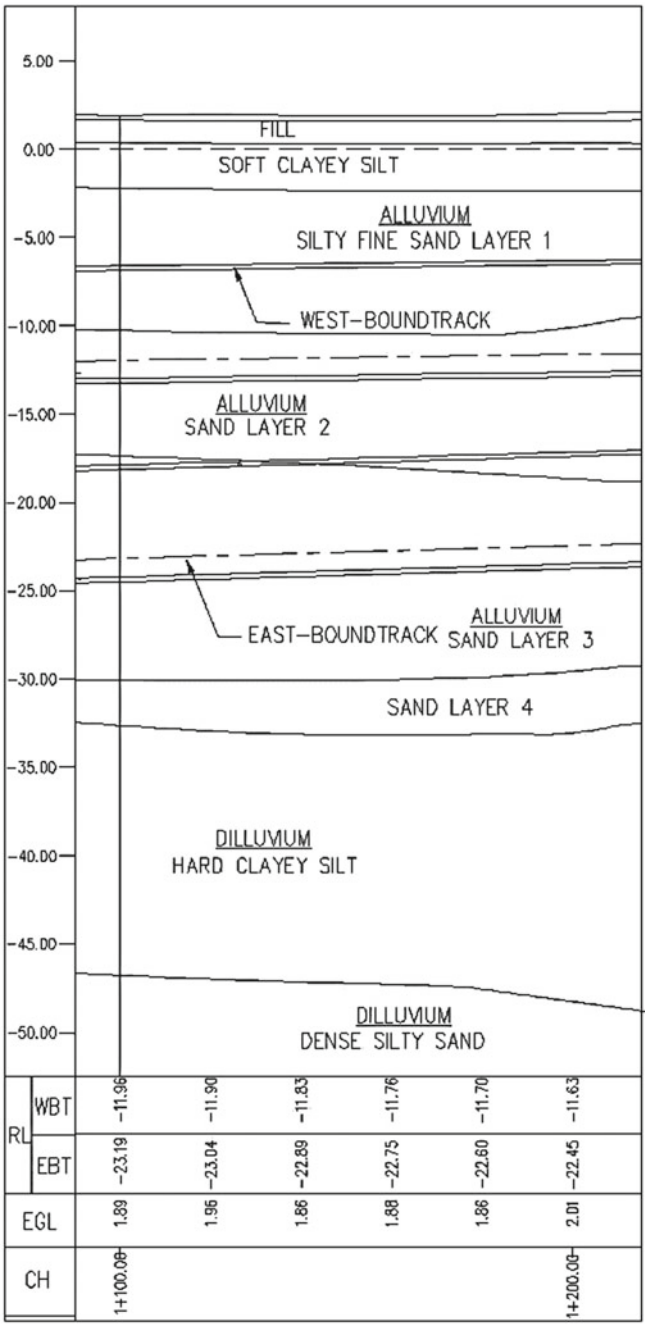

Fig. 4 Longitudinal geoprofile of metro line no.1 in HCMC city from the chainage Km 1 + 100 to Km 1 + 200

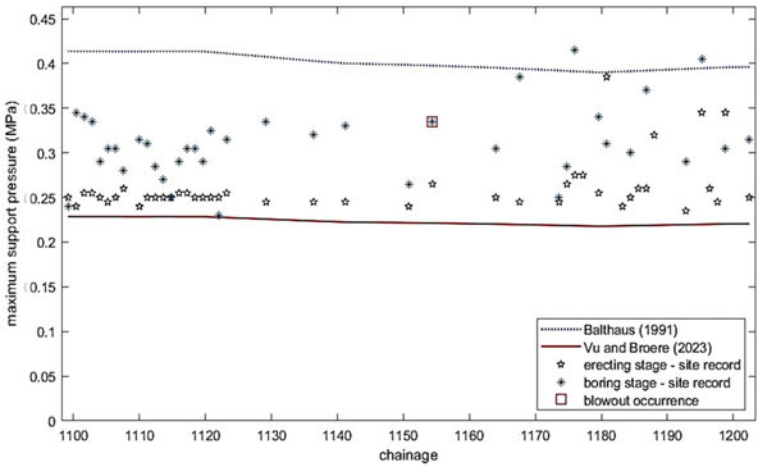

Fig. 5 Maximum support pressure estimation for HCMC Metro Line No.1 from the chainage Km 1 + 100 to Km 1 + 200

"greenfield" tunneling conditions. However, it is not advisable to incorporate these effects when estimating safe ranges for support pressure, particularly in urban areas.

A detailed analysis of the blowout occurrence at the chainage of Km 1 + 154.4 is shown in Fig. 6. It can be seen that the maximum support pressure derived from Balthaus's model is larger than the recorded site value. The Vu and Broere model reveals a more appropriate blowout pressure, which remains conservative, estimated at approximately 70% of the recorded blowout pressure observed at the site. A numerical modeling of the tunneling process was carried out under "greenfield" conditions in Plaxis 3D. It was found that the tunneling operations indeed were expected to experience failure when a support pressure of 340 kPa was applied.

4 Maximum Support Pressure Estimation for Hanoi Metro Line No.3 Project

The Hanoi Metro Line No.3 is a part of the metro network in Hanoi, Vietnam. It constitutes the second line currently under construction and is anticipated to be finalized by 2024. Spanning a distance of 12.5 km, the line encompasses 12 stations running from Nhon terminal to Hanoi station. Figure 7 illustrates the line's configuration, featuring an elevated section of 8.5 km and an underground section of 4 km.

Similar to the first metro line in Ho Chi Minh City, the underground portion of Hanoi Metro Line No.3 passes beneath densely populated areas, particularly from Kim Ma station to Hanoi station. Notably, it traverses underneath the Temple of Literature Van Mieu, a significant historical building in Hanoi. Due to the importance

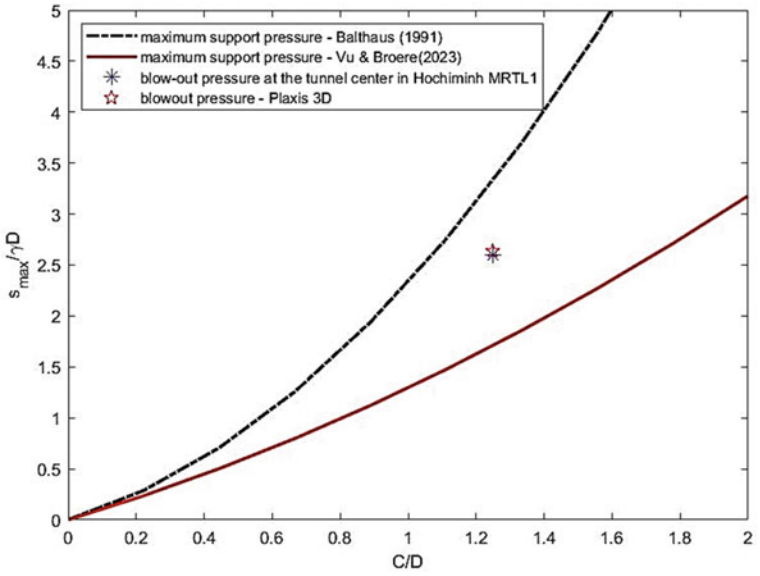

Fig. 6 Maximum support pressure estimation in blowout case in HCMC Metro Line No.1

Fig. 7 Hanoi metro line no. 3 alignment in Vietnam

of preserving nearby buildings, the tunnel construction adheres to strict settlement requirements to prevent any potential damage.

This study focuses on the underground segment between chainages Km 20 + 022 and Km 20 + 222. The geological composition of this specific segment is depicted in Fig. 8. It can be observed that the tunnel traverses beneath two to four layers of

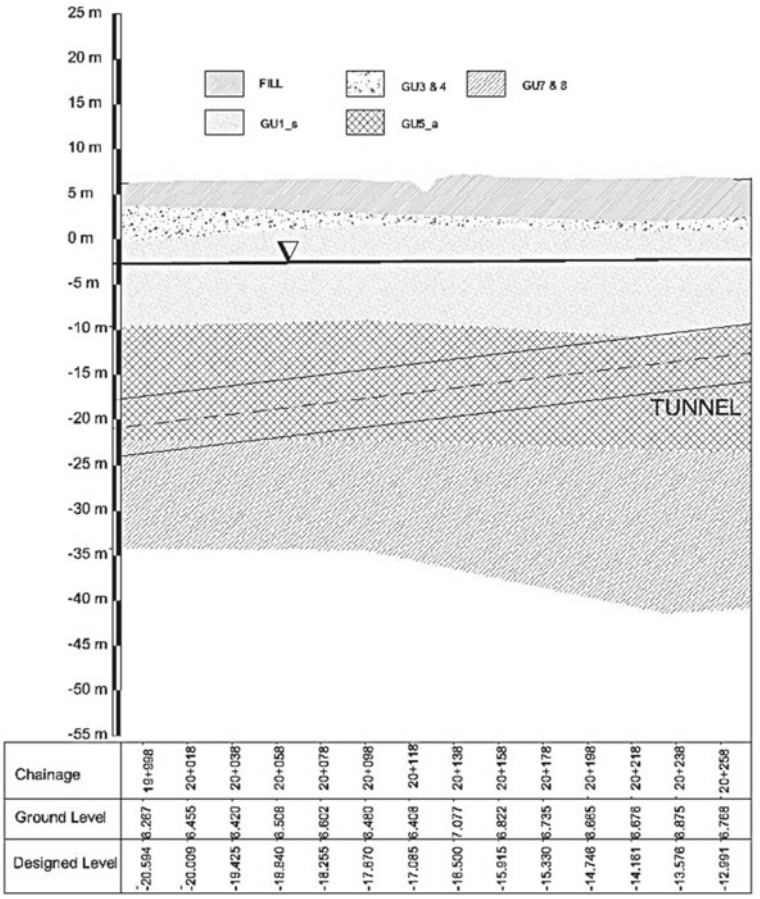

Fig. 8 Longitudinal geoprofile of Hanoi Metro Line No.3 from the chainage Km 20 + 022 to Km 20 + 222

soil in this section. The geotechnical properties of these four layers, namely the fill layer, GU1 layer (including GU3&4 layer) ranging in depth from 4 to 24 m, GU5 layer ranging in depth from 12 to 32 m below the surface, and GU7&8 layer ranging in depth from 26 to 45 m are provided in Table 2.

Figure 9 shows the predicted maximum support pressure applied to the tunnelling face from the chainage Km 20 + 022 to Km 20 + 222. It can be seen that the value of the maximum support pressure varies depending on the depth of the tunnel from about 600 kPa to 1000 kPa. With the depth of the tunnel from 25 to 30 m below the surface as can be seen in Fig. 8, the recommended support pressure that should be applied in tunnelling process in this section is from 450 to 600 kPa depending on the depth of the tunnel. Given the increased depth and cover compared with HCMC Metro Line No.1, these higher blowout pressures are expected and should allow for sufficient margin between the minimal support required and the maximum allowed pressure.

Table 2 Geotechnical properties of overburden soils of 200 m underground segment in Hanoi Metro Line No.3 project

Layer	γ (kN/m^3)	c (kPa)	φ (°)	K (–)
Fill	19	–	–	–
GU3&4	18.5	10	25	0.58
GU1s	16	5	20	0.66
GU5a	20	0	34	0.44
GU5b	20.5	0	35	0.43
GU7&8	21	0	40	0.36

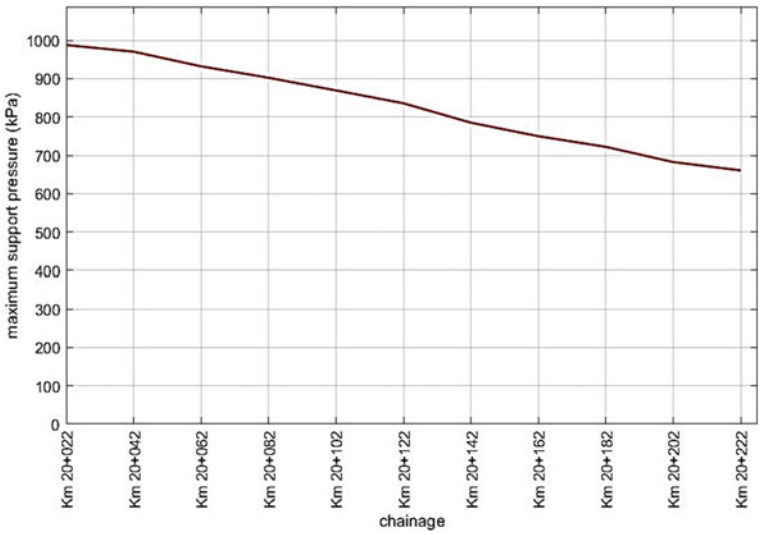

Fig. 9 Predicted maximum support pressures in Hanoi Metro Line No.3 from the chainage Km 20 + 022 to Km 20 + 222

5 Conclusions

The demand for urban transportation infrastructure is growing at a fast pace as the economy continuously develops and the urban population steadily rises. Since available surface area is restricted, underground space becomes crucial in order to maintain livable and operational cities. Consequently, the number of tunnel construction in soft soil conditions is on the rise. Particularly when tunneling with shallow depth, it becomes crucial to consider face instabilities risks. The paper uses a state-of-the-art compact blowout model developed by Vu and Broere [13] and assesses the maximum support pressure for two specific cases: HCMC Metro Line No.1 and Hanoi Metro Line No.3. The findings reveal that the support pressure exerted on the tunnel face in the HCMC Metro Line No.1 project of approximately 350 kPa was excessively high,

resulting in a blowout incident. For future projects, it is recommended to maintain a lower support pressure as a conservative limit, which in the case of the Hanoi Metro Line No.3 project, results in a recommended support pressure range from 450 to 600 kPa, depending on the tunnel's depth.

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