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TECHNICAL NOTE

Analysis of the Blow-out Occurrence When Tunnelling an Urban Shallow Tunnel: a Case Study of Ho Chi Minh Metro Line 1 Project

Minh-Ngan Vu[1] · Minh-Ngoc Vu[2,3] · Duc-Tho Pham[1] · Tuan Nguyen-Sy[4,5]

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Abstract
Several analytical models have been proposed to estimate the maximum support pressure exerting on the tunnel face during the tunnelling corresponding to the blow-out failure analysis. However, most of those models deal with homogeneous soil. Numerical solution, laboratory tests, and data from real works were used to verify those analytical models, in which the real works' data are the most relevant. However, it has rarely reported on the blow-out failure of real tunnel construction projects yet. The purpose of this paper is twofold: firstly, to report on the blow-out occurrence in the underground segment of the Ho Chi Minh Metro Line 1 project (in Vietnam), and secondly, to propose an analytical model to predict the blow-out of the tunnelling face of the tunnel in multi-layered soils. The proposed model interestingly gives the maximum support pressure very close to the site pressure recorded in Ho Chi Minh Metro Line 1 project at the position where the blow-out occurred. Moreover, a comparison between support pressures resulted from the current model for multi-layered soil, equivalent homogeneous soil and monitoring data at the project site highlights the role of multi-layer model when considering the tunnelling within a multi-layer soil.

Keywords Blow-out · Tunnelling · Shallow tunnel · Analytical model · Multi-layered soil · Ho Chi Minh Metro Line 1

1 Introduction

Underground transportation systems have been increasingly considering and building in cities to solve issues of traffic congestion, air pollution, noise, and lack of surface space. In the urban condition, the shield tunnelling method, or tunnel boring machine (TBM) is the most adaptive method for the construction of the underground transportation tunnel. Indeed, with the advance of state-of-the-art technology, using TBM when tunnelling can limit surrounding soil movements and create a precise

Extended author information available on the last page of the article tunnel lining. Moreover, TBM has become a popular in tunnel construction in urban areas due to the safety and the short construction time. Almost metro projects in the world have adopted TBM to excavate the tunnel, for examples, the Elizabeth metro line (London, England), the Grand Paris project (Paris, France), the Riyadh Metro (Saudi Arabia), the Sydney Metro (New South Wales, Australia), the Jakarta Mass Rapid Rail Transport (Java, Indonesia), and the Ho Chi Minh Metro (Hochiminh city, Vietnam).

Building underground subways is much more expensive (~ 10 times more) than a streetcar/tramway with the same distance. Therefore, optimizing the design to reduce the cost in line with satisfying all the technical criteria is essential when considering a metro project. The cover-to-diameter ratio (C/D) is an important parameter for the optimization of economic-technical solutions. Reducing the overburden depth (C) could decrease the construction and maintaining costs, ensure the safety, and lessen the travel time from the surface to the platform.

TBM provides a support pressure at the tunnel face and the support lining sequencing. The support pressure helps against the initial effective horizontal stress at the tunnel face position, which controls the tunnel face stability during the excavation. The support pressure is determined to avoid the tunnel face failure. There are two failure modes of the tunnelling faces including the collapse (active failure) and the blow-out (passive failure). The collapse or active failure takes place when the support pressure is underestimated to balance the soil self-weight, whereas the blow-out or passive failure occurs when the support pressure is overestimated and push the soil towards the ground surface. It is significant to consider carefully passive failure for shallow tunnels.

Analytical models have been proposed to determine the support pressure corresponding a given ratio C/D and satisfying the blow-out criterion. They are based on both the limit equilibrium method (Anagnostou and Kovári 1994; Broere 2001; Jancsecz and Steiner 1994; Vu 2016; Vu et al. 2015) and the limit analysis method (Atkinson and Potts 1977; Leca and Dormieux 1990; Subrin and Wong 2002; Soubra et al. 2008; Tang et al. 2014; Mollon et al. 2011; Han et al. 2016; Li et al. 2019). In the case of limit equilibrium method, Horn (Horn 1961) presented firstly a 3D wedge model for analysing the active failure. Anagnous and Kovari (Anagnostou and Kovári 1994), Broere (Broere 2001), Jancsecz and Steiner (Jancsecz and Steiner 1994), and Anagnous (Anagnostou 2012) then developed the Horn's model to obtain more precise analysis when taking into account the arching effect. For the passive failure, few models have been proposed including a simple assumption of a pushed upward 3D obelisk soil mass model introduced by Balthaus (Balthaus 1991), and simple 2D model proposed by Broere (Broere 2001) including the shear forces along the pushed upward soil body. Other authors applied limited analysis methods to determine the upper and lower boundaries of support pressure including Leca and Dormieux (Leca and Dormieux 1990), Mollon et al. (Mollon et al. 2011), Soubra et al. (Soubra et al. 2008), and Quarmout et al. (Qarmout et al. 2019).

Sloan (Sloan 2013) and Han et al. (Han et al. 2016) compared these two approaches and stated that the limit analysis methods exhibit more strictly in theoretical basic than the equilibrium analysis. In the methods based on limit analysis, the soil behaviour (i.e., stress–strain relation) is considered and equilibrium is verified everywhere. This is not the case in the limit equilibrium methods. However, the limit equilibrium methods are still widely used in practical engineering due to its clear physic meaning and its simplicity (Zizka and Thewes 2016). Moreover, the stress–strain relationship introduced into the limit analysis methods is normally obtained from the laboratory tests on the sample, which may not cover the ground of the work scale due to its important length compared to the sample size. A confident model should be validated against numerical and/or experimental results, as well as the real case studies. Berthoz et al. (Berthoz et al. 2012) showed a large difference between experimental data and analysis results obtained from limited analysis. Moreover, most of previous studies have dealt with the case of homogeneous soil. The multi-layered soils have been rarely considered.

Different numerical methods, such as finite different method (FDM) (Chen et al. 2013; Senent et al. 2013; Senent and Jimenez 2015), finite element method (FEM) (Lu et al. 2014; Ibrahim et al. 2015), and discrete element method (DEM) (Funatsu et al. 2018; Zhang et al. 2011; Chen et al. 2011), have been used to analyse the tunnel face stability. However, many assumptions should be also made in the numerical modelling (e.g., soil behaviour or stress–strain relation, small strain). It exists also a few laboratory tests based on centrifuge testing method (Han et al. 2016; Al-Hallak 1999; Chambon and Corté 1994) to study the failure mechanism of the tunnel face, as well as provide data for the validation of analytical models and numerical modelling. Laboratory test is performed in an ideal condition and still in a small scale. The best way to validate and evaluate both analytical and numerical models is to confront their results to the real cases. However, real blow-out case in the construction of subway tunnel has been rarely mentioned or documented. To the best of the authors' knowledge, blow-out cases occurred during the construction in the world are rarely recorded. Among them, the blow-out case when building the Second Heinenoord Tunnel project in Netherlands is reported by Bezuijen and Brassinga (Bezuijen and Brassinga 2006).

This paper aims to report a study case of the Ho Chi Minh Metro Line 1 project where a blow-out occurred. All the data for 300 m of an underground segment in which there were the blow-out occurrence, including the tunnel geometry and depth, soil layers' thickness and properties, as well as the water table, are carefully documented. These data should be valuable for the validation of analytical and empirical models proposed to predict the maximum support pressure exerting on the tunnelling face of a tunnel.

The minimum and maximum support pressures are estimated and compared to the site pressure. The underground subway tunnel is under three soil layers in the considered project. Therefore, appropriated multi-layer models should be used to study the collapse and the blow-out. The well-known Jancsecz and Steiner's (Jancsecz and Steiner 1994) 3D wedge stability model is used to estimate the minimum support pressure corresponding to the active failure criterion. An extension of Broere (Broere 2001) and Vu et al. (Vu et al. 2015) to the case of multi-layer soil is performed to determine the maximum support pressure corresponding to the blow-out criterion. Applying the proposed muliti-layer model to the Ho Chi Minh Metro Line 1 project data, a maximum support pressure is obtained, which is very close to the site pressure. This shows the validation of the proposed model.

A comparison between one layer models of Broere (Broere 2001) and Vu et al. (Vu et al. 2015) and the current multi-layer model, as well as the site observation, is also made. This comparison clarifies how important it is to give consideration of the real multi-layer soil in the passive failure analysis in compared to the equivalent homogeneous soil.

2 Ho Chi Minh Metro Line 1

2.1 Project Overview

The metro line 1 in Ho Chi Minh city is the first metro project in Vietnam. The project started construction in 2012 and is scheduled for completion in late 2023, with operations starting in 2024. The length of this line is 19.7 km including 2.6 km of subway tunnel under densely populated areas and important historical patrimonies, such as Ba Son shipyard, the Saigon Municipal Opera House and the Saigon river. The tunnel section is designed with an inner diameter of 6.05 m and an outer diameter of 6.65 m (i.e., 0.3 m thickness of the concrete lining). There are two tunnels (West and East tunnels) severing the transport in two directions. This metro line including 14 stations relates Ben Thanh to Suoi Tien Terminal. The plan of the Metro Line 1 in Ho Chi Minh city is shown in Fig. 1.

The underground part of the metroline was excavated by an earth pressure balance (EPB) TBM that its face is shown in Fig. 2. The TBM is charaterized by 6.82 m of the cutter head diameter and 6.79 m of the shield diameter. There are eight injection pipes on the cutter head to apply the support pressures on the tunnelling face, which is controlled by the operator in the control room.

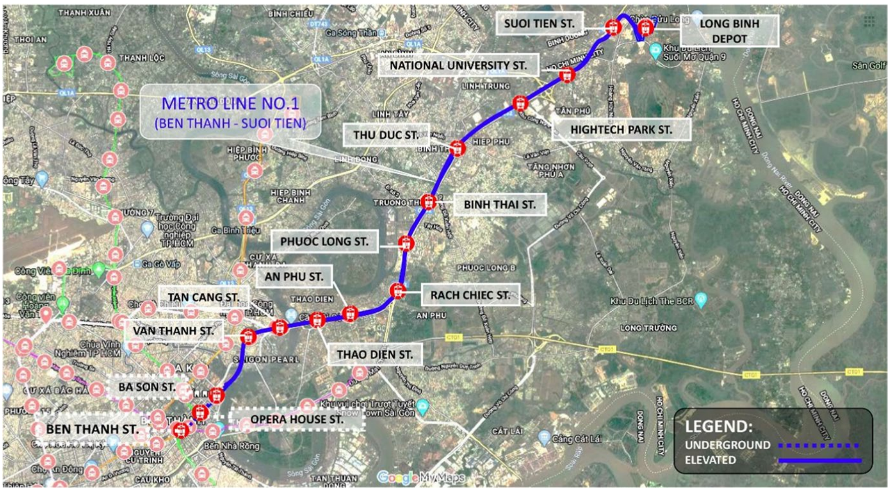


Fig. 1 Plan of Ho Chi Minh Metro Line 1 (Ho Chi Minh City, Vietnam)

Fig. 2 Excavation face of the EPB TBM in Ho Chi Minh Metro Line 1

From the ground surface to 50 m of depth, there are three soil layers: ~ 2 m in thickness of fill layer; ~ 30 m in thickness of alluvium soils, and diluvium soils. The alluvium layer is split into (from the top to the bottom of this layer) soft clayey silt (Ac2 and Ac3), silty fine sand layer 1 (As1), and sand layer 2 (As2). Diluvium layer includes diluvium clayey silt (Dc) just below alluvium layer and then silty sand layer (Ds). Median main geotechnical properties of these soil layers are recapitulated in Table 1. Figure 3 shows the longitudinal geological profile of 300 m from km 0 + 940 to km 1 + 240, which also includes West and East tunnels (top, bottom, and track lines). As a reminder, the length of underground segment is 2.6 km. We cannot show all this 2.6 km of longitudinal profile for 50 m of depth. Therefore, only 300 m representative segment is shown. The cover depth (C) of the upper tunnel varies from 8.3 to 9.5 m for this 300 m of underground segments, as shown in Fig. 3 (or see Table 2). For all 2.6 km of underground segment in the Ho Chi Minh Metro line 1 project, C varies from 8.2 to 10.4 m with the West tunnel.

Table 1 Main geotechnical parameters of soil layers surrounding the underground subways

Layer	Description	Unit weight γ (kN/m^3)	Cohesion c (kPa)	Friction angle ψ (°)	Coefficient of lateral K
1	Fill layer F	19	10	28	0.6–0.5
2	Alluvium clay layer 2 Ac2	16.5	10	24	0.6–0.5
3	Alluvium silty fine sand layer 1 As1	20.5	0	31	0.6–0.5
4	Alluvium sand layer 2 As2	20.5	0	31	0.5
5	Hard clay silt Dc	20.5	22	0	0.5

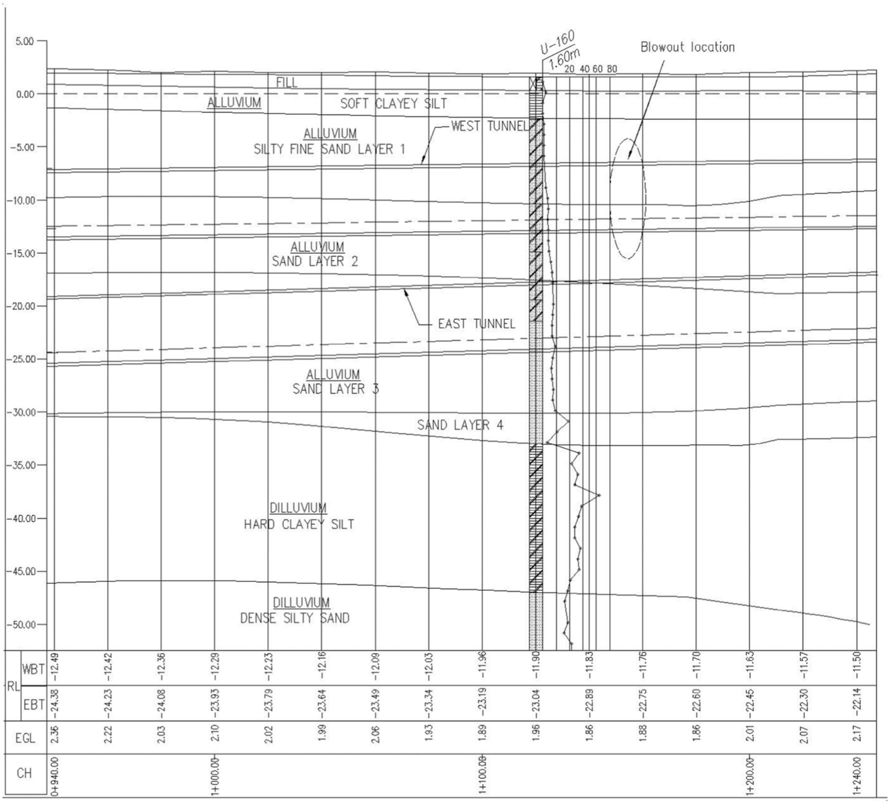


Fig. 3 Longitudinal geological profile of the underground segment in the Ho Chi Minh metro line 1 project from chainage km 0 + 940 to km 1 + 240

2.2 Blow-out Occurrence

During the construction of the underground segment in Ho Chi Minh metro line 1, a blow-out (passive failure) occurrence was recorded on 23th April 2018 at the chainage km 1 + 154.4 of the West tunnel. The location of the blow-out event is shown in Fig. 4. The blow-out was recognized by the fact that the injected fluid spouted on the street above. However, there is no significant damage for the tunnel under construction and its environment.

The support pressure at the tunnelling face was observed at 335 kPa when the blow-out took place. This is the maximum value of the polymer injection pressure of line 1 and line 2 in the excavation chamber of the TBM (there were five lines in the TBM but only two lines were used at that moment).

Table 2 Cover depth of 300 m underground tunnel from chainage km 0 + 940 to km 1 + 240

Chainage	Depth of the cover (m)	Depth of soil layers (m)			
		Fill	Ac1	As1	As2
km 1 + 238.4	8.31	1.93	2.63	6.84	3.55
km 1 + 220.4	8.28	1.79	2.68	7.08	3.36
km 1 + 202.4	8.29	1.73	2.69	7.62	2.88
km 1 + 180.8	8.21	1.59	2.68	8.14	2.43
km 1 + 220.4	8.27	1.61	2.66	8.07	2.56
km1 + 164	8.30	1.60	2.65	8.09	2.59
km 1 + 154.4	8.33	1.58	2.63	8.10	2.65
km 1 + 119.6	8.50	1.66	2.60	8.09	2.78
km 1 + 99.2	8.50	1.56	2.56	8.06	2.94
km 1 + 080	8.61	1.56	2.53	8.01	3.13
km 1 + 059.6	8.80	1.64	2.49	7.95	3.35
km 1 + 040.4	8.80	1.51	2.45	7.93	3.54
km 1 + 039.2	8.80	1.51	2.45	7.92	3.56
km 1 + 020	8.89	1.46	2.41	7.92	3.73
km 1 + 000.8	9.03	1.46	2.37	7.94	3.89
km 0 + 980.4	9.04	1.31	2.33	8.03	3.99
km 0 + 960	9.30	1.42	2.28	8.20	4.03
km 0 + 940.8	9.49	1.47	2.24	8.42	3.99

Fig. 4 Blow-out at the chainage km 1 + 154.4 of the West tunnel when tunnelling of Ho Chi Minh metro line 1

3 Estimation of Maximum and Minimum Support Pressure

In this section, an analytical blow-out model is proposed to estimate maximum support pressure based on the limit equilibrium method. This model takes into account the nappe (i.e. hydrostatic pore pressure) and geological multi-layered soils surrounding the tunnel, whereas Jancsecz and Steiner (Jancsecz and Steiner 1994) model is used to determine the minimum support pressure. The estimation of maximum and minimum support pressure will be compared to the recorded site support pressure in Section 4.

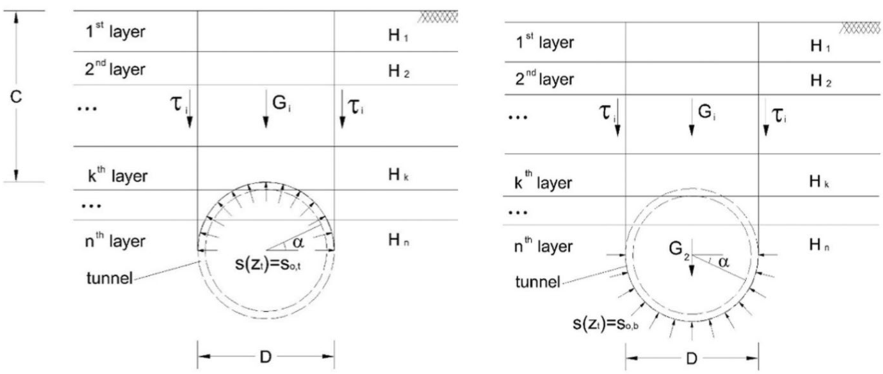
3.1 Multi-layered Soil Blow-out Model — Maximum Support Pressure

The derivation method consists in considering the equilibrium of the soil column above the tunnel in 2D section passing the tunnelling face. This derivation method is firstly proposed by Broere (Broere 2001). To sake of clarity, the novelty of the proposed model compared to Broere's (Broere 2001) model consists of giving consideration the multi-layered soils, the support pressure gradient at the tunnel face, and the lower part equilibrium. The comparison between the current model and Broere's (Broere 2001) model will be made in Section 4 when considering the Ho Chi Minh Metro Line 1 project to highlight the impact of multi-layered soils.

The main assumption is that an exceeded support pressure applies both

(1) on the top of the tunnel, which may push upwards the soil column above the tunnel (Fig. 5a)
(2) on the lower part of the tunnel, which shoves the soil column above the tunnel and the tunnel itself (Fig. 5b).

The pressure exerting on both upper and lower parts of the tunnel is a function of the depth (z coordinate). Indeed, the EPB TBM can apply a non-uniform pressure on the tunnel face to account the initial stress gradient in z coordinate. This gradient is significant and should be taken into account for a very shallow tunnel. By noting $\delta_b = \mathrm{d}p/\mathrm{d}z$ the support pressure gradient, the support pressure on the upper part is



(a) upper part (b) lower part

Fig. 5 Blow-out analysis for a shallow tunnel within multilayer soils

$$s = s_{0,t} + \delta_p R\cos\alpha \tag{1}$$

and on the lower part,

$$s = s_{0,b} - \delta_p R\cos\alpha \tag{2}$$

where $s_{0,t}$ and $s_{0,b}$ are the support pressure values at the top and the bottom of the tunnel; R is the radius of the tunnel, and α is the angle as shown in Fig. 6. A discussion about the support pressure gradient in the reality of the tunnel construction can be referred to Bezuijen and Talmon (Bezuijen and Talmon 2008).

The determination of the maximum support pressure at the upper and lower parts is based on the equilibrium condition of the soil column upstairs of the tunnel and the assembly of the soil column and the tunnel. According to the scheme in Fig. 5 for multilayer soil overhead the tunnel, the mass of the soil column G_1 is:

$$G_1 = \sum_{i=1}^{n} G_{1,i} = \sum_{i=1}^{n} D\gamma_i H_i - \frac{\pi}{8} D^2 \frac{\sum_{i=k}^{n} H_i \gamma_i}{\sum_{i=k}^{n} H_i} \tag{3}$$

where $G_{1,i}$ is the body weight of the layer ith; H_i is the ith layer thickness; γ_i is the density of the soil layer ith; and D is the tunnel diameter.

When the soil column above the tunnel starts moving upwards, the shear force between this soil column and its surrounding soil can be estimated as:

$$2T = 2\sum_{i=1}^{n} T_i = 2\sum_{i=1}^{n} H_i (c_i + \sigma'_{h,i} \tan\varphi_i) = 2\sum_{i=1}^{n} H_i (c_i + \sigma'_{h,i} \tan\varphi_i) \tag{4}$$

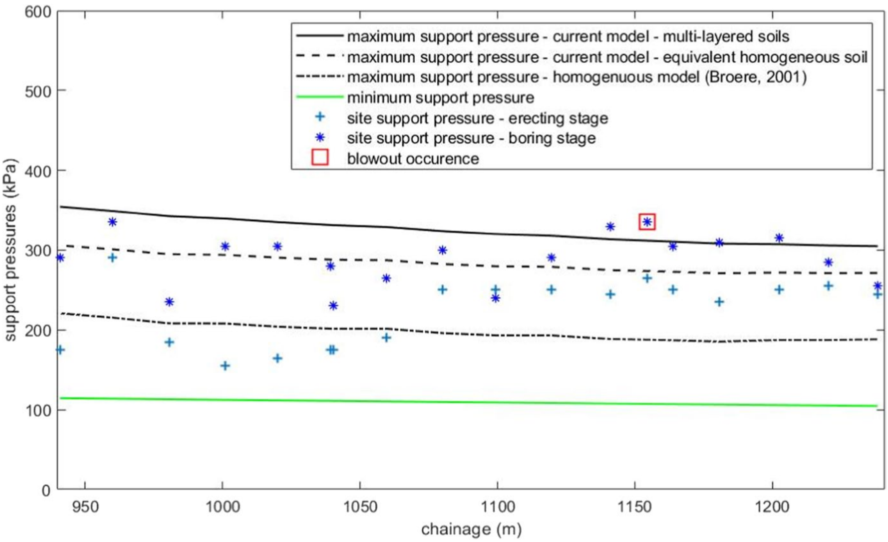


Fig. 6 Support pressure at the tunnelling face from km0 + 940.8 to km 1 + 238.40

where T_i is the shear force on the one side of the layer ith; c_i is the cohesion of the soil layer ith; φ_i is the friction angle of the soil layer ith; $\sigma'_{h,i} = K_{0,i}\sigma'_{v,i} = K_{0,i}(\gamma_i - \gamma_w)z_i$ is the effective horizontal stress with z_i as the depth of the soil layer ith.

The mass of the tunnel liner is approximated by:

$$G_2 = \pi\gamma_T Dd \tag{5}$$

where d is the tunnel lining thickness; γ_T is the density of the tunnel lining.

The total support pressure applying on the upper part is estimated as follows:

$$S_v = \int_0^\pi d\alpha \int_0^R \left(s_{0,t} + \delta_p R\cos\alpha\right)\sin\alpha dr = Ds_{0,t} + \delta_p \frac{D^2}{4} \tag{6}$$

Considering the upper part, the equilibrium condition of the soil column above the tunnel is verified the following equation:

$$G_1 + 2T = S_v \tag{7}$$

Substituting Eqs. (3), (4), and (6) into Eq. (7) yields:

$$\sum_{i=1}^{n} D\gamma_i H_i - \frac{\pi}{8}D^2 \frac{\sum_{i=k}^{n} H_i\gamma_i}{\sum_{i=k}^{n} H_i} + 2\sum_{i=1}^{n} H_i\left(c_i + \sigma'_{h,i}\tan\varphi_i\right) = Ds_{0,t} + \delta_p \frac{D^2}{4} \tag{8}$$

Rearrangement this equation gives the maximum support pressure at the top of the tunnel, beyond this value the blowout occurs:

$$s_{0,t} = \sum_{i=1}^{n} \gamma_i H_i - \frac{\pi}{8}D \frac{\sum_{i=k}^{n} H_i\gamma_i}{\sum_{i=k}^{n} H_i} + 2\sum_{i=1}^{n} \frac{H_i}{D}\left(c_i + \sigma'_{h,i}\tan\varphi_i\right) - \delta_p \frac{D}{4} \tag{9}$$

Considering the lower part, the equilibrium equation of the assembly of the tunnel and the column soil upstairs of the tunnel reads:

$$G_1 + G_2 + 2T = S_v \tag{10}$$

Introducing Eqs. (3), (4), (5), and (6) into Eq. (10) and rearrangement yield the maximum support pressure at the bottom of the tunnel, beyond this value the passive failure takes place.

$$\begin{aligned} s_{0,b} = \sum_{i=1}^{n} \gamma_i H_i - \tfrac{\pi}{8}D \tfrac{\sum_{i=k}^{n} H_i\gamma_i}{\sum_{i=k}^{n} H_i} + 2\sum_{i=1}^{n} \tfrac{H_i}{D}\left(c_i + \sigma'_{h,i}\tan\,\varphi_i\right) \\ +\pi d\gamma_T - \delta_p \tfrac{D}{4} \end{aligned} \tag{11}$$

In the case of a homogeneous cover soil with homogeneous properties: density γ; cohesion c, friction angle φ, coefficient of lateral K_0, the maximum support pressure at the top and the bottom of the tunnel becomes

$$s_{0,t} = \gamma(H - \frac{\pi}{8}D) + 2\frac{H}{D}(c + \frac{1}{2}HK_0\gamma'\tan\,\varphi) - \delta_p \frac{D}{4} \tag{12}$$

$$s_{0,b} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + \frac{1}{2}HK_0\gamma'\tan\varphi\right) + \delta_p\frac{D}{4} + \pi d\gamma_T \tag{13}$$

Introducing the cover notion $C=H+D/2$ into these two equations, the maximum support pressure can be expressed as a function of cover-to-diameter (C/D) ratio such as:

$$s_{0,t} = 2DK_0\gamma'\tan\varphi\left(\frac{1}{2} + \frac{C}{D}\right)^2 + (\gamma D + 2c)\left(\frac{1}{2} + \frac{C}{D}\right) - \frac{\pi}{8}\gamma D - \delta_p\frac{D}{4} \tag{14}$$

$$s_{0,b} = 2DK_0\gamma'\tan\varphi\left(\frac{1}{2} + \frac{C}{D}\right)^2 + (\gamma D + 2c)\left(\frac{1}{2} + \frac{C}{D}\right) - \frac{\pi}{8}\gamma D + \delta_p\frac{D}{4} + \pi d\gamma_T \tag{15}$$

In summary, a simple equilibrium analysis is developed in this section for estimating the maximum support pressure applying on the top and the bottom of the tunnel to avoid the blowout failure when excavating the tunnel by using the TBM. The support pressure should be lower than the value resulted from Eqs. (9) and (11) for a multi-layered soil, and from Eqs. (12) and (13) (or (14) and (15)) for particular case of a homogeneous cover soil).

In the literature, an equivalent homogeneous soil is usually proposed to consider implicitly multi-layered soil, in which a geotechnical property of the equivalent soil is estimated as the average of this property of multi-layered soils. The presence of the multi-layered model in this study (i.e. Equations (9) and (11)) allows to evaluate the role of multi-layered aspect (see in Section 4 below).

3.2 Minimum Support Pressure

As a reminder, the minimum support pressure exercising on the face is required to avoid the collapse. Several models have been proposed to estimate the minimum support pressure (Anagnostou and Kovári 1994; Broere 2001; Jancsecz and Steiner 1994; Subrin and Wong 2002; Broms and Bennermark 1967; Dias et al. 2008; Murayama Endo Hashiba Yamamoto Sasaki 1966; Fang et al. 2012, 2015); among them, the 3D wedge stability model proposed by Jancsecz and Steiner (Jancsecz and Steiner 1994) has been widely used for this purpose. The principle of that model is based on the limit equilibrium analysis of a 3D wedge-silo failure mechanism, which considers the effect of soil arching above the tunnel face. According to Jancsecz and Steiner (Jancsecz and Steiner 1994) model, the minimum support pressure is estimated as follows:

$$s_{\min} = \eta K_{A3,i}\sigma_{v,i}' + p \tag{16}$$

where $\eta=1.5$ is the safety factor; p the pore pressure, $\sigma_{v,i}'$ is the effective soil pressure, and $K_{A3,i}$ is the three-dimensional earth pressure coefficient of the soil layer ith depth.

$K_{A3,i}$ is determined by the following formulation:

$$K_{A3,i} = \frac{\sin\beta_i\,\cos\beta_i - \cos^2\beta_i\,\tan\varphi_i - \frac{K_i\alpha}{1.5}\cos\beta_i\,\tan\varphi_i}{\sin\beta_i\,\cos\beta_i + \sin^2\beta_i\,\tan\varphi_i} \tag{17}$$

with

$$K_i = \frac{1 - \sin\varphi_i + \tan^2\left(\frac{\pi}{4} - \frac{\varphi_i}{2}\right)}{2} \tag{18}$$

$$\alpha = \frac{1 + 3\frac{C}{D}}{1 + 2\frac{C}{D}} \tag{19}$$

The value of β_i indicated in Jancsecz and Steiner (Jancsecz and Steiner 1994) depends on the ratio of C/D and soil friction angle φ_i.

The effective vertical stress for the soil layer ith is resulted from the Terzaghi theory as indicated in Broere (Broere 2001) as:

$$\sigma'_{v,i} = \frac{a\gamma_i{}' - c_i}{K_i\tan\varphi_i}\left(1 - e^{K_i\tan\varphi_i\frac{z}{a}}\right) + \sigma'_{v,i-1}(t_i)e^{-K_i\tan\varphi_i\frac{z}{a}} \tag{20}$$

where $a = R\frac{1}{1+\tan\beta}$ is a relaxation length with three-dimensional arching.

We only summarize the main formulations of the wedge stability model from Eqs. (16) to (20). The detail about the derivation of these formulations can be referred to Jancsecz and Steiner (Jancsecz and Steiner 1994).

The recommended operation support pressure is calculated by adding 50 kPa of safety margin into the minimum support pressure such as (Kanayasu et al. 1995):

$$s_{op} = s_{\min} + 50\ \text{kPa} \tag{21}$$

The maximum support pressure is then estimated from Broere (Broere 2001) model, and the current model presented in the previous section for both real data from multi-layered soil (see Eqs. (9)) and equivalent homogeneous soil (see Eqs. (12) or (14)). The support pressure gradient δ_p is equal to 7 kP/m as recommended by Bezuijen and Talmon (Bezuijen and Talmon 2008) behind the TBM at the end of the monitoring.

4 Stability Analysis in Ho Chi Minh Metro Line 1 Project

4.1 Support Pressure from km 0 + 940 to km 1 + 240

This section is devoted to considering the face tunnel stability of the West tunnel from chainage km 0+940 to km 1+240 within the underground segment of the Ho Chi Minh Metro Line 1 project. To do so, the support pressure observed at the site is shown and compared to the minimum support pressure resulted from wedge stability model (Jancsecz and Steiner 1994), the maximum support pressure, and the recommended operation pressure. The maximum support pressure is resulted from the proposed model for both multi-layered soils and equivalent homogeneous soil, and from Broere's (Broere 2001) model. A particular attention is made for the section km 1 + 154.4 where the blow-out occurred to verify the current model and assess the role of multi-layered soils compared to equivalent homogeneous soil.

As a reminder, the thicknesses of soil layers of the cover from km 0 + 940 to km 1 + 2 40 and their geotechnical properties are shown in Table 1 and 2. The lateral earth pressure coefficient of three upper layers (F, Ac2, and As1) is assumed to be 0.55. Observed support pressure during the construction is obtained from the monitoring of TBM, which is the average of two injection pressures at two active pipes on TBM excavation face.

Figure 6 presents the variation of maximum, minimum, recommended operation, and site support pressures along the underground segment within two chainages km 0 + 940 and km 1 + 240. The result shows that the maximum support pressure resulted from the current model with multi-layered soils is about 7–17% higher than that with equivalent homogeneous soil for the data of Ho Chi Minh Metro Line 1 project. Moreover, the current model with equivalent homogeneous soil gives the maximum support pressure about 25–30% higher than that obtained by Broere (Broere 2001) model for equivalent homogeneous soil. Among these three maximum support pressures, the support pressure applying on the tunnelling face at the boring stage is close to the predicted model with multi-layered soils at the blowout location. The observed support pressure is slightly higher than the predicted pressure. This means that the proposed multi-layered model could derive a precise support pressure in this case.

4.2 Blow-out Section (km 1 + 154.4)

This section considers the data, including the soil condition and the site support pressure at the chainage km 1 + 154.4 within the underground segment of the Ho Chi Minh Metro Line 1, where the blow-out occurs. As a reminder, the site support pressure is recorded to be 335 kPa when the blow out took place. The comparison between site support pressure and maximum support pressure determined by Eqs. (9) and (11) to verify the accuracy of the proposed model. The comparison between the current models with multi-layered soils and equivalent homogeneous soil, as well as Broere (Broere 2001) model, is also made for the blow-out section to underscore the role of multi-layered model.

The soil condition at the chainage km 1 + 154.4 is zoomed in Fig. 7. The thickness of soil layers at this chainage is recalled in Table 3 and Fig. 8. Overall, the parameters of the multilayer model developed in Section 2 are recapitulated below:

- the nappe − 1.875 m from the ground surface;
- the tunnel diameter $D = 6.65$ m; the cover $C = 8.302$ m and the cover-to-diameter ratio $C/D = 1.25$;
- the liner thickness $d = 30$ cm and density $\gamma_T = 24$ kN/m^3;
- the soil multilayer geometry in Table 3 and Fig. 8;

Fig. 7 Soil condition at the chainage Km1 + 154.4 (blow-out occurrence)

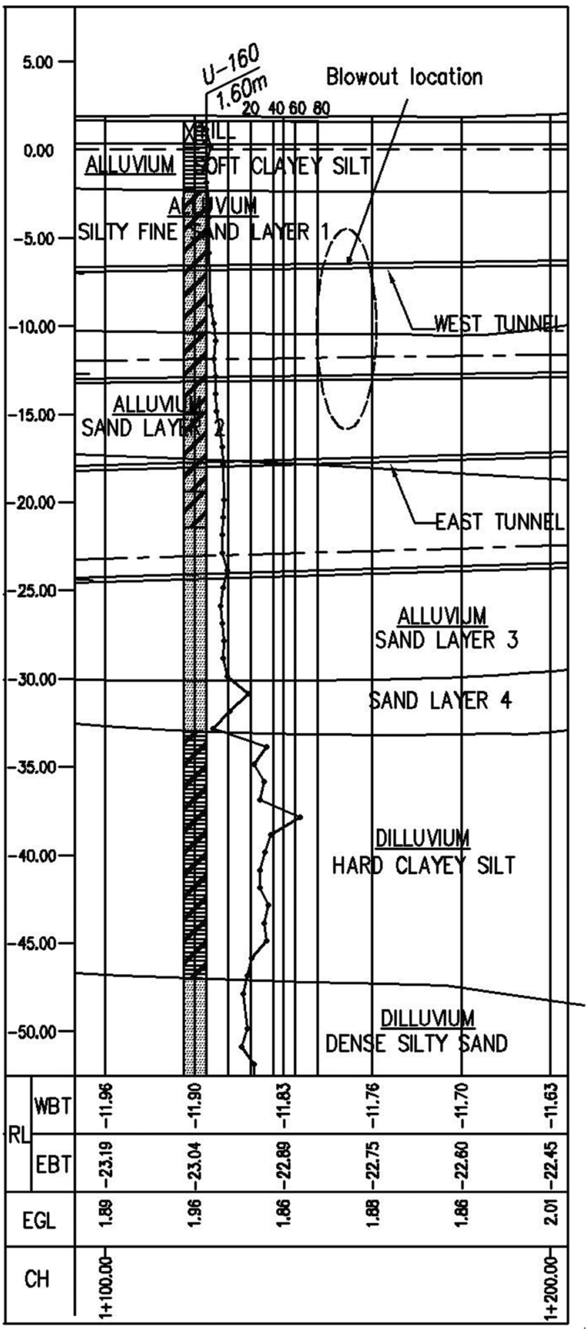


Table 3 Thickness of the soil layers at the chainage km 1 + 154.4

Layer	Description	Level		Thickness
		From	To	
1	Fill layer	1.875	0.275	1.6
2	Alluvium clay layer 2	0.275	−2.375	2.65
3	Alluvium sand layer 1	−2.375	−10.385	8.01
4	Alluvium sand layer 2	−10.385	−17.895	7.51

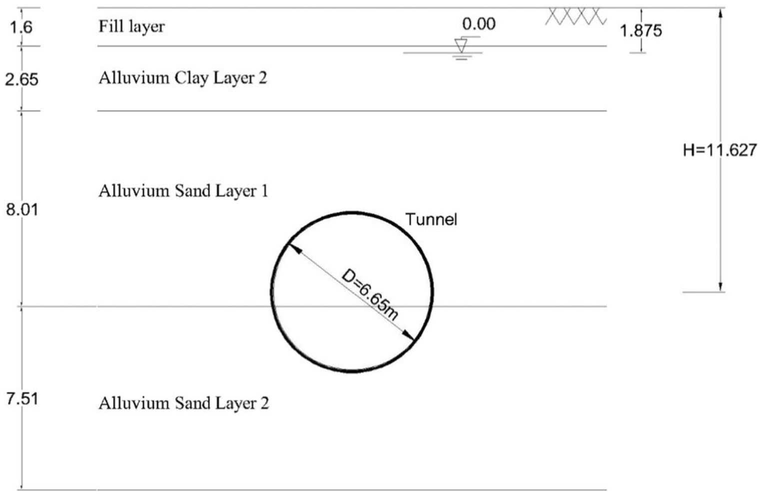


Fig. 8 Geometry model at the chainage Km1 + 154.4

- the support pressure gradient $\delta_p = 7$ kN/m^3.
- main geotechnical properties of soil layers in Table 1 (with $K = 0.55$ for three upper layers F, Ac2, As1).

From these input data, Fig. 9 shows the evolution of maximum support pressures at upper parts of tunnel resulted from the current model with both multi-layered soils and equivalent homogeneous soil, as well as Broere (Broere 2001) model versus the cover-to-diameter ratio (*C*/*D*). The maximum support pressure at the centre of the tunnel obtained by the multi-layered model and site support pressure at the chainage km1 + 154.4 (*C*/*D* = 1.25), where the blow-out occurred, are also added for the comparison. Obviously, the maximum support pressure increases when *C*/*D* increases due to the increase in the weight of the soil column above the tunnel. The weight of this soil column is equal to the maximum support pressure at the tunnel. However, the support pressure at the upper part is used as the criterion to evaluate passive failure. Correspondingly, the site support pressure 335 kPa is about 7.5% higher than the maximum support pressure (313 kPa)

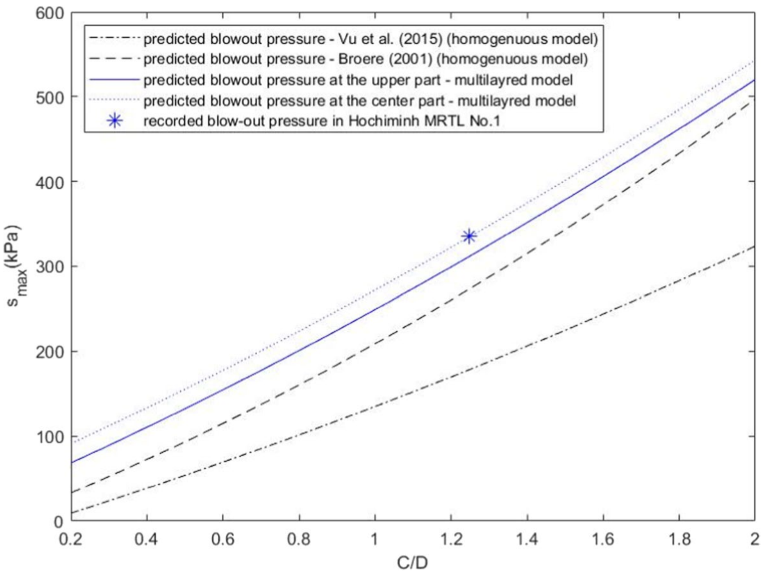


Fig. 9 Comparison between site support pressure and current blow-out model at the blow-out section (Km1 + 154.4)

at the tunnel top. Indeed, the site limit value should lower than 335 kPa and closer to the maximum support pressure at the upper part of the tunnel predicted by the proposed model (Eq. (9)). The comparison between the predicted value and data from the case study of Ho Chi Minh Metro Line 1 show a high accuracy of the proposed multi-layered model.

Anew, when plotting the maximum support pressure versus C/D, the current muti-layered model result is always higher than the current model with equivalent homogeneous soil (for Ho Chi Minh Metro Line 1 data). The difference between them is more pronounced when *C/D* small and decreases when *C/D* increases. Two curves meet when $C/D > 2$. Moreover, the maximum support pressure obtained by the current model with equivalent homogeneous soil is also higher than that resulted from Broere (Broere 2001) model. The difference between two later ones increases when *C/D* increases.

The comparison between maximum support pressures obtained by three blow-out models brings out the significant role of multi-layered soils, which has been rarely considered in previous studies.

5 Conclusion

Tunnelling in urban area must face challenges of face stability and risks of damages on surrounding infrastructure utilities and buildings. Instability issues including active failure (collapse) and passive failure (blowout) are the main consideration in estimating the support pressures applied in TBM excavation. Analytical, empirical, and numerical models have been proposed to assess the minimum and maximum support pressures corresponding to the active and passive failure occurrences. Data from laboratory tests or/and from real case study are valuable to check those models. In particular, the real case study observation is the most relevant data for the validation of the models.

This paper reported valuable data of the blow-out occurrence in the Ho Chi Minh Metro Line 1 project. The geological medium surrounding the tunnel is multi-layer soil. Thus, a muti-layered blow-out model is proposed to predict the maximum support pressure corresponding to the blow-out (passive failure) of the tunnelling face. This model is an extension of the previous work, which is derived by considering the equilibrium of the soil column above the tunnel in the plan passing through the tunnel face in the case of tunnelling in a multi-layered soil condition.

Applying the proposed multi-layered blow-out model to data of Ho Chi Minh Metro Line 1 project and comparing the predicted maximum support pressure to the site support pressure at the chainage of blow-out occurrence shows the accuracy of the proposed model. Moreover, the case of equivalent homogeneous soil is considered for the data multi-layered soils of Ho Chi Minh Metro Line 1 project. The comparison between support pressures resulted from the current models with multi-layered soil and equivalent homogeneous soil shows the multi-layered model gives results much closer to the blow-out site support pressure than equivalent homogeneous soil. Therefore, this is essential to consider the real multi-layered soil to assess the maximum support pressure when tunnelling the subway.

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Data Availability Not applicable.

Declarations

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Consent for Publication Not applicable.

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References

Al-Hallak, R.: Etude expérimentale et numérique du renforcement du front de taille par boulonnage dans les tunnels en terrains meubles PhD Thesis. Ecole Nationale des Ponts et Chaussées, Paris (1999)

Anagnostou, G.: The contribution of horizontal arching to tunnel face stability. Geotechnik **35**(1), 34–44 (2012)

Anagnostou, G., Kovári, K.: Face stability condition with earth pressure balanced shields. Tunn. Undergr. Space Technol. **11**(2), 165–173 (1994)

Atkinson, J.H., Potts, D.M.: Stability of a shallow circular tunnel in cohesionless soils. Géotechnique **27**(2), 203–215 (1977)

Balthaus, H.: Tunnel face stability in slurry shield tunnelling. In Congrès international de mécanique des sols et des travaux de fondations **12**, pp. 775-778 (1991)

Berthoz, N., Branque, D., Subrin, D., Wong, H., Humbert, E.: Face failure in homogeneous and stratified soft ground: theoretical and experimental approaches on 1g EPBS reduced scale model. Tunn. Undergr. Space Technol. **30**, 25–37 (2012)

Bezuijen, A., Brassinga, H.E: Blow-out pressures measured in a centrifuge model and in the field. Tunnelling: a decade of progress: GeoDelft 1995–2005, p. 143 (2006)

Bezuijen, A., Talmon, A.: Processes around a TBM. In: Proceedings of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground (IS-Shanghai 2008), pp. 10–12 (2008)

Broere, W.: Tunnel face stability and new CPT applications Ph.D. Thesis. The Netherlands: Delft University of Technology. Delft University Press, **193** pages (2001)

Broms, B.B., Bennermark, H.: Stability of clay in vertical openings. Am. Soc. Civ. Eng. – J.Soil Mech. Found. Div. **193**(1), 71–94 (1967)

Chambon, P., Corté, J.F.: Sallow tunnels in cohesionless soil: stability of tunnel face. J. Geotech. Eng. **120**(7), 1148–1165 (1994)

Chen, R.P., Tang, L.J., Ling, D.S., Chen, Y.M.: Face stability analysis of shallow shield tunnels in dry sandy ground using the discrete element method. Comput. Geotech. **38**, 187–195 (2011)

Chen, R.P., Li, J., Kong, L.G., Tang, L.J.: Experimental study on face instability of shield tunnel in sand. Tunn. Undergr. Space Technol. **33**, 12–21 (2013)

Dias, D., Janin, J P., Soubra, A.H., Kastner, R.: Three-dimensional face stability analysis of circular tunnels by numerical simulations. In: Proceedings of GeoCongress: Characterization,Monitoring, and Modeling of GeoSystems. Louisiana: ASCE, pp. 886–893 (2008)

Fang, Q., Zhang, D.L., Wong, L.N.Y.: Shallow tunnelling method (STM) for subway station construction in soft ground. Tunn. Undergr. Space Technol. **29**(3), 10–30 (2012)

Fang, Q., Zhang, D., Li, Q.Q., Wong, L.N.Y.: Effects of twin tunnels construction beneath existing shield-driven twin tunnels. Tunn. Undergr. Space Technol. **45**, 128–137 (2015)

Funatsu, T., Hoshino, T., Sawae, H., Shimizu, N.: Numerical analysis to better understand the mechanism of the effects of ground supports and reinforcement on the stability of tunnels using the distinct element method. Tunn. Undergr. Space Technol. **23**(5), 561–573 (2018)

Han, K.H., Zhang, C.P., Zhang, D.L.: Upper-bound solutions for the face stability of a shield tunnel in multilayered cohesive–frictional soils. Comput. Geotech. **79**, 1–9 (2016)

Horn, N.: Horizontal earth pressure on the vertical surfaces of the tunnel tubes. In National Conference of the Hungarian Civil Engineering Industry, Budapest, pp. 7–16 (1961)

Ibrahim, E., Soubra, A.H., Mollon, G., Raphael, W., Dias, D., Reda, A.: Three-dimensional face stability analysis of pressurized tunnels driven in a multilayered purely frictional medium. Tunn. Undergr. Space Technol. **49**, 18–34 (2015)

Jancsecz, S., Steiner, W.: Face support for a large mix-shield in heterogeneous ground conditions. In Tunnelling'94. Papers presented at seventh International Symposium Tunnelling'94, held 5–7 July 1994, London (1994)

Kanayasu, R., Yamamoto, Y., Kitahara, Y.: Stability of excavation face in earth pressure balanced shield . Proc. Int. Symposium on Underground Construction in Soft G round, New Delhi, 1994 (eds. K. Fuiita and O. Kusakabe). Balkema. p. 265 (1995)

Leca, E., Dormieux, L.: Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. Géotechnique **40**(4), 581–606 (1990)

Li, P., Chen, K., Wang, F., Li, Z.: An upper-bound analytical model of blow-out for a shallow tunnel in sand considering the partial failure within the face. Tunn. Undergr. Space Technol. **91**, 12 (2019)

Lu, X.L., Wang, H.R., Huang, M.S.: Upper bound solution for the face stability of shield tunnel below the water table. Math Probl Eng, **2014**, 727964, 11 pages (2014)

Mollon, G., Dias, D., Soubra, A.H.: Rotational failure mechanisms for the face stability analysis of tunnels driven by pressurized shields. Int. J. Numer. Anal. Meth. Geomech. **35**(12), 1363–1388 (2011)

Murayama, S., Endo, M., Hashiba, T., Yamamoto, K., Sasaki, H. (Eds.).: Geotechnical aspects for the excavating performance of the shield machines. In: The 21st Annual Lecture in Meeting of Japan Society of Civil Engineers, p [73-1]-[73-4], (1966) (in Japanese)

Qarmout, M., König, D., Gussmann, P., Thewes, M., Schanz, T.: Tunnel face stability analysis using kinematical element method. Tunn. Undergr. Space Technol. **85**, 354–367 (2019)

Senent, S., Jimenez, R.: A tunnel face failure mechanism for layered ground, considering the possibility of partial collapse. Tunn. Undergr. Space Technol. **47**, 182–192 (2015)

Senent, S., Mollon, G., Jimenez, R.: Stability of tunnel face in rock masses with the Hoek-Brown failure criterion. Int. J. Rock Mech. Min. Sci. **60**, 440–451 (2013)
Sloan, S.W.: Geotechnical stability analysis. Géotechnique **63**(7), 531–571 (2013)
Soubra, A.H., Dias, D., Emeriault, F., Kastner, R.: Three-dimensional face stability analysis of circular tunnels by a kinematical approach, pp. 894–901. Geotechnical Special Publication (2008)
Subrin, D., Wong, H.: Tunnel face stability in frictional material: a new 3D failure mechanism. C.R. Mecanique **330**, 513–519 (2002)
Tang, X.W., Liu, W., Albers, B., Savidis, S.: Upper bound analysis of tunnel face stability in layered soils. Acta Geotech. **9**(4), 661–671 (2014)
Vu, M.N., Broere, W., Bosch, J.W.: The impact of shallow cover on stability when tunnelling in soft soils. Tunn. Undergr. Space Technol. **50**, 507–515 (2015)
Vu, M.N.: Reducing the cover-to-diameter ratio for shallow tunnels in soft soils. PhD Thesis. Delft University of Technology (2016)
Zhang, Z.X., Hu, X.Y., Scott, K.D.: A discrete numerical approach for modeling face stability in slurry shield tunnelling in soft soils. Comput. Geotech. **38**, 94–104 (2011)
Zizka, Z., Thewes, M.: Recommendations for face support pressure calculations for shield tunnelling in soft ground. German Tunnelling Committee (ITA-AITES), Cologne, Germany (2016)

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Authors and Affiliations

Minh-Ngan Vu[1] · Minh-Ngoc Vu[2,3] · Duc-Tho Pham[1] · Tuan Nguyen-Sy[4,5]

✉ Minh-Ngoc Vu
vungocminh@dtu.edu.vn

Minh-Ngan Vu
vuminhngan@humg.edu.vn

Duc-Tho Pham
phamductho@humg.edu.vn

Tuan Nguyen-Sy
tuan.nguyensy@vlu.edu.vn

1 Department of Infrastructure Engineering, Hanoi University of Mining and Geology, Hanoi, Vietnam

2 Institute of Research and Development, Duy Tan University, Danang 55000, Vietnam

3 Faculty of Civil Engineering, Duy Tan University, Danang 55000, Vietnam

4 Laboratory for Computational Mechanics, Institute for Computational Science and Artificial Intelligence, Van Lang University, Ho Chi Minh City, Vietnam

5 Faculty of Mechanical - Electrical and Computer Engineering, School of Technology, Van Lang University, Ho Chi Minh City, Vietnam