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A compact blowout model for shallow tunnelling in soft soils

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ABSTRACT

Tunnelling in soft soil conditions, especially with a shallow overburden, faces the risk of face instability due to blowout. Although several blowout models have been proposed to estimate the blowout pressure, mostly based on limit analysis or limit equilibrium, there is a significant gap between the allowable blowout pressures predicted by these models and the values observed in case studies, laboratory experiments and numerical simulations. This paper proposes a compact blowout model, which is more compact compared to a model proposed by Balthaus (1991). This new blowout model is able to predict blowout pressures more closely to the value observed by centrifuge testing, reduced scale experiments and case studies, whilst staying conservative. Its application on the Hochiminh Metroline No. 1 project in this study resulted in a smaller support pressure in the boring stage to avoid the occurrence of a blowout.

1. Introduction

The increasing population density in cities leads to many social problems including traffic congestion and environmental pollution. As surface space becomes more limited and expensive, use of underground space becomes the preferred solution for new transportation modes. However, tunnel construction in urban areas faces many challenges, including face instability, generating large surface settlements and potential damage to surrounding buildings. These risks are especially present when tunnelling in soft soil conditions with shallow overburdens.

In order to limit the effect of tunnelling on surrounding structures and limit soil displacements, the support pressure applied at the excavation face and at the TBM tail has a vital role in tunnelling design. The support pressure is often derived from face stability models including as input factors tunnel depth and geotechnical parameters. The range of support pressures should allow a TBM operator to simultaneously satisfy the conditions to prevent collapse (active failure) for minimum support pressure and blow-out (passive failure) for maximum support pressures. Thus, face stability models have been discussed extensively in literature.

Several authors use the limit equilibrium method with a wedge shaped model based on a relatively simple rectangular soil silo and triangular wedge, including (Horn, 1961; Jancsecz and Steiner, 1994; Anagnostou and Kovári, 1994) and (Broere, 2001). Anagnostou and Kovári (1994) consider slurry infiltration and the effect on face stability during stand-still. Broere (2001) considers slurry infiltration and groundwater flow away from the face, whereas (Perazzelli et al., 2014) consider seepage towards the face. Other authors used limit analysis to derive upper and lower boundaries of support pressure, including (Leca and Dormieux, 1990) who used failure mechanisms based on conical blocks. Mollon et al. (2011, 2013), Soubra (2000a,b) and Subrin and Wong (2002b) developed a 3D logarithmic spiral model and extended this to multi-block models. This group of models trends towards a horn-shaped failure mechanism. Senent and Jimenez (2015) extended this approach for layered soils. Recently, kinematic models have also been proposed based on limit analysis by Soubra (2000a), Soubra et al. (2008), Mollon et al. (2011) and Qarmout et al. (2019).

A number of laboratory experiments and centrifuge tests have been carried out in order to investigate the mechanism of the face collapse. Chambon and Corte (1994) performed centrifuge tests to identify the stability of tunnel in dry sandy soil. Takano et al. (2006) carried out 1 g experiments to find the 3D shape of the failure in tunnelling. Kirsch (2010) worked on 1 g experiments to investigate the stability of shallow tunnels. Other authors tried to study the stability of the tunnelling process by applying numerical methods such as finite element methods (FEM) and discrete element method (DEM) including (Augarde et al., 2003; Funatsu et al., 2008; Chen et al., 2011; Lu et al., 2014; Zhang et al., 2015; Alagha and Chapman, 2019).

Mostly, these stability models focus on the face collapse mechanism at the lower limit of the support pressure, below which an active collapse takes place. Meanwhile, the maximum allowable pressure estimated from the passive failure condition has been studied less.

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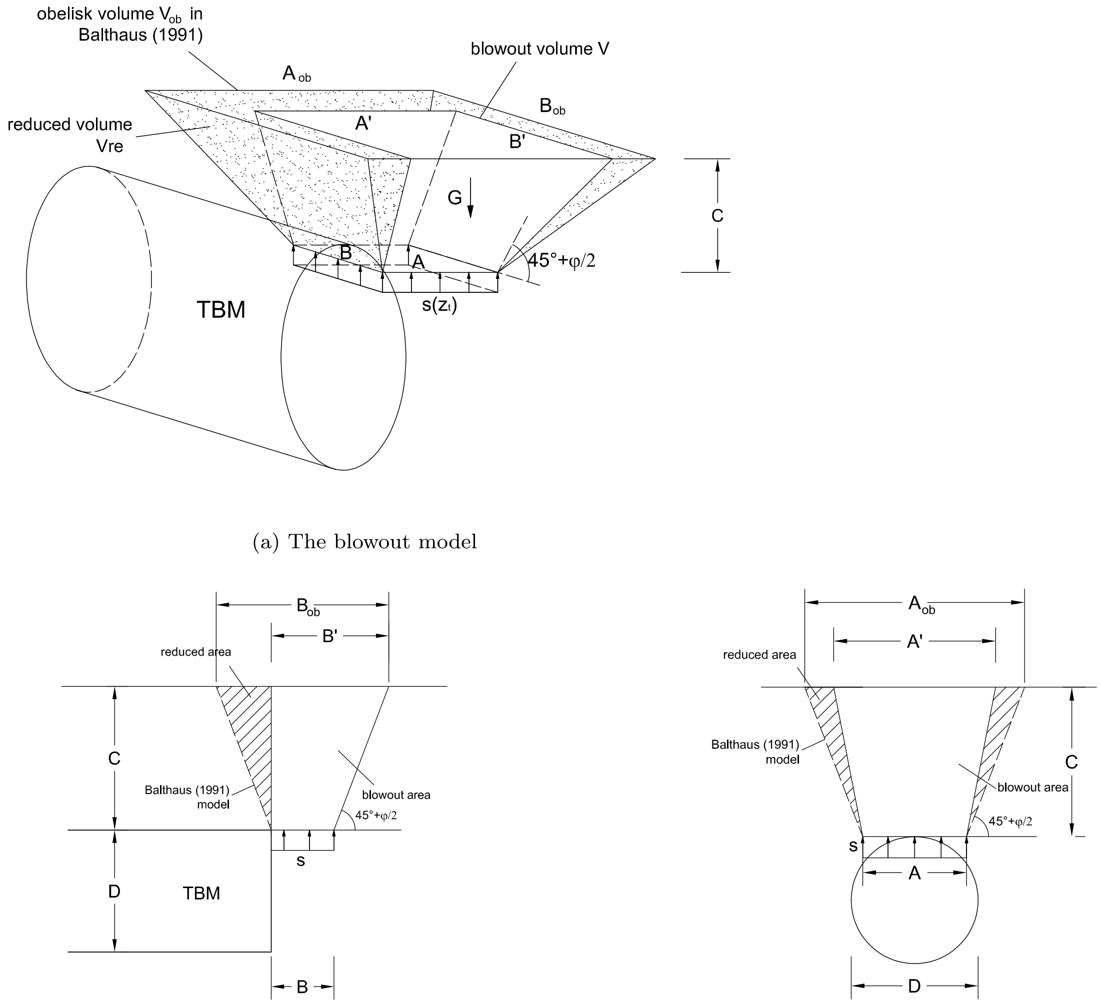


(a) The blowout model

(b) Side view

(c) Front view

Fig. 1. The compact blowout model proposed in this study.

Using limit analysis, the maximum support pressure is often estimated from the upper boundary condition. Leca and Dormieux (1990) show from a limit analysis for passive failure using elliptic cones emanating from the tunnel face, that this approach results in very high allowable maximum support pressure. Mollon et al. (2011) use a similar approach and find comparable high support pressures for the case of a solid passive failure mechanism, as do Li et al. (2020) and Liu et al. (2021). The theoretical maximum allowable support pressure, or blowout pressure, estimated with the upper boundary condition in limit analysis approaches is often higher than what has been observed in field conditions and in experiments.

Numerical and analytical simulation has been an important method to investigate the actual size and shape of the mechanism and the resulting scope of the blowout zone. Verruijt and Booker (1998) developed a 2D stability analysis including blowout. More recently, using numerical simulation combined with limit analysis models, Wong and Subrin (2006) and Mollon et al. (2010, 2013) introduced models including 3D failure mechanisms and derived a good agreement between limit analysis models and numerical models, but still these models tend to predict higher allowable pressures than actual field observations. Liu et al. (2022) studies the passive failure and presents one of the few model tests available in literature. When a passive failure of the excavation face occurs in their tests, there is a mud channel formed by the slurry splitting the stratum in front of the excavation face. When the cover-to-diameter ratio is relatively small, the channel extends to the ground surface and forms a channel whose size is basically the same as the diameter of the shield. This is a more compact area than predicted by conical limit analysis models. Similarly, Li et al. (2009) argue that for blow out or passive failure a partial face mechanism needs to be considered, as a full face mechanism overestimates the activated resistance of the soil body.

Balthaus (1991) proposed a 3D blow-out limit equilibrium model with the assumption that an obelisk shaped solid mass of soil above the tunnel face is displaced. This is consistent with a more limited failure shape than obtained from limit analysis, but still results in relatively high allowable support pressure.

On the other hand, there are simple models based on limit equilibrium which start from different mechanisms and are extremely conservative, such as models assuming simple fracturing, but these can often be used as a first (overly) conservative estimate. For example, Broere (2001) introduced a 2D model that assumes that only a rectangular soil body above the tunnel lining is pushed upwards by high support pressure, which is applicable for cases where fracturing is dominant, but often higher allowable face pressure can be applied before blow out actually occurs. Vu et al. (2015) introduced a model with simple overburden assumptions that seem to be most suitable for the estimation of the maximum support pressure at the tail of the shield in cases of shallow overburden.

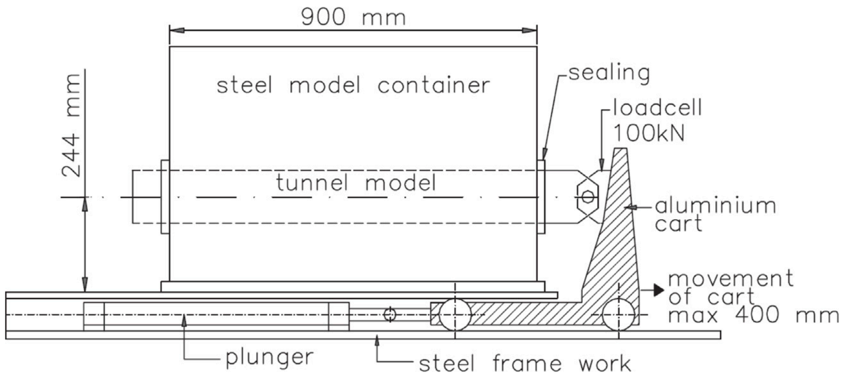


Fig. 2. Sketch of centrifuge tests in Bezuijen and Brassinga (2006).

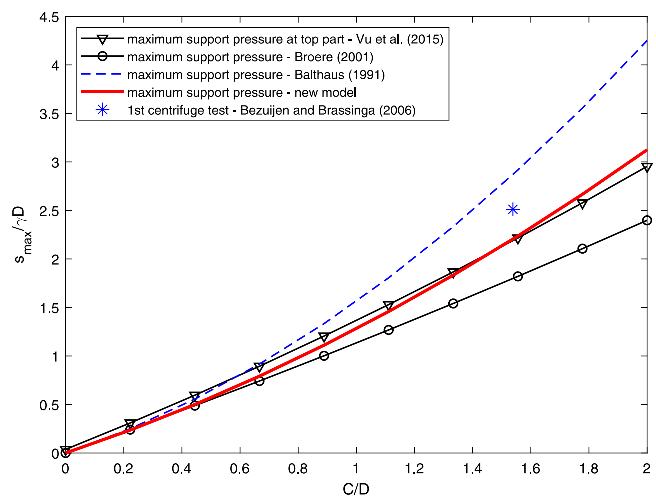
Only a limited number of experimental studies were carried out studying blowout incidents when tunnelling. Apart from the study by Liu et al. (2022) mentioned above, Bezuijen and Brassinga (2006) report on centrifuge tests performed by GeoDelft when studying the grouting process along the TBM. Berthoz et al. (2012) performed several tests to investigate both face collapse and blowout with an original reduced-scale model of an EPB machine in stratified soils. The blowout pressure recorded from these test shows a large difference with the calculated pressures derived from the limit analysis model of Subrin and Wong (2002a) and Wong and Subrin (2006). As far as case studies go, only few cases of blowout have been reported in literature for recent tunnelling projects, even though it is suggested that blowouts occur more frequently. A clear and documented blowout case occurred during Second Heinenoord Tunnel project, in the Netherlands, as reported by Bezuijen and Brassinga (2006) and Broere (2001).

Although many models have been proposed to predict the maximum support pressure that should be applied in tunnelling to avoid blowouts, large differences still exist between the predicted blowout pressure derived from recent models and the actual blowout pressures recorded in both experimental tests and case studies. Thus, there is a need for a model that results in predictions closer to observed blow out cases. This paper proposes a new blowout model, which can be characterized as a more compact and modified version of the limit equilibrium model proposed by Balthaus (1991).

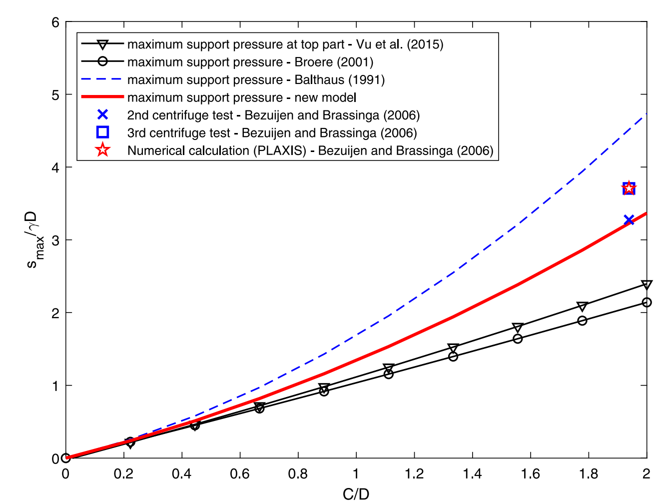
2. A new compact blow-out model

As indicated above, relatively simple models are still useful in practice as they offer the advantage of easy calculation and simple assumptions, compared to more advanced numerical models. The three-dimensional limit equilibrium model originally proposed by Balthaus (1991) is still applied as a useful calculation tool in tunnelling design, despite the relatively high maximum support pressures it predicts, which is due to the assumption of an obelisk shaped solid mass of soil being pushed upward by the blowout pressure. Berthoz et al. (2012) carried out a study on a 1 g reduced scale model of an earth pressure balanced (EPB) shield to investigate the failure mechanisms of face collapse and blowout. The analysis of these experiments shows that the resulting failure body shape is not an obelisk shaped soil column as indicated by Balthaus (1991). This might be the cause of too high maximum support pressures being estimated for blowout conditions in the case of Balthaus' model. The observations in Berthoz et al. (2012) are in good agreement with the illustrations and mechanism derived from numerical analysis in Soubra (2000b), Li et al. (2009), Zhang et al. (2015) that all show the scope of the uplift soil volume is limited to above and ahead the tunnelling face.

Based on these observations, the obelisk shaped soil mass assumed in Balthaus's model seems to be inappropriate and should be modified. This study proposes a new blow-out model with a more compact failure shape. The scope of the soil body pushed up during blowout is reduced as shown in Fig. 1, to the extent that only the soil mass ahead the tunnelling face is taken into account, and the impact of the two side wedges of the soil body is halved by assuming the failure plane to have a steeper angle here.



(a) with the first centrifuge test



(b) with the second and the third centrifuge tests

Fig. 3. Validations with three centrifuge tests in GeoDelft.

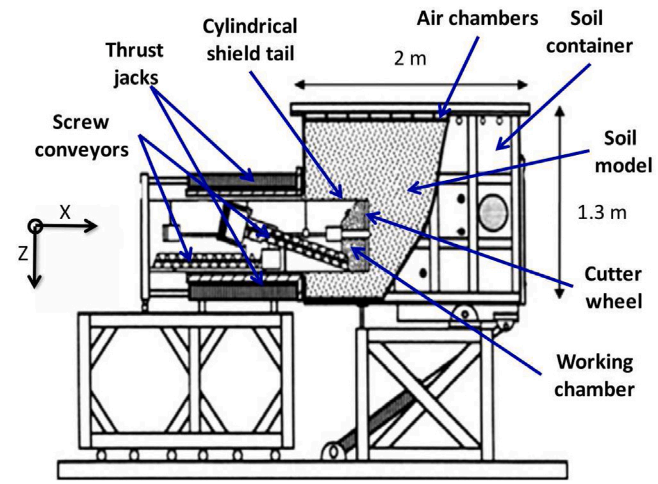


Fig. 4. Sectional diagram of EPBS model of ENTPE (Berthoz et al., 2012).

Table 1
Soil parameters used in centrifuge tests (Bezuijen and Brassinga, 2006).

Soil parameters	Speswhite clay	Sand med. dens.
γ_{wet} (kN/m^3)	17	19.6
c(kPa)	1	8.3
Friction angle (deg.)	23	37
Dilatancy angle (deg.)	–	9
Poisson's ratio(–)	0.45	0.3
E_{50} (MPa)	0.53	0.4
n(–)	–	0.394

Table 2
Parameters of soil used in reduced scale experiments in Berthoz et al. (2012).

Test	Weight unit (kN/m^3)	Cohesion (kPa)	Friction angle (^{o})	Elastic modulus (MPa)
MC1	12.20	3.2	36	6
MC3	13.20	2.5	36	10
MC5	13.05	1.5	36	10

The weight of the soil body G can be calculated as follows:

$$G = \gamma V \tag{1}$$

where V the volume of the blowout soil body, γ the soil weight unit.

The volume of the blowout soil body V can be estimated as follows:

$$V = V_{ob} - V_{re} \tag{2}$$

where V_{ob} the volume of the obelisk shaped soil mass indicated in Balthaus (1991) and V_{re} the volume by which is it reduced as proposed in this study.

The volume of the obelisk shaped soil mass V_{ob} is calculated as:

$$V_{ob} = \frac{C}{6}\left[AB + A_{ob}B_{ob} + (A + A_{ob})(B + B_{ob})\right] \tag{3}$$

where A the width of equivalent rectangle, B the effective support pressure length at the tunnel roof and A_{ob}, B_{ob} are dimensions of the obelisk shaped soil mass as shown in Fig. 1, and C the overburden depth of the tunnel.

The obelisk dimensions A_{ob} and B_{ob} can be calculated as:

$$A_{ob} = A + 2C\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{4}$$

$$B_{ob} = B + 2C\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{5}$$

where φ the soil friction angle.

Then, the volume of the obelisk shaped soil mass V_{ob} can be expressed:

$$V_{ob} = ABC + AC^2\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + BC^2\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{4}{3}C^3\cot^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{6}$$

The dimensions A' and B' of the proposed blowout soil mass as shown in Fig. 1 can be calculated as:

$$A' = A + C\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{7}$$

$$B' = B + C\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{8}$$

Then, the volume of the reduced soil mass V_{re} is identified as follows:

$$V_{re} = \frac{1}{2}AC^2\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{1}{2}BC^2\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + C^3\cot^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \tag{9}$$

From Eqs. (2), (6), and (9), the volume of the blowout soil body in Eq. (2) is shown to be equal to:

$$V = C\left[AB + \frac{1}{2}BC\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{1}{2}AC\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{1}{3}C^2\cot^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)\right] \tag{10}$$

and the weight of the soil body in Eq. (1) is:

$$G = \gamma C\left[AB + \frac{1}{2}BC\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{1}{2}AC\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) + \frac{1}{3}C^2\cot^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)\right] \tag{11}$$

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

$$P = ABs \tag{12}$$

where s is the support pressure. Although defined in Eq. (11) only as a function of the friction angle of the soil, the effective support pressure length at the tunnel roof B in reality depends on many factors including the ratio of depth to diameter of the tunnel C/D, the soil types encountered, the type of support fluids used and the penetration mechanism of the support fluid. This actual length of B is difficult to determine a priori as it depends on the interaction of the soil conditions and support fluid applied on the tunnelling face. Therefore, Balthaus (1991) suggested the weight of the soil body might be presented as a function of the effective support pressure length at the tunnel roof B as:

$$G = \alpha + \beta B \tag{13}$$

where the indexes α and β are given as:

$$\alpha = \gamma C^2\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)\left[\frac{A}{2} + \frac{C}{3}\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)\right] \tag{14}$$

$$\beta = \gamma C\left[A + \frac{1}{2}C\cot\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)\right] \tag{15}$$

The maximum allowable support pressure can then be derived from the equilibrium of the total support force P and the weight of soil body G. When including the safety factor, in line with (Balthaus, 1991), the maximum support pressure is estimated as:

$$\eta = \frac{G}{P} = \frac{\beta}{As} + \frac{\alpha}{ABs} \tag{16}$$

where η is the safety factor.

Since Eq. (16) involves the effective support pressure length at the tunnel roof B, which is difficult to determine, two other safety indexes have been proposed by Balthaus (1991) as:

$$\eta = \frac{\beta}{As} + \frac{\alpha}{ABs} > \eta_1 = \frac{\beta}{As} > \eta_2 = \frac{\gamma C}{s} \tag{17}$$

where η_1, η_2 are safety factors. The safety factor η_2 is often used for the case of very deep tunnels (Balthaus, 1991) and safety factor η_1 is used in case of shallow and moderate tunnels. These safety factors might be different based on the national construction code applied. Balthaus (1991) proposed a value of safety factor $\eta_1 = 1.1$ for blowout analysis. In this paper, the blowout in the case of shallow tunnelling in soft soils is studied, thus the blowout pressures can be estimated from Eq. (17) with the factor of safety η_1 as:

$$s = \frac{\beta}{A\eta_1} \tag{18}$$

3. Validations with experimental data

3.1. Validation with GeoDelft centrifuge tests

Validation of the new model is first performed against three centrifuge tests reported by Bezuijen and Brassinga (2006). These experiments were carried out in the GeoDelft geotechnical centrifuge when investigating the grouting process around the tunnel after a blowout incident during construction of the Second Heinenoord Tunnel in the Netherlands. In the centrifuge tests, a tube with an outer diameter of 130 mm and an inner diameter of 125 mm is used for representing the tunnel. The 25 mm tail void in this model was directly filled by a bentonite slurry as can be seen in Fig. 2.

The parameters of the soil used in the three centrifuge tests, as reported by Bezuijen and Brassinga (2006), are shown in Table 1. The tunnel tube in the first centrifuge experiment is covered by 200 mm saturated sand. The second and the third tests have similar conditions in that the tube is covered by sand and clay layers including a 77.5 mm thick sand layer directly of above the tunnel, a 170 mm clay layer above the sand layer and a 5 mm sand layer on top, with the water level at the top of the 5 mm sand layer.

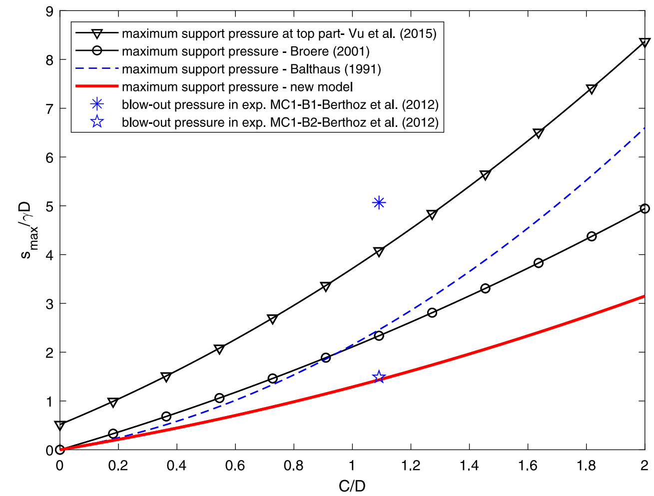
The first centrifuge test was performed at 150 g representing a large diameter tunnel of 18.75 m and an excess bentonite pressure was recorded at 620 kPa when the blow-out occurred. Meanwhile, the second and the third centrifuge tests were carried out at 40 g representing a tunnel with a smaller diameter of 5 m. The blowout pressures recorded in these tests are 190 kPa and 215 kPa, correspondingly. Bezuijen and Brassinga (2006) also used a numerical analysis by the Plaxis FEM programme with a hardening soil model in the case of the second and the third centrifuge tests. The blow-out pressure found in the numerical analysis was equal to the results from the third centrifuge case, as the total of the pore pressure and 2.5 times the vertical effective stress.

In order to compare the accuracy of the newly proposed model, the blowout pressure is compared to experimental data and overburden models proposed by Broere (2001) and Vu et al. (2015), the limit equilibrium model by Balthaus (1991), the limit analysis for passive failure as indicated in Leca and Dormieux (1990) and numerical analysis results.

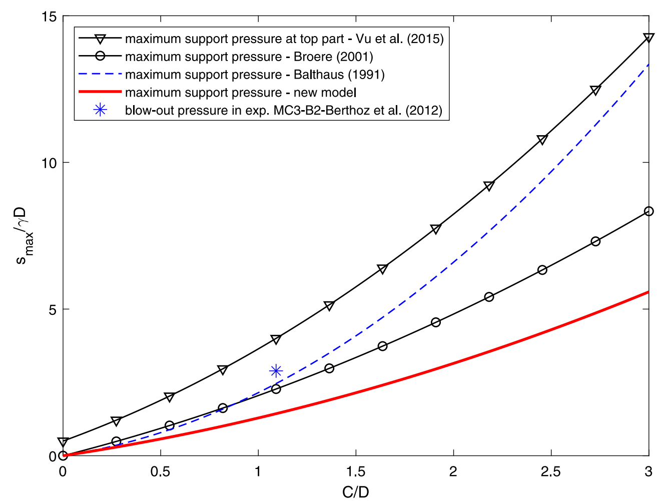
Fig. 3 shows the validation against blowout centrifuge tests from GeoDelft. The blowout pressures predicted by the limit analysis model by Leca and Dormieux (1990) are too high compared to the other results, so these pressures are off the chart and are not shown. Fig. 3(a) shows results for conditions identical to the first centrifuge test. This shows that the blow-out pressure predicted by the new model is close to the blow-out pressure recorded in the centrifuge test. The allowable pressure predicted by Vu et al. (2015) is rather similar. Meanwhile, the pressure predicted by Balthaus' model is about 1.5 times larger than the experimental values and the pressure predicted by Broere's model seems rather conservative.

For the second and the third centrifuge tests, as shown in Fig. 3(b), the blowout pressures predicted by the new model are nearly equal to the centrifuge test data and numerical analysis results. In these cases, the blowout pressures predicted by Broere (2001) and Vu et al. (2015) are still conservative, and the blowout pressures predicted by Balthaus (1991) are about two times higher than numerical analysis data and experimental data.

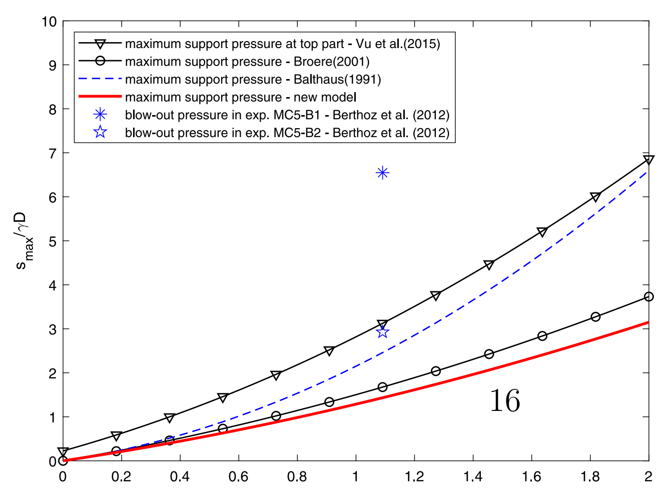
This validation shows the new model can predict blowout pressure more accurately than Balthaus' model and closer to the observed pressures in the tests.



(a) with MC1-B1 and MC1-B2 tests

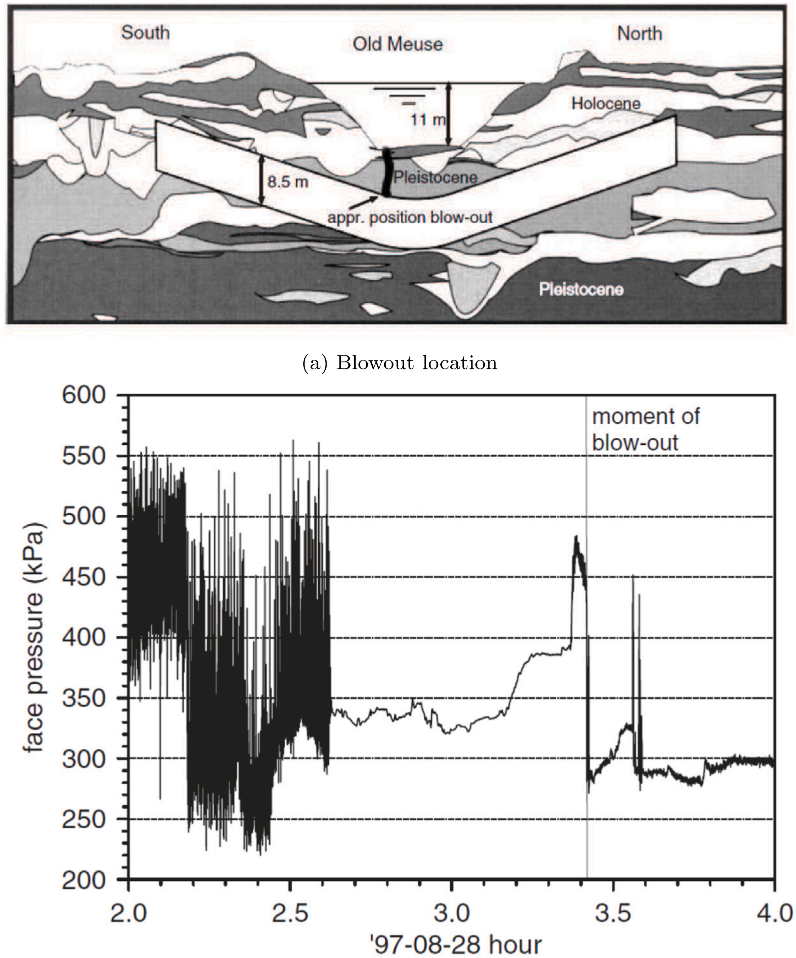


(b) with MC3-B2 test



(c) with MC5-B1 and MC5-B2 tests

Fig. 5. Validations with blowout cases in reduced scale experiments in Berthoz et al. (2012).



(a) Blowout location

(b) Recorded support pressure at the tunnelling face when the blowout occurred

Fig. 6. The blow-out case in the Second Heinenoord Tunnel project (Bezuijen and Brassinga, 2006).

Table 3
Blowout pressures recorded in reduced scale experiments in Berthoz et al. (2012) and estimated from blowout models.

Test	Experimental data Berthoz et al. (2012) (kPa)	Kinematic approach Subrin and Wong (2002a) (kPa)	Balthaus (1991) model (kPa)	Broere (2001) model (kPa)	Vu et al. (2015) model (kPa)	The new compact blowout model (kPa)
MC1-B1	34	612	15.50	15.68	27.37	9.61
MC1-B2	10	612	15.50	15.68	27.37	9.61
MC3-B2	21	612	17.85	16.51	29.04	10.40
MC5-B1	47	515	17.65	12.02	22.43	10.29
MC5-B2	21	515	17.65	12.02	22.43	10.29

3.2. Validation with the reduced scale experiments in Berthoz et al. (2012)

Berthoz et al. (2012) carried out a number of 1 g Earth Pressure Balance (EPB) shield reduced scale experiments to investigate the mechanism of face collapse and face blowout. The dimensions of the experimental container are $2 \times 1.3 \times 1.3$ m. The EPB equipment involves a 55 cm cutter head, a conical working chamber, a screw conveyor, a horizontal screw conveyor, a cylindrical shield tail, a cutter wheel and four thrust jacks as can be seen in Fig. 4. The soil used in the experiment was Houston $S28$ sand with soil parameters shown in Table 2. In this study, only the experiments leading to blowout are considered.

Table 3 compares predicted blowout pressures calculated by the kinematic approach as indicated in Berthoz et al. (2012), by simple models proposed by Broere (2001) and Vu et al. (2015), by the limit equilibrium model proposed by Balthaus (1991) and the new model. The blowout pressures predicted by the kinematic approach according to Subrin and Wong (2002b) are too large, as are the results from Leca and Dormieux (1990), so that they are off the graph in Fig. 5.

Fig. 5 shows the validation against the reduced scale experiments by Berthoz et al. (2012). In Fig. 5(a), in the case of test MC1-B1, all predicted blowout pressures are smaller than the pressure recorded in the test, whereas in the case of test MC1-B2, only the new model can predict the blowout pressure with good agreement, while other models predict larger values. Fig. 5(b) shows the validation with the test MC3-B2. It can be seen that the blowout pressure predicted by Balthaus (1991) is slightly lower than the observed value in the test. The simple models of Broere (2001) and Vu et al. (2015) show a relatively good agreement for this case, while the new model delivers a conservative prediction. Fig. 5(c) presents the validation with the tests MC5-B1 and MC5-B2. For MC5-B1, all models predict a lower blowout pressure than the observed pressure in the test. However, in the case of test MC5-B2, the (Balthaus, 1991) model shows a lower predicted blowout

Table 4
The geotechnical conditions of the blow-out location in Hochiminh Metroline No. 1 project.

Layer	Thickness (m)	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (°)
Fill - F	1.6	19.0	10	28
Alluvium soft clayed silt - Ac2	2.65	16.5	10	24
Alluvium sand - As1	8.01	20.5	0	31
Alluvium sand - As2	7.51	20.5	0	31
Alluvium sand - As3	15.21	20.5	0	31
Dilluvium hard clayed silt - Dc	14.07	21.0	22	0
Dilluvium dense silty sand - Ds	–	21.0	0	34

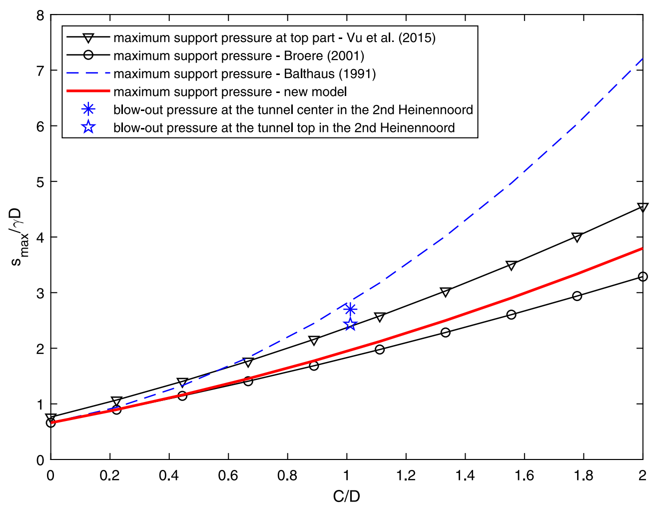


Fig. 7. The validation with the blowout case study at the Second Heneinoord Tunnel.

pressure than the value recorded in the test. The other models show a good agreement, deriving lower blowout pressures than the test value, with (Broere, 2001) model and the new model obtaining conservative results in comparison to Vu et al. (2015).

The validation against reduced scale experiments carried out by Berthoz et al. (2012) shows that the proposed model can predict blowout pressure more closely to the observed pressures in the experiments than previous models, even though these predictions are sometimes still conservative.

4. Validations with case studies

4.1. Validation with a case study of Second Heinenoord Tunnel

The next validation is carried out with the case study of the observed blowout in Second Heinenoord Tunnel project in the Netherlands (see Fig. 6). The tunnel was constructed under the Oude Maas river, in the period from 1996 to 1999. The tunnel has an outer diameter of 8.3 m. As shown in Fig. 6(a), the blowout occurred when the tunnel passes the Oude Maas river. At that location, the tunnel is covered by 4 m of Pleistocence sand with a friction angle of 36.5° whilst the total cover of 8.6 m consist of this sand layer and very soft Holocene layers and recent river deposits, as well as 11 m of water column above the soil (Bezuijen and Brassinga, 2006).

Fig. 6(b) shows the support pressure recorded at the tunnelling face when the blow-out occurred, which was 450 kPa at the centre of the tunnel or 405 kPa at the crown of the face.

Fig. 7 shows that the results from blowout models proposed by Balthaus (1991), Broere (2001), Vu et al. (2015) and the new model are all close to the case study observation. The blowout pressure derived (Leca and Dormieux, 1990) by limit analysis for passive failure is again too high and off the graph. It can be seen that Balthaus' model predicts a pressure slightly higher than the recorded site value. The simple overburden model proposed by Vu et al. (2015) gives the blowout pressure equal to the blowout pressure recorded at the top of the tunnel, but higher than the recorded value if safety factors are applied. Meanwhile, the blowout pressure derived from the new model is about 0.86 times the recorded blowout pressure at the top of the tunnel and slightly larger than the blowout pressure predicted by Broere (2001).

The validation against the Second Heneinoord Tunnel case shows that the new model results in a close but still conservative prediction of blowout pressure for tunnelling in soft soil, although all simple models result yield similar blowout pressures.

4.2. Validation with a case study of Hochiminh Metroline no. 1, Vietnam

A second case study used to validate the new model is Hochiminh Metroline No1 project, where a blowout occurred. Hochiminh Metroline No. 1 – a pilot metro project in Vietnam – launched in the year 2012 and completed the underground construction works in the year 2020. This metro line has a total length of 19.7 km from Ben Thanh station to Suoi Tien park with fourteen stations. The underground part of this metroline is about 2.6 km with two circular tunnels under a dense urban area with many important buildings such as Ba Son shipyard and the Saigon Municipal Opera House. Two circular tunnels were excavated at depths varying from 15 to 20 m in the case of The West Tunnel and varying from 20 to 30 m in the case of the East tunnel. The outer diameter of the tunnel is 6.65 m and the inner diameter is 6.05 m. The tunnel is located in Mekong deltaic soils with predominantly soft clay layers and silty sand layers.

During tunnel construction, a blowout occurred on the 23th April 2018, at chainage Km1 + 154.4 of the West Line as can be seen in Fig. 8. The tunnel here is located at a depth of −11.627 m corresponding to a C/D ratio of 1.25. The support pressure at the site when blowout occurred is recorded as 335 kPa. Detailed soil condition at this location are presented in Table 4.

The validation with the blowout case at Hochiminh Metroline No. 1 is shown in Fig. 9. As the C/D ratio is approximately 1.25, the difference between predicted blowout pressures for the various models can be seen clearer. The limited equilibrium model proposed by Balthaus (1991) derives a higher blowout pressure than the value observed. Meanwhile, the simple overburden model proposed by Vu et al. (2015) gives a blowout pressure close to the blowout pressure recorded at the top of the tunnel. Broere (2001) delivers the most conservative value, about 0.6 times of recorded site blowout pressure. It can be seen that the new model derives a more suitable, but still conservative, blowout pressure of about 0.7 times the recorded site blowout pressure. The blowout pressure predicted by Leca and Dormieux (1990) is again too high and off the graph.

5. Discussion and application

Validations with laboratory results and two case studies as indicated above show that the new blowout model can obtain an accurate and safe estimate of blowout support pressures. For the GeoDelft centrifuge tests, the (Balthaus, 1991) limit equilibrium model predicts a high blowout pressure in comparison to other models and experimental

Fig. 8. Blowout case in Hochiminh Metroline No. 1 at the chainage of Km1 + 154.4.

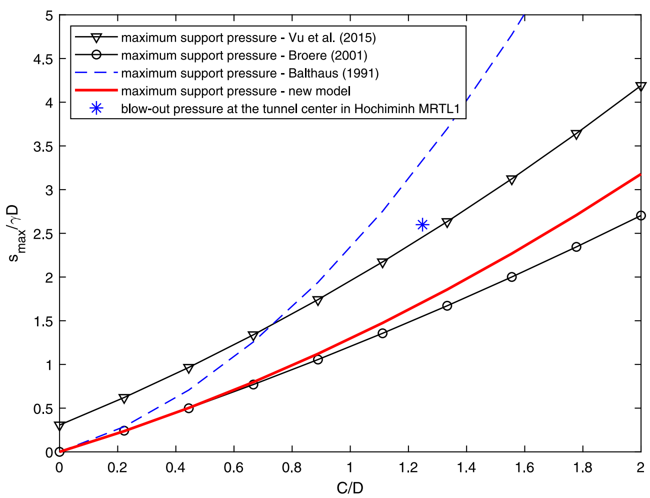


Fig. 9. The validation with the blowout case at the Hochiminh Metroline No. 1.

data. In the reduced scale experiments by Berthoz et al. (2012), the simple overburden model by Vu et al. (2015) shows a higher predicted blowout pressure than the test data, although this model performs better in the two case studies. In all validation cases, the simple overburden model by Broere (2001) shows to be very conservative. The new limit equilibrium model proposed in this study gives a reasonable predicted blowout in all validation cases. However, it is also clear that the number of laboratory tests focusing on blowout pressures, as well as the number of documented case studies, is rather small. A more extensive comparison with field cases would be preferred, also to see what uncertainties still remain in the models and the blowout mechanisms observed in practice.

Fig. 10 shows the support pressure range calculated for Hochiminh Metroline No. 1 chainages from Km1 + 080 to Km1 + 170. The maximum support pressure lines in the figure are derived from blowout models proposed by Balthaus (1991), Broere (2001), Vu et al. (2015) and the new model. The minimum support pressures are obtained from the wedge model proposed by Jancsecz and Steiner (1994). The operational support pressures recorded at the site for the boring stage and the erecting stage of the tunnelling process are also shown. The blowout occurrence as indicated above is also marked on the graph.

It can be seen that the support pressures applied in the boring stage fluctuated around the maximum support pressures predicted by the new model, which is lower than the maximum support pressures predicted by Balthaus and higher than the maximum support pressures predicted by Broere (2001) and Vu et al. (2015) models. Also, as can be seen in the figure, the recorded blowout pressure is not the highest support pressure applied in boring stage during the tunnelling process. This can be explained by the presence of other structures such as nearby buildings and road pavement that are not taken into account in the various models due to the complicated calculation it would result in. Thus, the actual blowout pressure including the effect of surrounding structures might be higher than the blowout pressure when tunnelling in "greenfield" conditions, but it is not recommended to include these effects when estimating safe support pressure ranges, especially when tunnelling in urban areas.

The support pressure during the erecting stage, when the TBM is at standstill for erecting tunnel segments inside the TBM, is less than the maximum support pressures predicted by the new model and higher than the values predicted by Broere (2001) and Vu et al. (2015), and much higher than the minimum support pressure. This indicates that the TBM operators tended to drive the TBM at a relatively high support pressure in order to reduce overexcavation of soil ahead of the face and improve face stability, but at an increased risk of blowout. This analysis shows that a smaller support pressure should be applied in the boring stage and less than 300 kPa should have been applied in the West tunnel in Hochiminh Metroline No. 1 project to avoid any risk of instability.

6. Conclusions

Tunnelling in soft soil conditions, especially in urban areas with deltaic soils, faces the challenge of face stability. Based on the observed scope and shape of the failure mechanism of blowout in experiments and numerical simulations, a new three-dimensional blowout is proposed in this study. Validations with three centrifuge tests and a number of reduced scaled experiment by Berthoz et al. (2012) show that the new model can predict a more precise and safe blowout pressure than blowout models by Balthaus (1991), Broere (2001) and Vu et al. (2015). Further validation in two blowout case studies of the Second Heinenoord Tunnel and the Hochiminh Metroline No. 1 also confirm the good agreement between the site observations and the blowout pressure predictions from the new model. The application of the new model to the Hochiminh Metroline No. 1 shows that smaller support pressures should have been applied in the boring stage when tunnelling with a low cover to diameter ratio C/D of less than 2, in order to avoid the risk of blowout. Overall the new model provides a more reliable but still conservative estimate of allowable support pressure. This study also highlights the limited number of reported blowout case studies where detailed data for analysis is available.

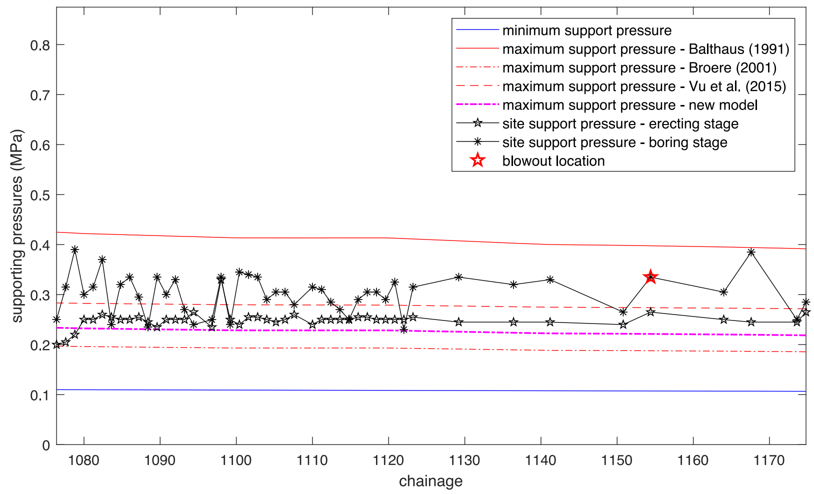


Fig. 10. Predicted and recorded support pressures in Hochiminh metroline No. 1 chainages from Km1 + 080 to Km1 + 170.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request

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