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New blow-out models for shallow tunnelling in soft soils

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ABSTRACT: In tunnelling design, blow-out is an upper boundary to estimate the maximum support pressure at the tunnelling face and at the tail. This paper proposes new models both for uniform support pressures and for linear support pressures which take into account the grouting pressure gradient. Validations with a case study and centrifuge tests in this study also show that the new models can predict the maximum support pressure with blow-out condition more accurately than recent models.

1. INTRODUCTION

The demand for underground infrastructure in urban areas is increasing due to economic developments and the growth of populations. Tunnel boring machines (TBMs) are widely used in the construction of underground infrastructure due to the limitation of the disturbance at surface level, settlements and damage to existing buildings during the construction. In an urban environment with soft overburden, particularly in soft Holocene layers, the tunnel is often designed well below the pile tip level in order to reduce effects on existing buildings, which are generally built on pile foundations. This leads to deep track tunnels and deep station boxes. When the tunnels would be located at more shallow levels, above the pile tip level, this largely eliminates the effect on the pile bearing capacity due to the reduction of the ground movement at the tip of the piles. This then also decrease the required depth of the station boxes and thus the construction costs. Other advantages are the low operational expenditure in the long-term and the shorter travelling time between the surface and the platforms.

One of the most important requirements of shallow tunnelling with TBMs in cities is to maintain existing buildings and infrastructure systems. When tunnelling in urban areas and especially in the historical areas, there might be a risk of damage to buildings due to the collapse of the tunnelling face and the subsequent surface settlement. Therefore, controlling the support pressures at the tunnelling face, around the TBM and at the tail is extremely necessary to avoid unexpected displacements in the surrounding ground and surface settlements.

In the case of tunnelling with a shallow cover, when the support pressures at the tunnelling face are too small, the tunnelling face will collapse and the soil will move towards the TBM. The minimum support pressure estimated from this condition was indicated in Anagnostou and Kovári (1994), Jancsecz and Steiner (1994), Broere(2001) and Vu et al. (2015). When the support pressure at the tunnelling face and/or the tail is too high, the soil column above is pushed upward. In the end, support medium will escape, the support pressures at the tunnelling face will decrease and the tunnelling face can collapse. The consequences of this are a risk of standstill or even damage of the

TBM, danger to people in case of maintenance, damage to buildings and transportation in case of the appearance of a hole and large soil displacements on the surface. This phenomenon is called a blow-out of the tunnel. In the case of shallow tunnelling, blow-out is a potential risk and should be carefully focused on. The occurrences of blow-out in the tunnelling process in Old Elbe Tunnel in 1909 and Second Heinenoord Tunnel in 1997 are the examples. To prevent this, the maximum allowable support pressure should be determined. Recent blow-out models in tunnelling design have been proposed by Balthaus (1991) and Broere (2001).

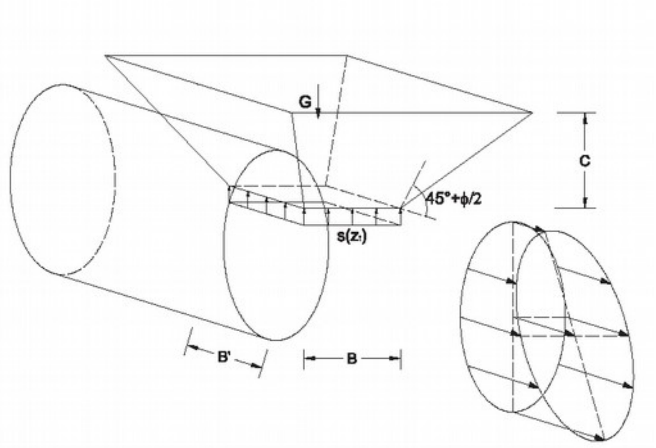


Figure 1. Calculation model of Balthaus for the safety against blow-out (Balthaus, 1991)

In the model proposed by Balthaus (1991), as can be seen in Figure 1, the up-lift soil body is modelled as a wedge shape, which is pushed upward when blow-out occurs. By balancing the wedge soil body weight G and the support force S, the maximum support pressure can be estimated. Safety indexes against the blow out were presented:

$$\eta = \frac{G}{S} > \eta_1 = \frac{\gamma C\left(B' + C\cot\left(45^0 + \frac{\varphi}{2}\right)\right)}{B' s(z_t)} > \eta_2 = \frac{\gamma C}{s(z_t)} \quad (1)$$

where C is the depth of the cover, φ is friction angel, γ is the volumetric weight of soil, and s is support pressure.

When the soil column is pushed upward by high support pressure, shear stress will appear between the soil column and surrounding ground. In a more accurate blow-out model proposed by Broere (2001), this shear stress should be taken into account. In the equilibrium condition (Figure 2), the support force is at least equal to the total of the weight of the above soil column and the shear forces along two vertical sides of the two dimensional rectangular soil body. Based on this, the maximum support pressure for the tunnelling face can be estimated as:

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

where c is cohesion and K_y is the coefficient of horizontal effective stress.

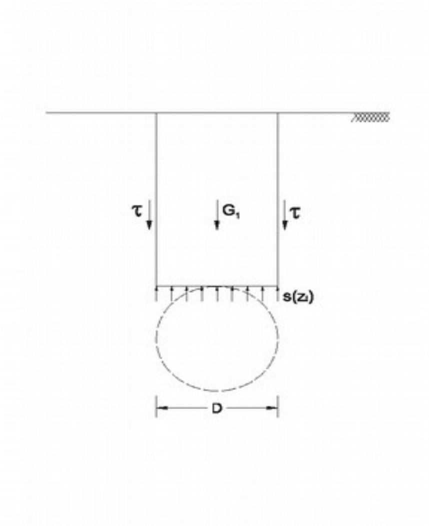


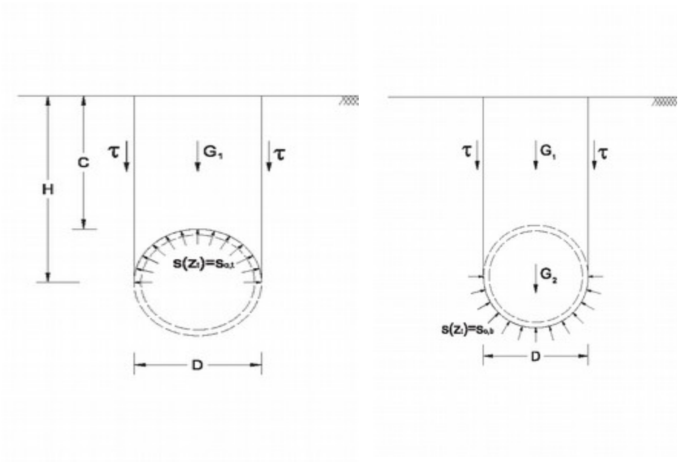
Figure 2. Blow-out model including friction at boundaries (Broere, 2001)

In this paper, new models for calculating the maximum support pressures with blow-out condition are proposed and validated with the case study of Second Heinenoord Tunnel and centrifuge experiments performed by GeoDelft, as indicated in Bezuijen et al. (2006). Comparisons between the maximum support pressures derived from the new models and recent models proposed by Balthaus (1991) and Broere(2001) are also carried out.

2. NEW BLOW-OUT MODELS

As Balthaus's model activates a large soil body above the tunnel, the calculated result is somewhat exaggerated. Meanwhile, Broere's model is probably too conservative. In practical tunnelling, the support pressure at the tunnelling face often changes along the vertical axis. In shallow tunnels, the difference between the required support pressures at the top and the bottom of the tunnel is large. This paper proposes new blow-out models in order to take this change into account with uniform support pressures and linear support pressures in which the effect of grouting flow is included.

In the model in Figure 3, the grouting pressure *s* is uniformly distributed on the perimeter of the tunnel section at the upper and lower part of the tunnel.



a) upper part b) lower part

Figure 3. Blow-out model with uniform support pressure

The maximum allowable grouting pressure is estimated in the upper part of the tunnel in which the soil body and the shear are taken into account, as follows:

$$s_{t,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) \quad (3)$$

where H=C+D/2 is the depth of the tunnel from the surface to the tunnel centre.

For the lower part of the tunnel, the tunnel weight is taken into account. The allowable grouting pressure which is shown in Figure 3, can be estimated as following equation:

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

where γ_T, d are the unit weight and the thickness of the tunnel lining.

The in-situ data from Talmon and Bezuijen (2005) shows that the grouting pressure gradient directly behind the TBM is nearly 20kPa/m at the start of grouting and at the end of the registration is about 7kPa/m in monitoring. This reduction of the grouting pressure is related to the consolidation and bleeding of the grout (Bezuijen and Talmon, 2005). The grout around the tunnel is assumed to behave as a Bingham liquid which has a viscosity and a yield stress. This liquid has a downward movement when more grout is injected through the upper injection points of the TBM. This downward flow creates a driving force larger than the yield stress. The pressure gradient, therefore, is smaller than the gradient estimated from the density. To be more accurate with the in-situ data, the gradient of the grouting movement in the tail void should be taken into account in blow-out analysis. According to Bezuijen and Talmon (2008), the maximum pressure gradient a is given by:

$$a = \frac{dP}{dz} = \rho_{gr}\, g - 2\frac{\tau_y}{d_{gr}} \quad (5)$$

where ρ_{gr} is the density of the grout, g is the acceleration gravity, τ_y is the shear strength of the grout and d_{gr} is the width of the tail void gap between the tunnel and the surrounding ground.

Figure 4 shows the blow-out model including a vertical pressure gradient a. The support pressure s in the upper part of the tunnel section in Figure 4a is given by:

$$s = s_{0,t} + aRcos\varphi \quad (6)$$

where $s_{0,t}$ is the support pressure at the top of the tunnelling face.

The maximum support pressure at the top of the tunnelling face is given by:

$$s_{0,t,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) - \frac{aD}{4} \quad (7)$$

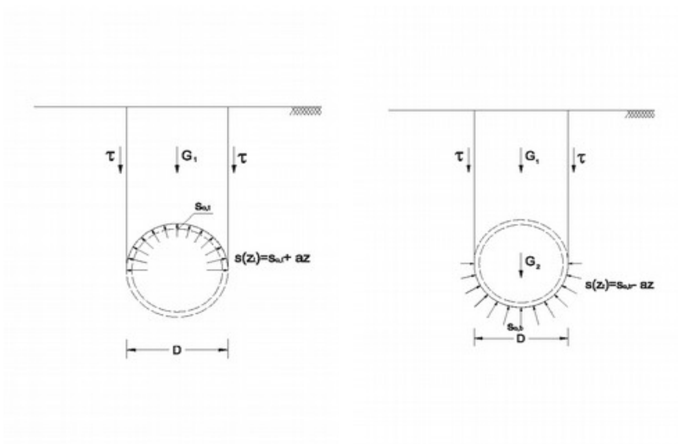
In the lower part as can be seen in Figure 4b, the support pressure in the upper part of the tunnelling face is:

$$s = s_{0,b} - aRcos\varphi \quad (8)$$

where $s_{0,b}$ is the support pressure at the bottom of the tunnelling face.

The maximum support pressure at the bottom of the tunnelling face is given by:

$$s_{0,b,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) + \gamma_T \pi d + \frac{aD}{4} \quad (9)$$



a) upper part b) lower part

Figure 4. Blow-out model with vertical support pressure gradient a

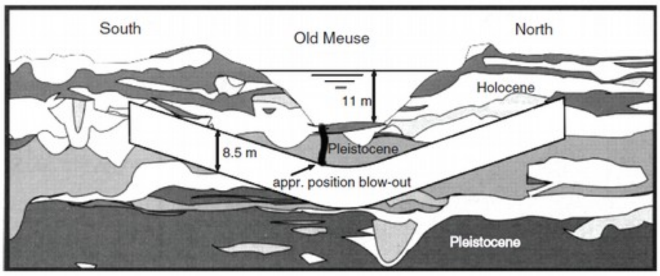
Based on Equations 7 and 9, the maximum required support pressures can be estimated in the case of linearly distributed support pressures. It is assumed that the unit weight of tunnel lining is γ_T=24kN/m^2 and the vertical gradient of the grout is a=7kPa/m.

3. VALIDATIONS FOR THE NEW BLOW-OUT MODELS

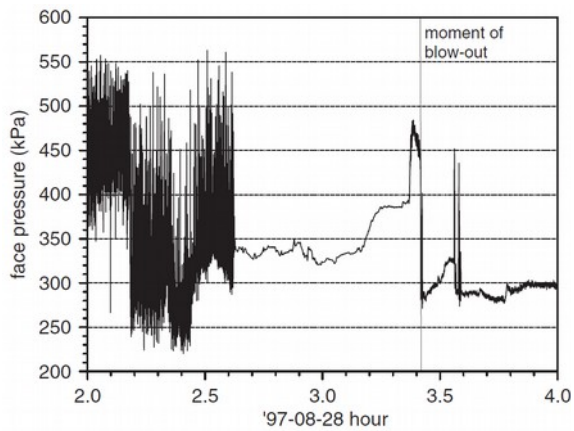
3.1 A blow-out case of Second Heinenoord Tunnel

In order to evaluate the new blow-out models, the blow-out case of the Second Heinenoord Tunnel in the Netherlands (Figure 5a) is used. A tunnel with an outer diameter of 8.3m was constructed below the Oude Maas river in the neighbourhood of Rotterdam between 1996 and 1999. At the blow-out position, the tunnel is covered by 4m of Pleistocence sand with a friction angle of 36.5^o. The cover depth of the tunnel is 8.6m in total including this sand layer and there was 11m of water above the soil (Bezuijen and Brassinga, 2006). Figure 5b shows the face pressures measured at the tunnel centre when the blow-out happened. During the blow-out, face pressure measured at the top of the tunnel was 405kPa and at the center of the tunnel was 450kPa.

Figure 6 shows the maximum support pressures calculated with the new blow-out model (Figure 4), Balthaus's model (Figure 1) and Broere's model (Figure 2) for the case of the blow-out position in the Second Heinenoord Tunnel. It can be seen that the maximum support pressures at the top and the bottom of the tunnel derived from the new blow-out models are in between the maximum support pressures calculated by Balthaus's model and Broere's model. Also, the measured face pressures at the top and the centre of the Second Heinenoord Tunnels at the blow-out position where C/D≈1 are plotted. It shows that the measured blow-out face pressures are in the range of calculated maximum support pressures with the new blow-out model for the lower and upper parts of the tunnel. The result also confirms the above statement that the maximum support pressure derived by Balthaus's model is somewhat exaggerated whereas this pressure estimate is too conservative when using Broere's model.



a) Scheme of the Second Heinenoord Tunnel and the blow-out position



b) Face support pressure measurement at the tunnel centre during blow-out

Figure 5. Blow-out at the Second Heneinoord Tunnel (Bezuijen and Brassinga, 2006)

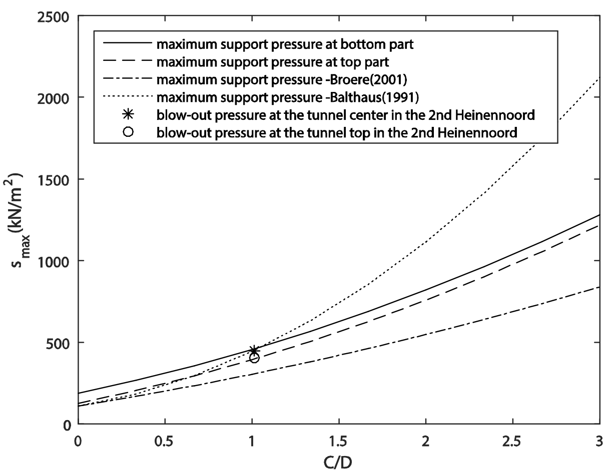


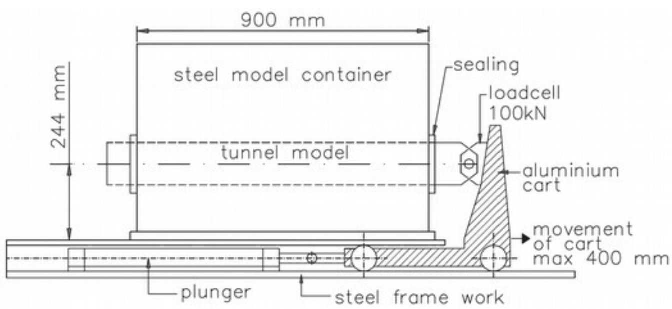
Figure 6. A comparison of maximum support pressures calculated from new blow-out models, Broere's model, Balthaus's model and in the Second Heinenoord Tunnel case

3.2 Centrifuge tests by GeoDelft

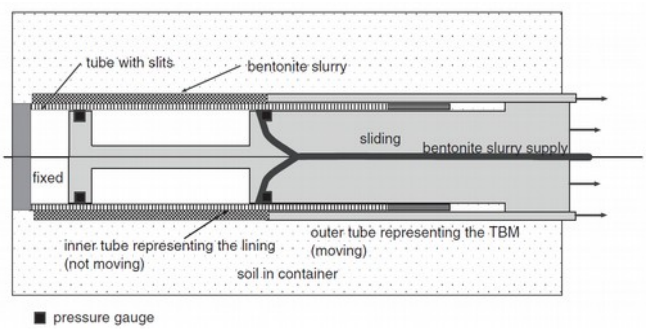
In order to validate with experimental data, centrifuge tests performed by GeoDelft and supervised by COB in order to investigate the grouting process (Brassinga and Bezuijen, 2002) are used to compare to the analysis results derived from the new models, Balthaus's model and Broere's model. These centrifuge tests were carried out with a tube representing a tunnel lining which has an outer diameter of 130mm and an inner diameter of 125mm as can be seen in Figure 7. The 25mm tail void in this model was directly filled by a bentonite slurry. The bentonite pressure was increased until the blow-out occurred in order to measure the maximum support pressures. The soil parameters used in these centrifuge tests are shown in Table 1. The maximum grouting pressures measured in these centrifuge tests are shown in Figure 8.

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

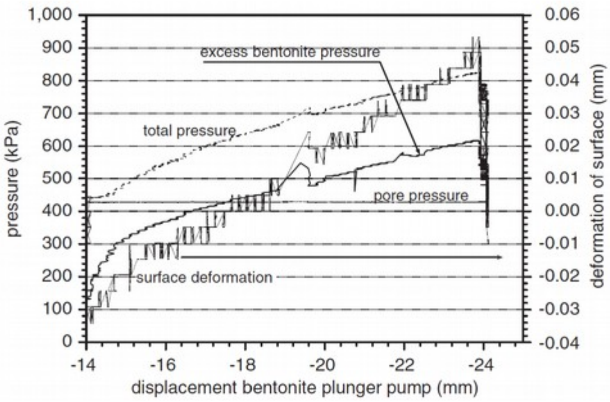


a) Side view

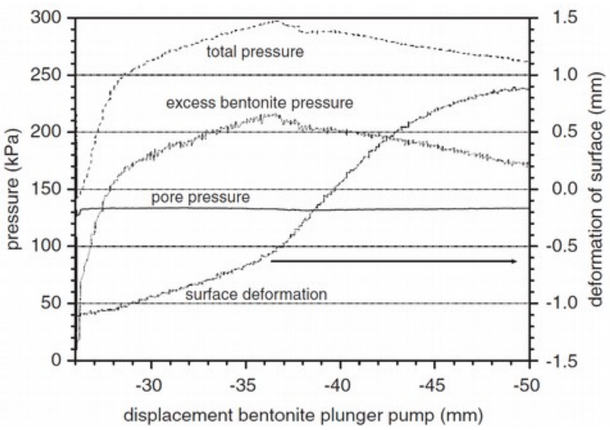


b) Sketch of the module made to simulate the grouting process

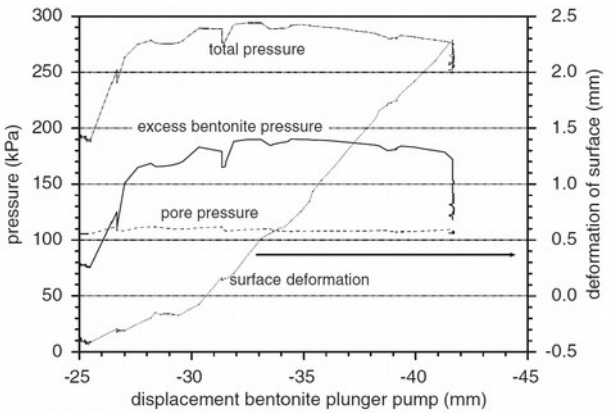
Figure 7. Sketch of centrifuge tests in Bezuijen and Brassinga (2006)



a) with the 1st centrifuge test



b) with the 2nd centrifuge test



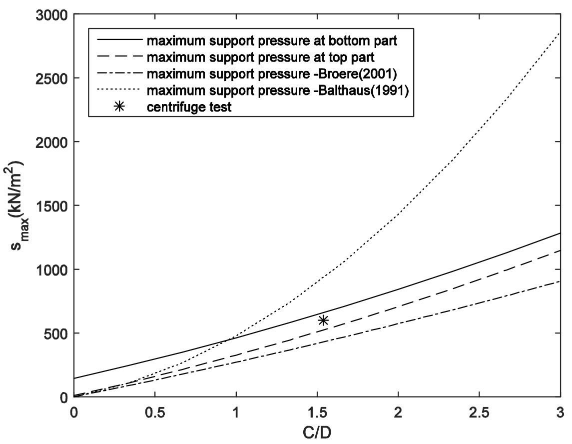
c) with the 3rd centrifuge test

Figure 8. Measured pressures in centrifuge tests in Bezuijen and Brassinga (2006)

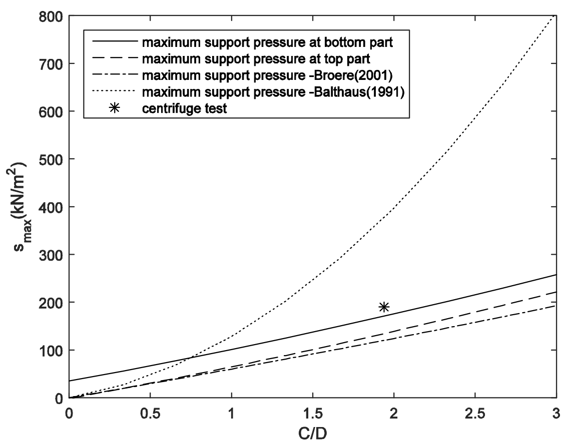
The first centrifuge experiment was carried out with a tunnel covered by sand and at 150g. This centrifuge test represented a large tunnel with a diameter of 18.75m, the tube was covered by 0.2m saturated sand with the parameters as shown in

Table 1. The maximum excess bentonite pressure was measured as 620kPa.

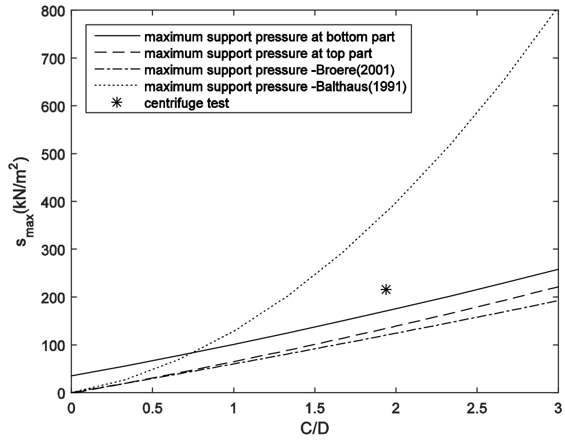
The second and third tests were carried out at 40g and represented a tunnel with diameter D=5m covered by sand and clay. There was a sand layer of 77.5mm above the tunnel. A clay layer of 170mm is above this sand layer and 5mm sand layer is on the top. The water level is at the top of the 5mm sand layer. The result in the second centrifuge test shows that failure was reached at a pressure of 190kPa. In the third centrifuge experiment with the same condition as the second test, the measured maximum excess bentonite pressure was of 215kPa.



a) with the 1st centrifuge test



b) with the 2nd centrifuge test



c) with the 3rd centrifuge test

Figure 9. In comparison with the centrifuge tests in Bezuijen and Brassinga (2006)

Figure 9 shows a comparison between the analytical results derived from the new models, Balthaus's model and Broere's model for these centrifuge test results. This figure also shows that the value of maximum support pressure derived by the new model is in between Balthaus's model and Broere's model with the soil conditions used in these centrifuge tests. It can be seen that the measured maximum support pressures in these centrifuge tests are approximately the maximum pressure calculated from the new models, while the maximum support pressure derived from Balthaus's model is larger and the results from Broere's model are smaller in comparison in these case. These results indicate that a more accurate result can be reached when applying the new model to maximum support pressure calculation.

4. CONCLUSIONS

Blow-out, which can occur in the case of shallow tunnelling, especially when tunnelling in soft soils, can lead to a risk of damage of the TBM, existing buildings and transportation on the surface. In tunnelling design, it is crucial to estimate the margin of support pressures applied at the tunnelling face as well as at the tail. In this calculation, the maximum support pressure is generally estimated via blow-out condition. The new blow-out models proposed in this paper not only calculate for the uniform support pressure but also for the linear support pressure, which takes into account the grouting pressure gradient. The validations for the new models have been carried out with a case study of Second Heinenoord Tunnel and experimental results of centrifuge tests performed by GeoDelft. The results show that the new model can predict the blow-out pressure better than the recent models proposed by Balthaus(1991) and Broere(2001).

5. REFERENCES

Anagnostou, G. and Kovári, K. (1994). The face stability of slurry-shield-driven tunnels. *Tunnelling and Underground Space Technology*, 9(2):165–174.

Balthaus, H. (1991). Tunnel face stability in slurry shield tunnelling. In *Proceeding 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, 13–18 August 1989 V2, P775–778*, volume 28, page A391. Pergamon.

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

Bezuijen, A. and Talmon, A. (2005a). Grout the foundation of a bored tunnel. *Tunnelling. A Decade of Progress. GeoDelft 1995-2005,* page 95.

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

Broere, W. (2001). *Tunnel Face Stability & New CPT Applications.* PhD thesis, Delft University of Technology.

Jancsecz, S. and Steiner,W. (1994). 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.*

Talmon, A. and Bezuijen, A. (2005). Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts. In *Geotechnical Aspects of Underground Construction in Soft Ground: Proceedings of the 5th International Symposium TC28.Amsterdam, the Netherlands, 15-17 June 2005,* page 319. Taylor & Francis US.

Vu, M. N., Broere, W., and Bosch, J. W. (2015). The impact of shallow cover on stability when tunnelling in soft soils. *Tunnelling and Underground Space Technology*, 50:507–515.