The impact of shallow cover on tunnelling in soft soil

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1. Introduction

Shield tunnelling is used widely in constructing underground infrastructure in cities due to the ability to limit settlements and damage to existing buildings. However, in an environment with soft overburden and buildings on pile foundations, the tunnel is often designed well below the pile tip level. There are two reasons for doing this: to reduce interaction between tunnelling process and piles, and to avoid having to drive through old abandoned piles that are still present below the streets. This results in deep station boxes.

When the tunnels would be located at a more shallow level above the pile tip level, this would largely eliminate the impact on pile bearing capacity as well as reduce the required depth of the station boxes and the construction cost. Moreover, other benefits are the low operational cost in the long-term and shorter travelling time from the surface to the platforms. This is possible only if there are no or very limited obstacles in the subsurface of the streets.

This paper looks into several aspects of shallow overburden tunnelling and seeks the limits on the cover-to-diameter ratio C/D when tunnelling in soft Holocene layers. Various geotechnical influences on the tunnel will be studied and the effect of low C/D ratio will be modelled. The analysis is carried out with a number of ideal soil profiles consisting of a single soil type with most important properties as defined in Table 1.

***Table 1** Soil parameters*

Soil type	$\gamma(\mathrm{kN}/m^3)$	$\varphi(°)$	K	c
Sand	17.9	35	0.4	2
Clay	16.5	33	0.5	7
Soft clay	15.5	20	0.65	5
Peat	10.5	20	0.65	5

2. Geotechnical analysis of tunnel stability

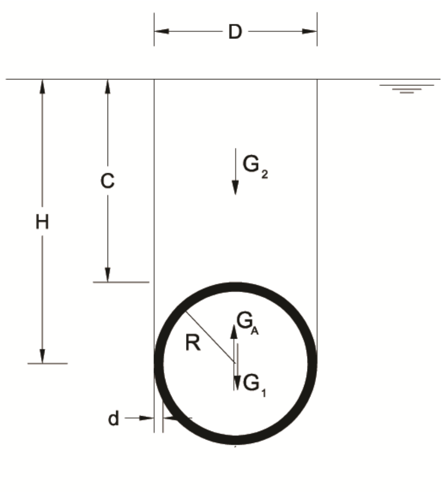


***Fig. 1** Uplift calculation*

2.1 Uplift

Analysing tunnel stability, one of the most important assessments is the uplift condition. Below the ground water level, the tunnel is loaded by the following vertical forces: the weight of the tunnel G_1 , the weight of overlaying soil layers G_2 and the uplift force G_A, as can be seen in Figure 1. The uplift force of the tunnel can be estimated according to the Archimedes's principle as:

$$G_A = \gamma_w \frac{\pi}{4} D^2 \tag{1}$$

where:
γ_w-the volumetric weight of water;
D- the diameter of the tunnel.
The weight of the tunnel lining follows from:

$$G_1 \approx \pi \gamma_T D d \tag{2}$$

where d is the thickness of tunnel segments, and γ_T is the weight unit of tunnel lining (concrete). The weight of the soil layers above the tunnel is given by:

$$G_2 \geq \ DH\gamma_g' - \frac{\pi}{8} D^2 \gamma_g' \tag{3}$$

where γ_g' is the volumetric weight of soil.

In the construction phase, it is assumed that friction between the lining and surrounding ground is not included in the vertical equilibrium (lower boundaries). If the uplift force G_A is smaller than the total of tunnel weight and the upper soil layers weight, there will be no risk of uplift in the tunnel.

$$G_A \leq G_1 + G_2 \tag{4}$$

Or

$$\gamma_w \frac{\pi}{4} D^2 \leq \pi \gamma_T D d + DH\gamma_g' - \frac{\pi}{8} D^2 \gamma_g' \tag{5}$$

Such that, the required depth of the tunnel H can be calculated from:

$$H \geq \frac{\pi \gamma_w D + \frac{\pi}{2}\gamma_g' D - 4\pi\gamma_T d}{4\gamma_g'} \tag{6}$$

From Figure 1, the depth of tunnel overburden is:

$$C = H - D/2 \tag{7}$$

From the Equation 6, the minimum required ratio of C/D can be calculated as:

$$\left(\frac{C}{D}\right)_{min} = \frac{\pi \gamma_w}{4\gamma_g'} - \frac{\pi d \gamma_T}{D \gamma_g'} - \frac{1}{2} + \frac{\pi}{8} \tag{8}$$

Assuming unit weight of tunnel lining $\gamma_T = 24\text{kN/m}^3$, the relation between the minimum required ratio of C/D and the unit weight of soil for the various the thickness-to-diameter ratios of the tunnel segment d/D is shown in Figure 2. For the case of $d/D = 1/10$, the cover C=0 and therefore the ratio $C/D_{min} = 0$ when $\gamma_g' = 2.92\text{kN/m}^3$. This means that there is no risk of uplift when the cross section of tunnel is designed with $d/D = 1/10$ or including ballast weight to a similar effect and the soil has a unit weight γ_g' more than 3kN/m^3.

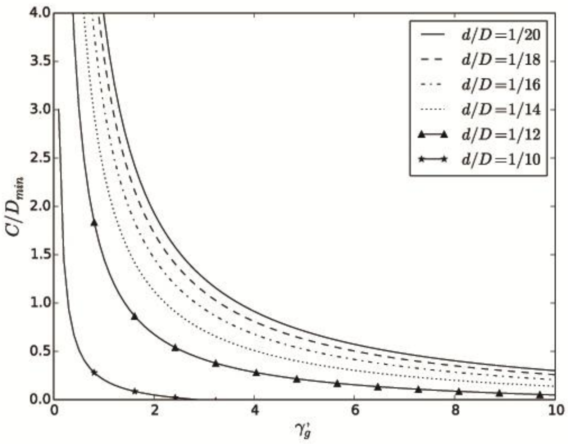


***Fig. 2** Relation between unit weight of soil γ_g' and the minimum required ratio C/D*

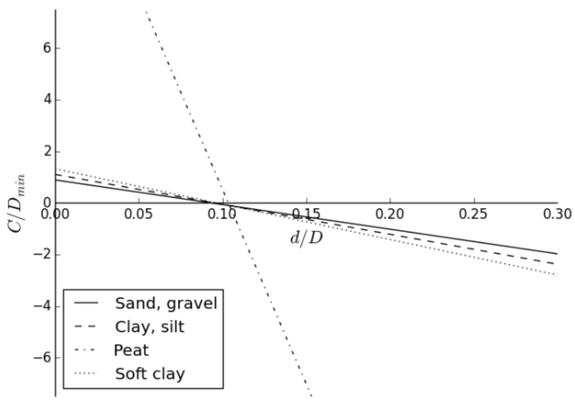


***Fig. 3** Relation between ratio of d/D and the minimum required ratio C/D*

***Table 2** Minimum required ratio d/D*

Soil type	γ(kN/m^3)	d/D
Sand	17.9	0.093
Clay	16.5	0.095
Soft clay	15.5	0.096
Peat	10.5	0.103

Based on Equation 8, Figure 3 indicates the required ratio d/D and the minimum required ratio C/D in various soil types. In these conditions, the minimum ratios d/D avoiding the uplift are identified as in Table 2 in the case of tunnel with $C/D = 0$. This shows that given enough ballast weight, the risk if uplift can be countered even in very soft soil conditions.

2.2 Wedge stability model

The support pressure at the tunnelling face must be higher than or at least equal to the total of water pressure and horizontal effective soil pressure to avoid collapse. The minimum required support pressure is estimated on the basic of this equilibrium condition. Over the years, many studies have been carried out to determine the minimum required support pressure. In 1961, Horn developed the first kinematic model including a soil wedge column based upon the silo theory to access the stability of the tunnelling face. This model consists of a wedge and overlying prismatic body.

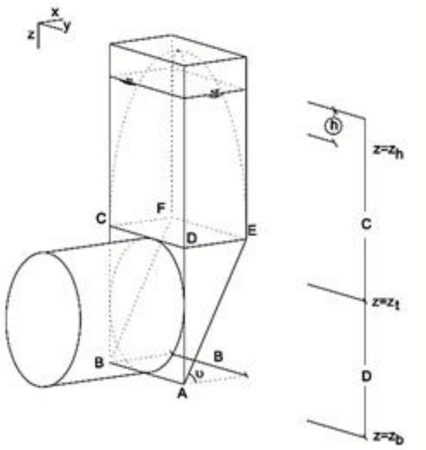


Fig. 4 *Wedge loaded by soil silo (Broere, 2001)*

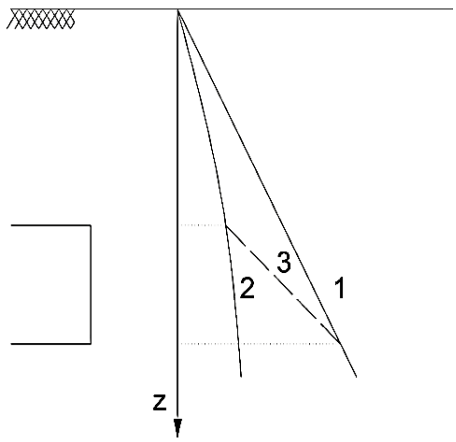


Fig. 5 *Three possible distribution of horizontal stress along the wedge sides (Broere, 2001)*

Anagnostou and Kovári (1994) developed Horn's wedge model using the silo theory of Janssen in drained condition. In this model, the vertical surcharge pressure σ'_v acting on the wedge can be reduced by the shear stresses on the sliding surface. From the computational analysis, the effects of the shear strength parameter of the ground, the permeability and the dynamic viscosity of the suspension were taken into account in stability assessments. It was concluded that the effectiveness of slurry support depends on the infiltration distance of suspension into the ground. However, these models only deal with the case of homogeneous soil.

Jancsecz and Steiner (1994) proposed a three-dimensional model that takes into account the effects of soil arching above the tunnel face. The three-dimensional effect is shown in this model by the three-dimensional earth pressure coefficient K_{A3} in calculation relating to the support pressure for the stability of the tunnelling face. In this study, the minimum required support pressure can be calculated as:

$$s_{min} = K_{A3}\sigma'_v + p \quad (9)$$

where p is the pore pressure.
The three dimensional earth pressure coefficient K_{A3} can be estimated as:

$$K_{A3} = \frac{sin\theta cos\theta - cos^2\theta tan\varphi - \frac{K\alpha}{1.5}cos\theta tan\varphi}{sin\theta cos\theta + sin^2\theta tan\varphi} \quad (10)$$

with: $K = \frac{1-sin\varphi+tan^2(45-\frac{\varphi}{2})}{2}$ and $\alpha = \frac{1+3\frac{C}{D}}{1+2\frac{C}{D}}$.
Broere (2001) presented a multilayered wedge model (Figure 4) for the case of heterogeneities or multilayered soil. From Terzaghi's model of a strip of soil loaded by stress $\sigma'_{v,a}$ from silo effect and effective weight γ', the effective vertical stress $\sigma'_{v,a}$ can be determined as:

$$\sigma'_{v,a} = \frac{a\gamma' - c'}{Ktan\varphi'}\left(1 - e^{-Ktan\varphi'\frac{z}{a}}\right) + q_0 e^{-Ktan\varphi'\frac{z}{a}} \quad (11)$$

where a is relaxation length, and q_0 is an arbitrary surface surcharge.
In a layered soil, similar calculations are applied for each layer. For i^{th} layer with $z = t^{(i)}$, the

distribution of effective vertical stress $\sigma_{v,a}^{\prime(i)}$ can be estimated as:

$$\sigma_{v,a}^{\prime(i)} = \frac{a\gamma^{\prime(i)} - c^{\prime(i)}}{K^{(i)}tan\varphi^{\prime(i)}}\left(1 - e^{-K^{(i)}tan\varphi^{\prime(i)}\frac{z}{a}}\right) + \sigma_{v,a}^{\prime(i-1)}(t_i)e^{-K^{(i)}tan\varphi^{\prime(i)}\frac{z}{a}} \quad (12)$$

In the case of surface loading $\mathrm{q} = 0\mathrm{kN/m^2}$, the effective horizontal stress can be calculated as:

$$\sigma_{h,a}^{\prime} = \sigma_{v,a}^{\prime}K = \frac{a\gamma^{\prime} - c^{\prime}}{tan\varphi^{\prime}}\left(1 - e^{-Ktan\varphi^{\prime}\frac{z}{a}}\right) \quad (13)$$

According to Broere (2001), three possible relaxation length a values can be estimated based on the applied wedge model:

- Without arching effect: $\mathrm{a} = \infty$;
- With two dimensional arching effect: $\mathrm{a} = \mathrm{R}$;
- With three dimensional arching: $\mathrm{a} = \mathrm{R}\frac{1}{1+\tan\theta}$,where θ is estimated in Jancsecz and Steiner(1994).

Three possible ways of vertical and horizontal stress distribution along the wedge body were also proposed by Broere (2001) (Figure 5). The line 1 and 2 show the horizontal stress distribution in the case of without and with arching effect. The dashed line 3 presents the assumed linear distribution with the stress including arching effect at the top of the tunnel and the stress without arching effect at the bottom of the tunnel.

By comparing the results of centrifuge test results and different models with and without arching, Broere (2001) indicated that the model with three dimensional arching effect with coefficient of neutral horizontal effective stress $\mathrm{K_0}$ is the best model to determine the minimum required support pressure for the case of shallow tunnel. This model is applied in this paper for calculating the minimum support pressure for the tunnel in varied soil parameters.

Figure 6 shows the relation between the effective horizontal pressures $\sigma_h^{\prime}$ and the ratio C/D based on Equation 13 for various tunnel diameters in varied soils. For the cases of $\sigma_h^{\prime} < 0$, it is often assumed that equal $\sigma_h^{\prime} = 0kN/m^2$ in practical purpose.

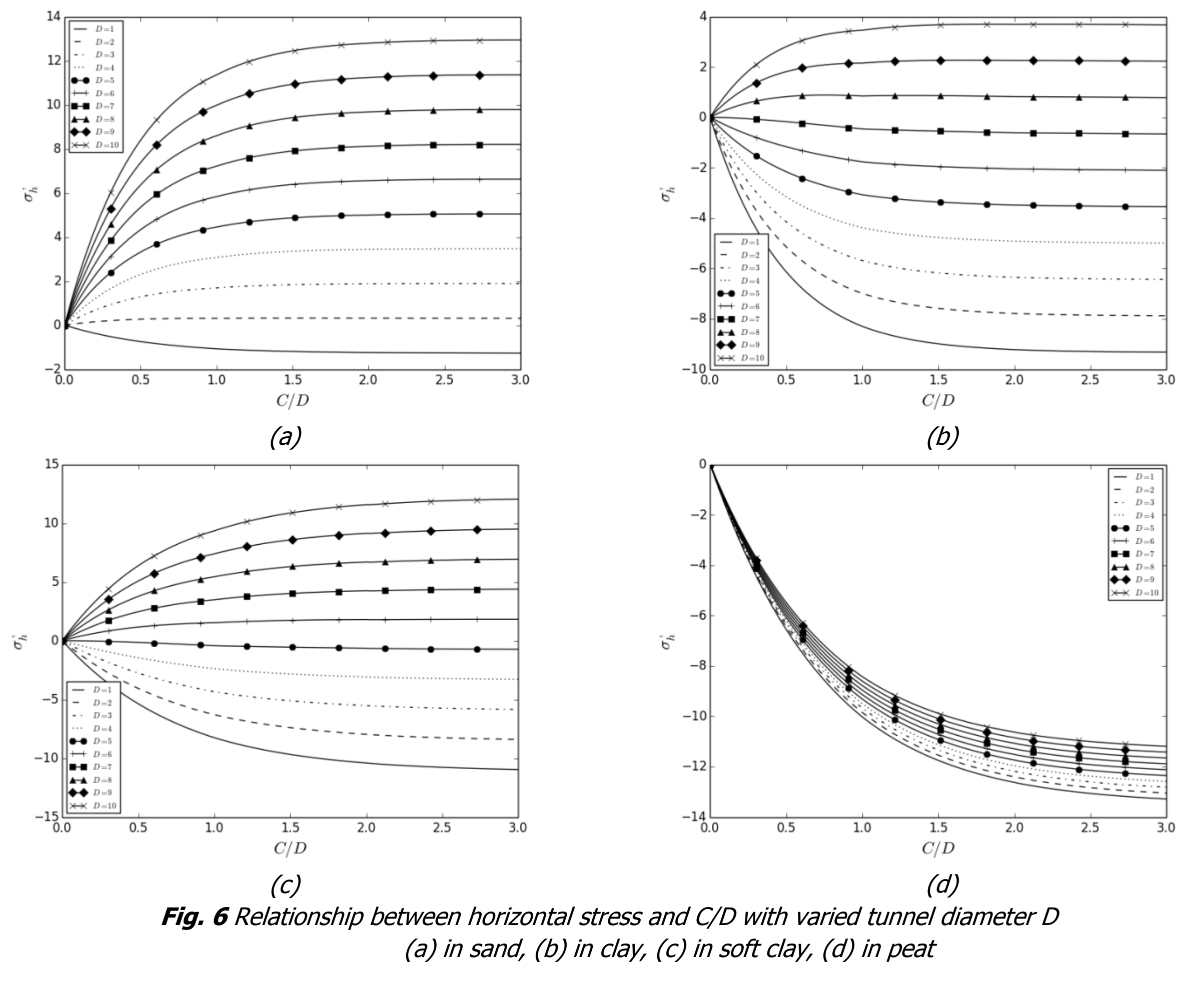


Fig. 6 *Relationship between horizontal stress and C/D with varied tunnel diameter D (a) in sand, (b) in clay, (c) in soft clay, (d) in peat*

2.3 Blow-out

When the support pressure at the tunnelling face is too high, the soil column above is pushed upward. In the end, support medium will escape, the support pressures at the face will decrease and the tunnelling face can collapse. The consequences of this are a danger of standstill or even damage of the TBM, danger to people in case of maintenance, 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. To avoid this, maximum allowable support pressure should be determined. In the simple case, when the friction between the failing soil body and the surrounding ground is not taken into account, the maximum pressure is estimated as:

$$s_{max} = \sigma_v \tag{14}$$

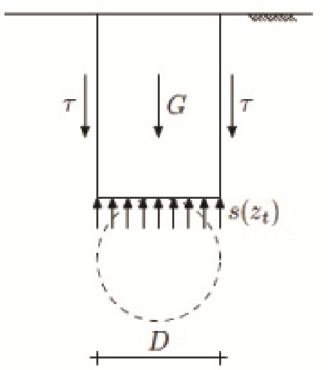


Fig. 7 *Blow-out model including friction at boundaries (Broere, 2001)*

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, this shear stress should be taken into account. In the equilibrium condition (Figure 7), 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 tunnel face can be estimated as:

$$s_{max} = C\left[\gamma + \frac{2c + CK_y\gamma' tan\varphi}{D}\right] \tag{15}$$

In the model proposed by Balthaus (1991), 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 and the support force, the maximum support pressure can be estimated. Safety indexes against the blow out were presented:

$$\eta = \frac{G}{S} > \eta_1 = \frac{6(B' + Ccot(45^o + \varphi/2))}{B's(z_t)} > \eta_2 = \frac{\gamma C}{s(z_t)} \tag{16}$$

Because 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 tunnel 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 report 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.

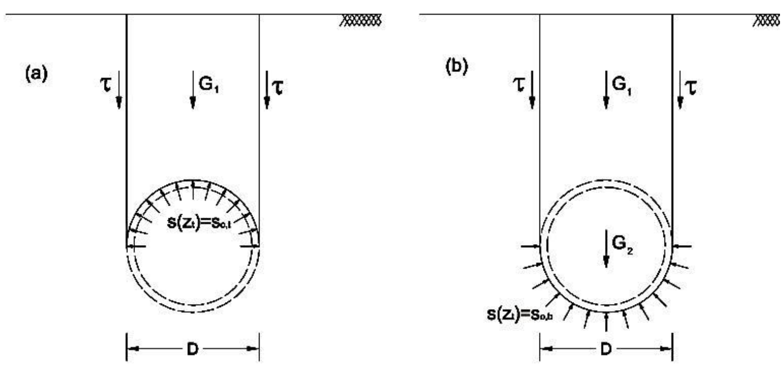


Fig. 8 *Blow-out model with uniform support pressures*
(a) at the upper part of the tunnel (b) at the lower part of the tunnel

In the model in the Figure 8, the grouting pressure s is uniformly distributed on the perimeter of the tunnel section at the upper and lower part of the tunnel. 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:

$$s_{t,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) \tag{17}$$

with $\mathrm{H} = \mathrm{C} + \frac{\mathrm{D}}{2}$. It can be written as:

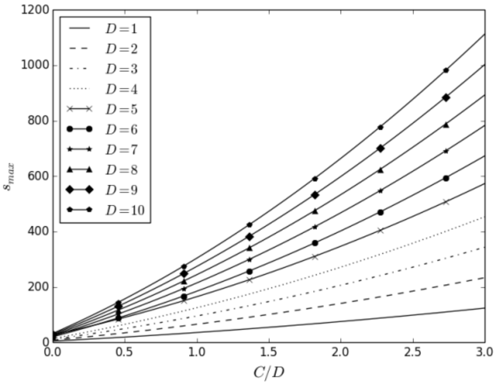
$$s_{t,max} = \left(\frac{C}{D} + \frac{1}{2}\right)^2 2DK_y\gamma' tan\varphi + \left(\frac{C}{D} + \frac{1}{2}\right)(\gamma D + 2c) - \frac{\pi}{8}\gamma D \tag{18}$$

For the lower part of the tunnel, the tunnel weight is taken into account. The allowable grouting pressure which is shown in Figure 8, can be estimated as following equation:

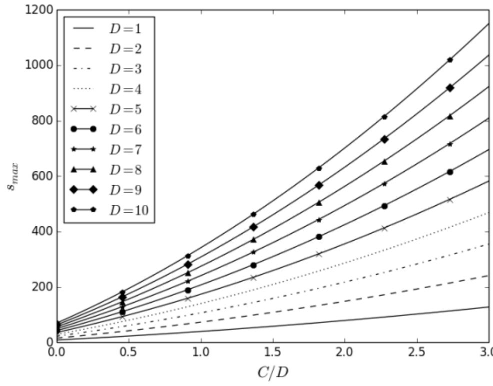
$$s_{b,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) + \gamma_T\pi d \tag{19}$$

Or

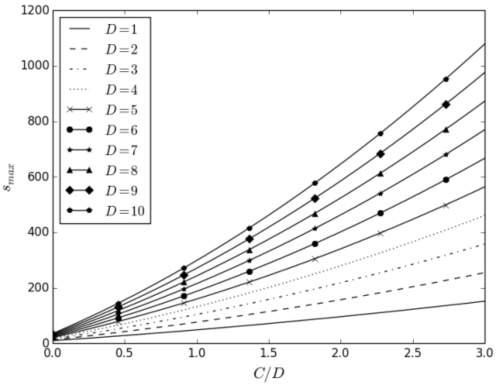
$$s_{b,max} = \left(\frac{C}{D} + \frac{1}{2}\right)^2 2DK_y\gamma' tan\varphi + \left(\frac{C}{D} + \frac{1}{2}\right)(\gamma D + 2c) + \gamma_T\pi d - \frac{\pi}{8}\gamma D \tag{20}$$



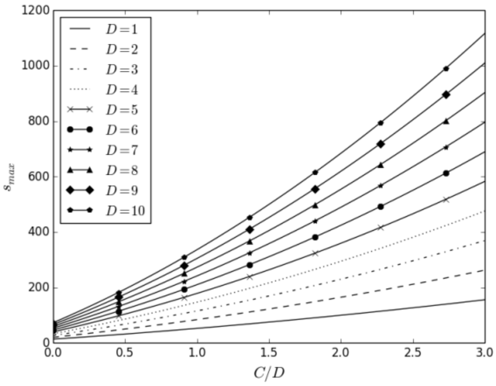
(a1)



(a2)



(b1)



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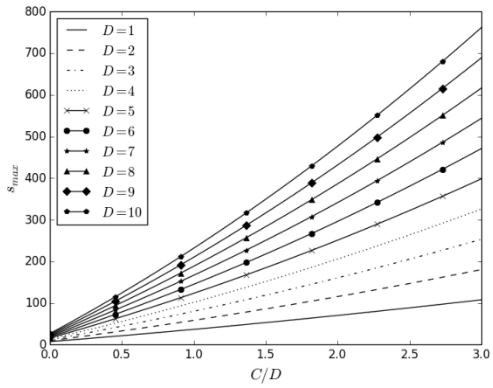
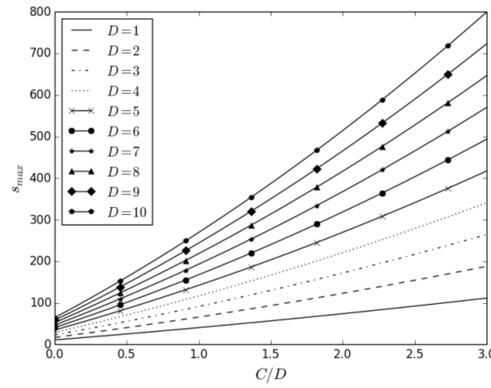
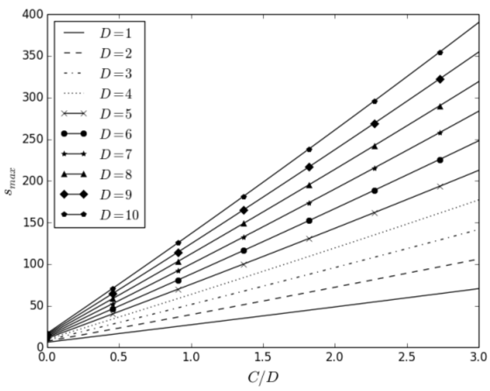


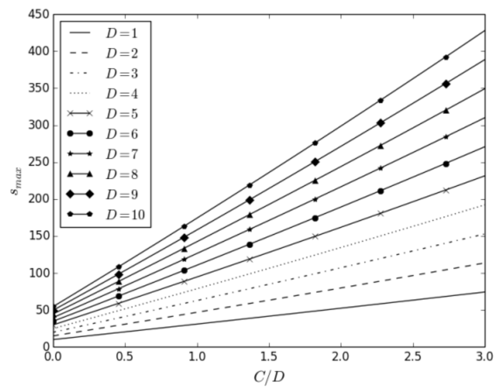
(c1)



(c2)



(d1)



(d2)

Fig. 9 *Maximum allowable pressures at upper part (1) and lower part (2) of the tunnel with uniform support pressures (a) in sand, (b) in clay, (c) in soft clay, (d) in peat*

Figures 9 presents the relation between the maximum required support pressure $s_{t,max}$ and $s_{b,max}$ at upper and lower part of the tunnel and the C/D ratio in the range of tunnel diameter D from 1 meter to 10 meters. This figure shows that the higher the ratio of C/D is, the larger the maximum support pressures are.

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 caused by the volume loss which related to the consolidation and bleeding of the grout (Bezuijen and Talmon, 2006). The grout around the tunnel is assumed 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:

$$a = \frac{dP}{dz} = \rho_{gr} g - 2\frac{\tau_\gamma}{d_{gr}} \quad (21)$$

where:
ρ_{gr}- is the density if the grout;
g- the acceleration gravity;
τ_γ - the shear strength of the grout;
d_{gr}- the width of the tail void gap between the tunnel and the surrounding ground.

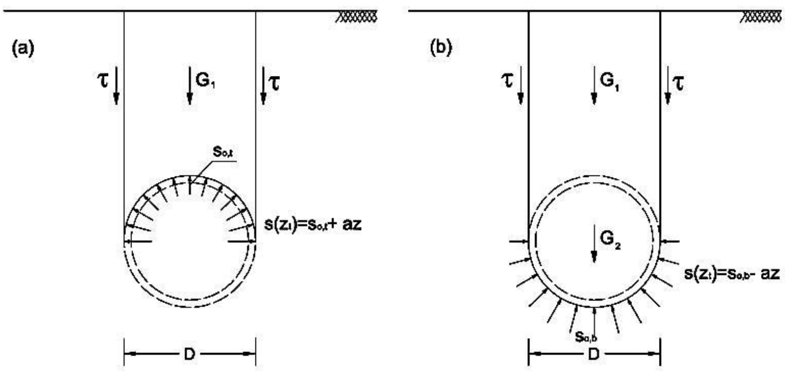


Fig. 10 *Blow-out model with vertical support pressure gradient a (a) at the upper part of the tunnel (b) at the lower part of the tunnel*

Figure 10 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 10(a) is given by:

$$s = s_{0,t} + aRcos\varphi \quad (22)$$

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

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

$$s_{0,t,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) - \frac{aD}{4} \quad (23)$$

Or

$$s_{0,t,max} = \left(\frac{C}{D} + \frac{1}{2}\right)^2 2DK_y\gamma' tan\varphi + \left(\frac{C}{D} + \frac{1}{2}\right)(\gamma D + 2c) - \frac{\pi}{8}\gamma D - \frac{aD}{4} \quad (24)$$

In the lower part as can be seen in Figure 10(b), the support pressure in the upper part of the tunnel section is:

$$s = s_{0,b} - aRcos\varphi \quad (25)$$

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

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

$$s_{0,b,max} = \gamma\left(H - \frac{\pi}{8}D\right) + 2\frac{H}{D}\left(c + HK_y\gamma' tan\varphi\right) + \gamma_T \pi d + \frac{aD}{4} \quad (26)$$

Or

$$s_{0,b,max} = \left(\frac{C}{D} + \frac{1}{2}\right)^2 2DK_y\gamma' tan\varphi + \left(\frac{C}{D} + \frac{1}{2}\right)(\gamma D + 2c) + \gamma_T \pi d - \frac{\pi}{8}\gamma D + \frac{aD}{4} \quad (27)$$

From Equations 24 and 27, the maximum required support pressures can be estimated depending on the ratio C/D in the case of linearly distributed support pressures. It is assumed that the unit weight of tunnel is $\gamma_T = 24\text{kN/m}^3$ and the vertical gradient of the grout $a = 7\text{kPa/m}$.

The relation between the maximum required support pressure at the upper and lower parts of the tunnel $s_{0,t,max}$ and the cover-to-diameter ratio C/D is showed in Figure 11 for tunnels with the diameter D from 1 meter to 10 meters in varied soil. The same conclusion is reached when analysing the relationship between the maximum support pressures and the ratio C/D.

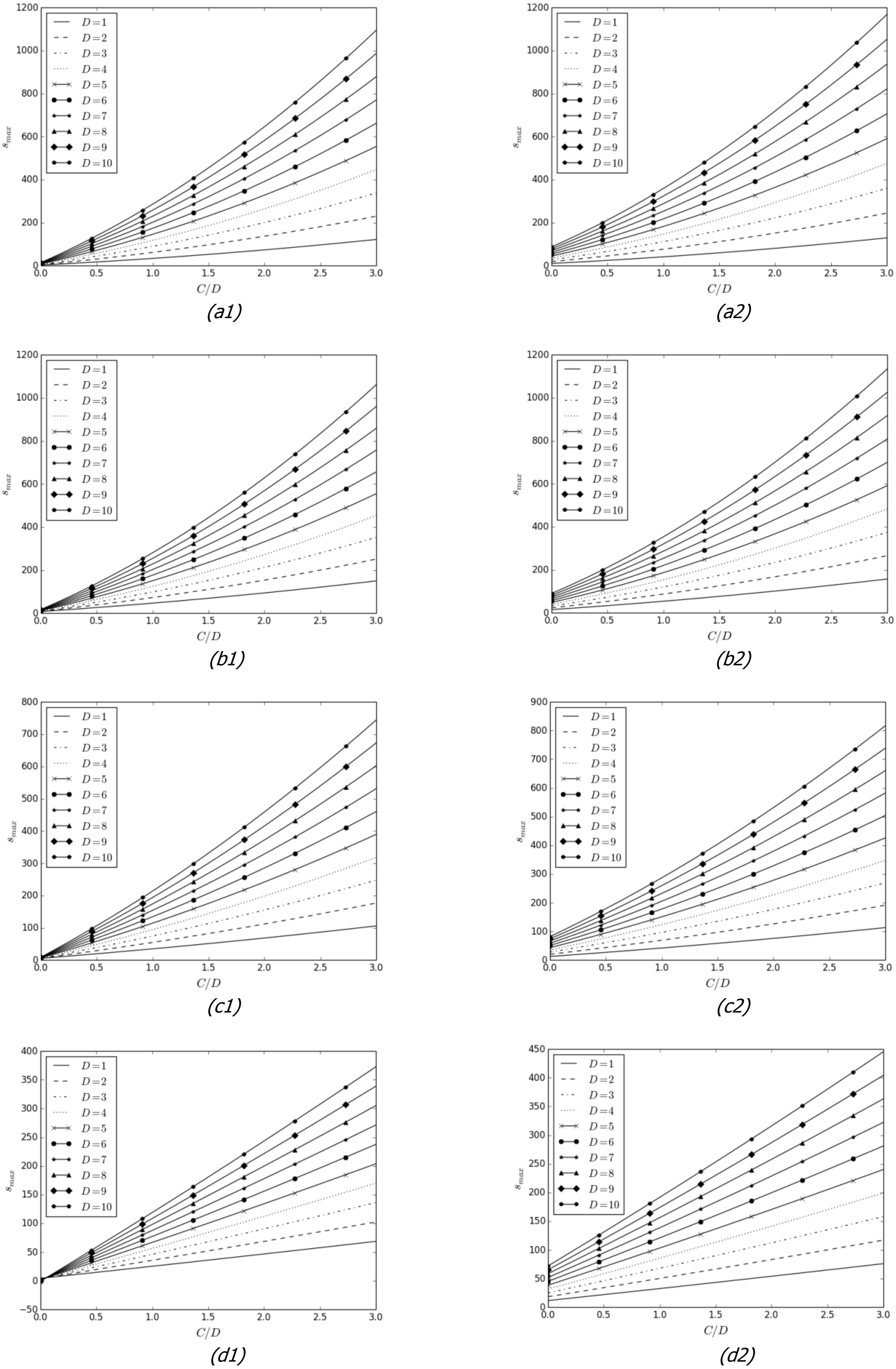


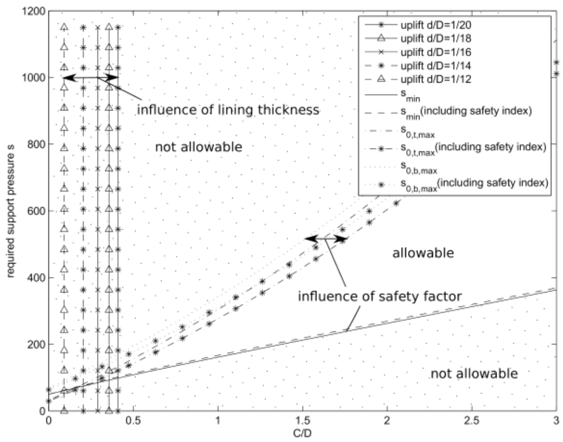
Fig. 11 *Maximum allowable pressure at the top (1) and bottom (2) of the tunnel with linear support pressure*

(a) in sand, (b) in clay, (c) in soft clay, (d) in peat

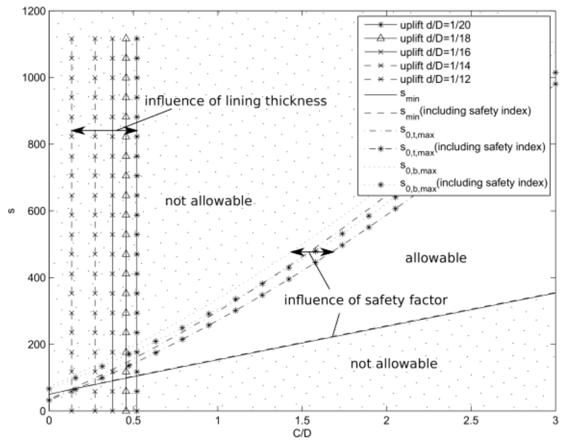
3. Combined analyses

In order to analyse the effects of the cover-to-diameter ratio C/D on the required support pressure, the uplift, blow-out and wedge stability models are combined with safety indexes for the cases of tunnels in sand, clay, soft clay and peat. In Figure 12, the following safety indexes are used in calculating: $\eta_{\text{blow-out}} = 1.1$ for blow-out; $\eta_{\text{uplift}} = 1$ for uplift; $\eta_{\text{porepressure}} = 1$ for pore pressure, and $\eta_{\sigma'_h} = 1.5$ for effective horizontal pressures.

This figure shows that tunnels in sand, clay or soft clay can be designed with very shallow overburden by changing the design of the tunnel segments, in particular, the d/D value. Nevertheless, it should be noted that there is a presence of sewage systems and other small infrastructure in the range up to about 4 meters below the surface. Therefore, for metro tunnels with a diameter in the order of 6 to $7m$, a ratio C/D in the range of 0.5 to 1 is the most shallow practical possibility.



(a)



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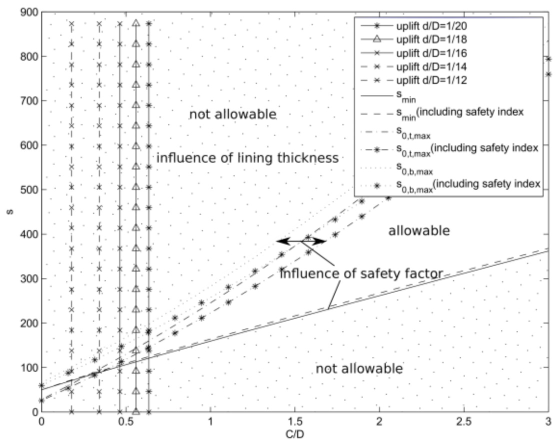
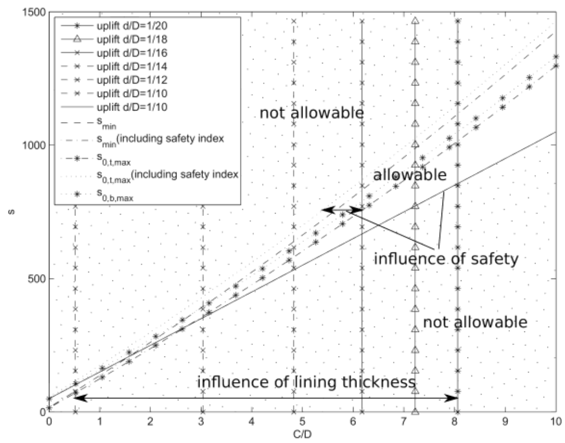


(c)



(d)

***Fig. 12** Relation between ratio of C/D and required support pressure in the case of tunnel (a) in sand, (b) in clay, (c) in soft clay, (d) in peat*

In the case of a tunnel in peat, Figure 12(d) shows that the tunnel can be designed theoretically at a very shallow level as the above cases. This would require increasing the weight of the lining (d/D in the order of $1/10$ or a similar amount of ballast in the tunnel) but would leave a small margin only between maximum and minimum support pressures. In practice, however, there needs to be a difference between the maximum and minimum pressures for safety reasons and to be practically workable. This implies that the tunnel cannot be designed with a low C/D ratio (should probably not be less than 6) in peat layers.

4. Conclusion

It is concluded that in the case of a tunnel in saturated sand and clay, the ratio of $\mathrm{C/D}$ can be reduced by changing the thickness of the tunnel in order to compensate the uplift or by adding ballast weight. However, the design depth of the tunnel should take into account the existence of utilities and other infrastructure systems. It should also be noted that in practice, there must be a limited difference between the maximum support pressure $\mathrm{s_{max}}$ and the minimum support pressure $\mathrm{s_{min}}$ in the order of about $50\mathrm{kPa}$ to guarantee a safe operation of the TBM. Therefore, in the case of a tunnel in peat, only high $\mathrm{d/D}$ ratio larger than $1/12$ allows a stable tunnel in somewhat condition even slightly predicted.

Based on the relation between $\mathrm{C/D}$ ratio and the support pressure, the range of support pressure can be estimated and can be used for estimating the support pressure in TBM machine, especially in EPB machine. In this paper, the effect of penetration of the support medium is not included. With slurry shields, the infiltration of the support medium may lead to excess pore pressure in front of the tunnel face and reduce the effective of the support (Broere, 2001). Therefore, the area of possible support pressure in the case of slurry shields may be smaller than suggested by Figure 12.

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