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Short-term tunnel face stability in clays

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ABSTRACT: On account of their low permeability, clays respond to tunnel excavation with a considerable delay. This is favourable for the interplay between ground, tunnel support and tunnelling equipment and, *inter alia,* also for the stability of the tunnel face. Nevertheless, even when tunnelling through practically impermeable clay deposits, the face may fail under certain conditions. The latter represents the subject of the present paper. Specifically, based upon a simple but accurate face stability model and a well-known empirical relationship, this paper provides generic answers to two important questions: which geotechnical conditions would result in an unstable face, thus necessitating, *e.g.*, closed-mode TBM operation? and which conditions would be prohibitive in this respect, *i.e.* the required face support pressure would be beyond today's technical feasibility limits, so that soil improvement measures would become indispensable?

1 INTRODUCTION

The response of saturated, low-permeability soils to tunnel excavation is pronouncedly time-dependent due to the progressive dissipation of excess pore pressures which soon develop in response to the tunnel excavation (Anagnostou 2007, Anagnostou *et al.* 2010). The delay in ground response is favourable for the interplay between ground, tunnel support and tunnelling equipment and, *inter alia,* also for the stability of the tunnel face (Schuerch *et al.* 2016). Despite this, the face may fail under certain conditions even in practically impermeable clays, for which undrained conditions prevail around the advancing face. In fact, undrained face stability belongs to the classic research topics in tunnelling (*cf., e.g.*, Broms and Bennermark 1967, Davis 1968).

The present paper analyses face stability conditions with the aim of answering two questions which are important from the tunnel engineering viewpoint: under which conditions (overburden, elevation of water table, over-consolidation ratio OCR etc.) would the face be unstable, thus necessitating, *e.g.*, closed-mode TBM operation? and which conditions would be prohibitive in this respect in terms of necessitating a face support pressure beyond today's technological feasibility limits? In the following pages, we attempt to provide generic answers to these questions based upon the combination of the recently proposed analytical face stability model of Pferdekämper & Anagnostou (2022) and Mesri's (1975) well-known and widely used empirical relationship between undrained shear strength and vertical effective stress of the ground.

2 FACE STABILITY MODEL

A circular tunnel of diameter D is considered, which crosses a clay deposit at at depth h (Figure 1). The water table may be located at the soil surface or – in the case of a subaqueous (open water) tunnel – at a distance d above the surface (*i.e.*, the seabed). The clay may be

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normally consolidated (NC) or overconsolidated (OC), whereby the maximum (past) burial depth of the soil surface will be denoted by the symbol b. The overconsolidation ratio (OCR) at the tunnel axis then equals $1+b/(h+D/2)$. The undrained shear strength of the soil s_u may be constant or increase linearly with depth z, from s_{u0} at the soil surface to s_{uT} at the tunnel crown.

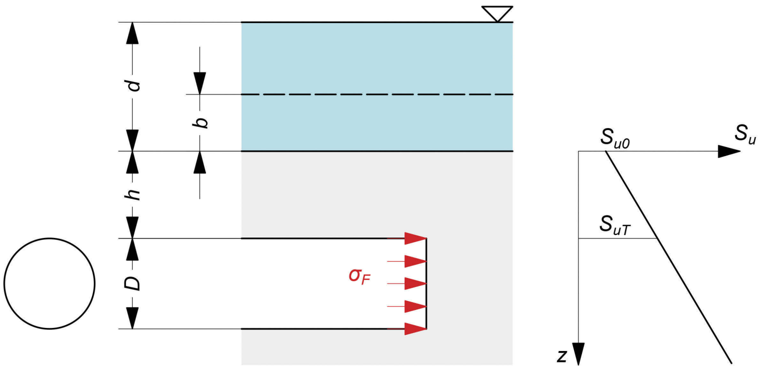

Figure 1. Problem layout.

Based upon an improvement of Gunn's (1980) lower bound trap door solution and Perazzelli & Anagnostou's (2017) tunnel face stability model, Pferdekämper & Anagnostou (2022) recently proposed the following equation for the required face support pressure σ_F in the problem under consideration:

$$\sigma_F = \sigma_{v0} - N_F\, s_{uT}, \tag{1}$$

where σ_{v0} and N_F denote the total vertical *in-situ* stress at the level of tunnel axis and the so-called stability factor, respectively, and read as follows:

$$\sigma_{v0} = \gamma'\left(h + \frac{D}{2}\right) + \gamma_w\left(h + \frac{D}{2} + d\right), \tag{2}$$

$$N_F \cong 4\ln\frac{h}{D} + 6.3 - \left(4 - \frac{4.5}{h/D}\right)\frac{s_{uT} - s_{u0}}{s_{uT}}, \tag{3}$$

where γ' and γ_w denote the submerged unit weight of the soil and the unit weight of the water, respectively.

The accuracy of Equation (3) was illustrated by comparative numerical analyses. Figure 2 shows the analytical predictions and the results obtained by finite element limit analysis (FELA). Equation (3) is very accurate in the case of constant shear strength (compare the black line with the black markers) and slightly conservative if the shear strength at the soil surface equals zero (compare the red line with the red markers).

In the following investigations into face stability conditions, the undrained shear strength will be taken after Mesri's (1975) empirical relationship, whereby the maximum past effective vertical stress (rather than the current effective vertical stress) will be considered for overconsolidated soils (*cf*. Ameratunga *et al*. 2016):

$$s_u(z) = \alpha\, \gamma'\, (z + b), \tag{4}$$

where, according to Mesri (1975), the coefficient $\alpha = 0.22 \pm 0.03$.

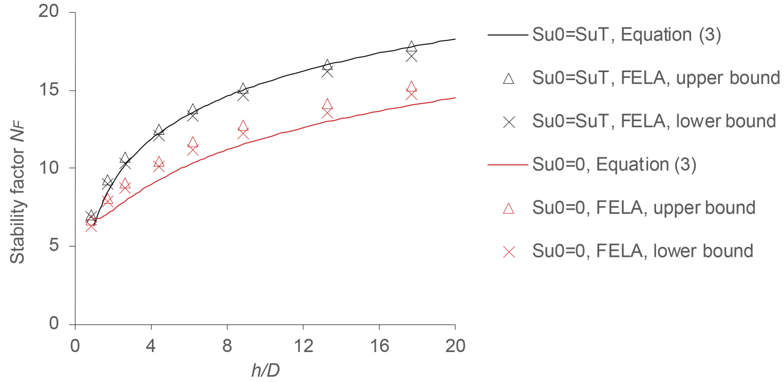

Figure 2. Stability factor N_F over normalized depth of cover h/D (after Pferdekämper & Anagnostou 2022).

Introducing Equations (2) to (4) into Equation (1) provides the required face support pressure as a function of the geometric parameters (h, D, d, b), the unit weights (γ', γ_w) and the coefficient α:

$$\sigma_F = (\gamma' + \gamma_w)\left(h + \frac{D}{2}\right) + \gamma_w d - \left(4(h+b)\ln\frac{h}{D} + 2.3h + 6.3b + 4.5D\right)\alpha\gamma'. \tag{5}$$

The computations of the next sections have been performed for $\alpha = 0.22$ and $\gamma' = 11$ kN/m^3, considering a variation of ±15% for both parameters. All results hold for a tunnel diameter of 10 m.

3 STABILITY CONDITIONS

3.1 *Land tunnel with water table located at soil surface*

Figure 3a shows the required support pressure σ_F over the depth of cover h for a normally consolidated (NC) soil. The solid curve of the diagram holds for the average values, the dashed one for the maximum values (+15%) and the dotted line for the minimum values (-15%). The dashed and the dotted lines represent best and worst cases, respectively, because the required support pressure decreases both with increasing α (trivial) and with increasing γ'. A higher γ' is favourable because it results in higher *in-situ* effective stresses and higher shear strengths (last right-hand side term of Equation 5), and this effect outweighs the unfavourable effect of a higher *in-situ* total stress (first right-hand side term of Equation 5).

According to Figure 3a, the support pressure increases with the depth, reaches a maximum at a depth of 2 – 3 tunnel diameters, decreases afterwards and becomes equal to zero for depths h greater than 3 to 9 tunnel diameters. (The normalization of depth h with respect to diameter D is approximately correct.) This highly non-linear dependency of support pressure on depth was also observed by Perazzelli & Anagnostou (2017) and is due to two conflicting effects: the unfavourable effect of the increasing *in-situ* total stress (first right-hand side term of Equation 1) and the favourable effect of the overburden (see second right-hand side term of Equation 1 in combination with Figure 2).

Considering that the dashed line and the dotted line represent best-case conditions and worst-case conditions, respectively, Figure 3 allows the following conclusions to be drawn for land tunnels crossing NC soils: face support is always indispensable in shallow tunnels with $h < 3D$; stable face conditions can be expected in deep tunnels with $h > 9D$ during advances or short standstills.

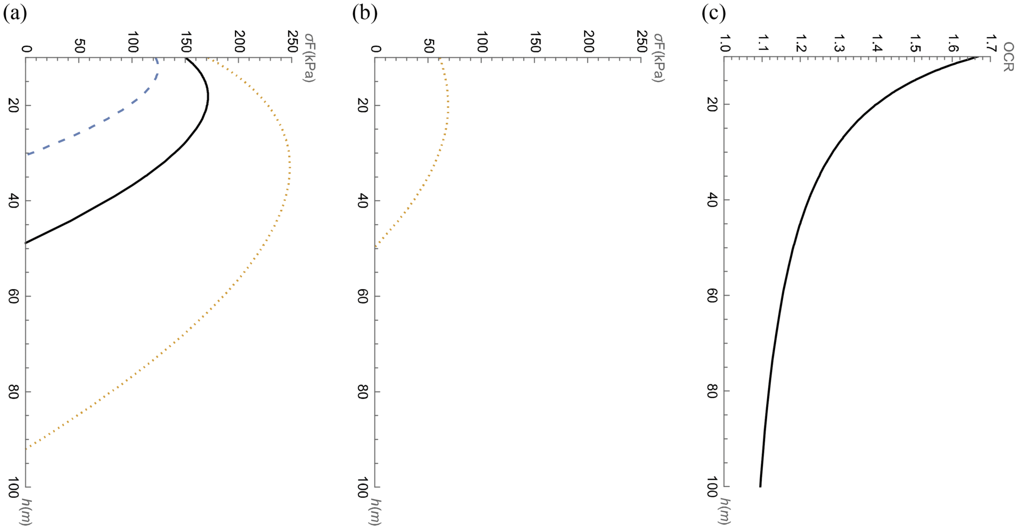

Figure 3. Land tunnel with water table at soil surface: Required face support pressure over depth of cover, (a), for a NC soil and, (b), for a slightly OC soil (past burial depth b = 10 m). (c) OCR at the tunnel axis in case (b) over depth of cover (tunnel diameter D = 10 m; strength coefficient $\alpha = 0.22 \pm 15\%$; submerged unit weight $\gamma' = 11$ kN/m^3 $\pm 15\%$).

Figure 3b holds for a slightly overconsolidated soil: the assumed past burial depth is equal to just 10 m; the corresponding OCR at the tunnel axis is mostly 1.2-1.5 (Figure 3c). The effect of overconsolidation is considerable: a stable face can be expected under the average- and best-case conditions; under the worst-case conditions, the maximum support pressure is four times lower than in the case of NC soil, while the critical depth, *i.e.*, the depth beyond which the support pressure equals zero, decreases from 90 m to 50 m (compare dotted lines of Figure 3a and 3b).

Figure 4 provides a complete picture regarding the critical depth of land tunnels in clays, showing the critical depth (vertical axis) as a function of the burial depth or the overconsolidation ratio (horizontal axes of Figure 4a and 4b, respectively). Points underneath the curves characterize stable face conditions. For an OCR of 1.5 (at the tunnel axis) and depths of cover greater than about 25 m (or, more generally, 2.5 diameters), stable face conditions can be expected even under the worst-case conditions (Figure 4b), that is always.

To summarize, in land tunnels, even a slight overconsolidation will result in stable face conditions or at least in a significant reduction in the face support pressure.

3.2 *Subaqueous tunnels*

The blue lines in the diagram of Figure 5 show the necessary face support pressure in a subaqueous NC clay deposit and hold for a water depth d of 50 m. The black lines hold for the limit case of d = 0 m (land tunnel) and are given for comparison. The weight of the water body results in an increase in the *in-situ* total vertical stress and an equal increase in the necessary support pressure (see first right-hand side term of Equation 1). The increase is equal to the hydrostatic pressure at the elevation of the sea bed, that is 500 kPa in the example of Figure 5, and causes a shift in the face support over depth curve (Figure 5). The critical depth of cover, that is the minimum depth of cover for stable face conditions (which in shield tunnelling would allow open mode operation), increases as well (by 30 - 60 m in the example of Figure 5).

Figure 6 provides a complete picture regarding the stability conditions in subaqueous tunnelling through NC soils in terms of minimum distance to the seabed h (vertical axes), sea depth d (horizontal axes) and material constants α and γ'. For points above the lines, the face

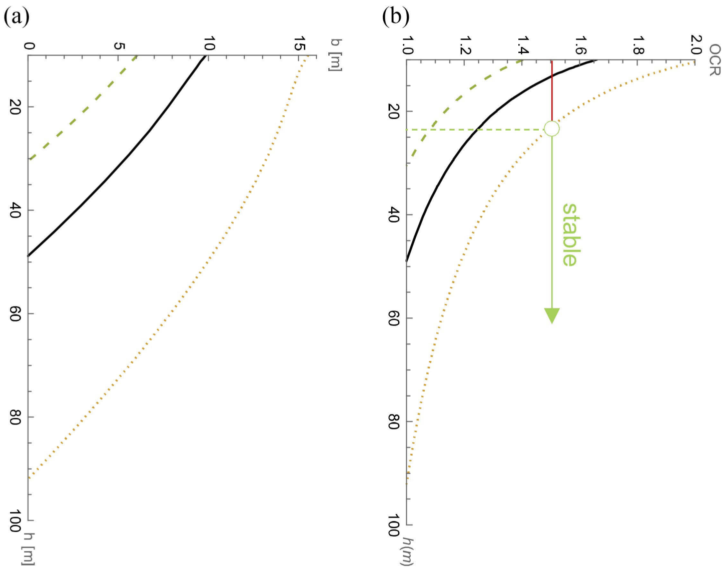

Figure 4. Land tunnel with water table at soil surface: (a) Required burial depth b and corresponding OCR at the tunnel axis for open mode excavation at a depth h (tunnel diameter D = 10 m; strength coefficient α = 0.22 ±15%; submerged unit weight γ' = 11 kN/m^3 ±15%).

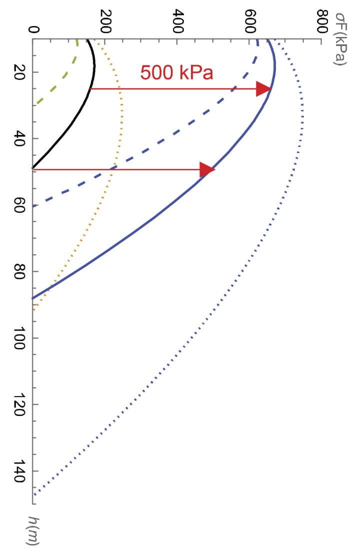

Figure 5. Subaqueous tunnel crossing a NC clay deposit: Required face support pressure as a function of the depth of cover for a sea depth d of 0 m (black lines) or 50 m (blue lines) (tunnel diameter D = 10 m; strength coefficient α = 0.22 ±15%; submerged unit weight γ' = 11 kN/m^3 ±15%).

is unstable: the distance of the tunnel from the seabed is too small for the given sea depth (or the hydrostatic pressure acting upon the seabed is too high for the given depth of cover). Two important limit cases of mechanized tunnelling are considered by the two diagrams of Figure 6: necessary face support pressure σ_F = 0, *i.e.* stable face conditions (Figure 6a); and σ_F = 15 bar, which was the slurry pressure over long stretches of the Lake Mead Intake No. 3 tunnel (Anagnostou *et al.* 2018), that is the highest face support pressure to be applied in

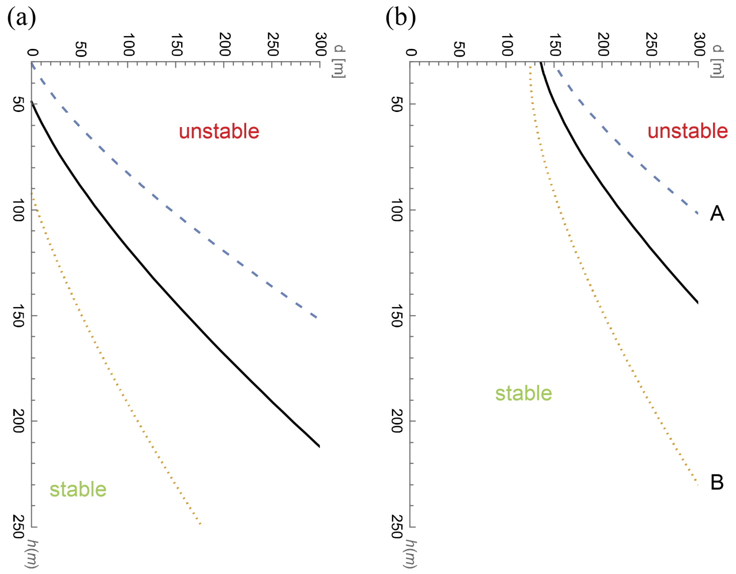

Figure 6. Subaqueous tunnel crossing a NC clay deposit: Required depth of cover h over sea depth d, (a), for open mode operation and, (b), for closed mode operation at 15 bar face support pressure (tunnel diameter D = 10 m; strength coefficient α = 0.22 ±15%; submerged unit weight γ' = 11 kN/m^3 ±15%).

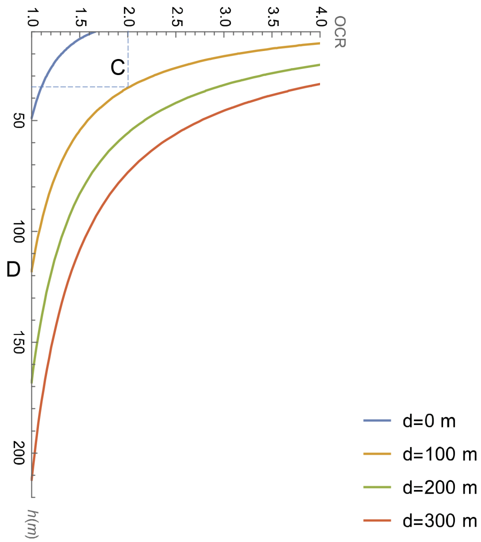

Figure 7. Open mode operation in a subaqueous tunnel crossing an OC clay deposit: Stability condition in terms of sea depth d, depth of cover h and OCR at the tunnel axis (tunnel diameter D = 10 m; strength coefficient α = 0.22; submerged unit weight γ' = 11 kN/m^3).

mechanized tunnelling so far (Figure 6b). The diagrams thus serve for assessing the feasibility of open mode TBM operation (Figure 6a) or of any shield tunnelling (Figure 6b) without auxiliary measures such as advance drainage, grouting or freezing. (Feasibility is understood here as feasibility of proven techniques and pressures, without considering conceivable and reasonable further technological progress.)

As an application example, the case of the planned Gibraltar Strait tunnel will be considered. The central part of this tunnel is expected to cross breccia, a NC clay matrix with hard inclusions (Dong *et al.*, 2013). The sea is 300 m deep. The minimum depth of cover for a such great sea depth is equal to 100 - 230 m, depending on the shear strength (Figure 6b, points A and B). According to the current vertical alignment, the tunnel will cross the breccias at a depth of 200 m underneath the seabed (Lombardi *et al.*, 2009). This appears reasonable, considering the conceivable technological advances with respect to feasible face pressure (Figure 6b assumes a maximum pressure of 15 bar) and that the breccia strength is towards the upper ranges (Dong *et al.*, 2013).

Overconsolidation is also extremely important, of course, for subaqueous tunnels. Figure 7 shows on the vertical axis the minimum distance to the seabed that would be necessary for open mode operation, as a function of the OCR at the tunnel axis (horizontal diagram axis) for sea depths d of 0 - 300 m. For a sea depth of, *e.g.*, 100 m, and an OCR of 1 - 2, the minimum depth underneath the seabed is equal to 40 - 120 m (Figure 7, points C, D).

4 CONCLUSIONS

Based upon a new and sufficiently accurate analytical solution to the undrained face stability problem and a well-known empirical relationship for the shear strength of clays, generic diagrams have been provided for the preliminary assessment of the face stability conditions of land tunnels and subaqueous tunnels crossing normally consolidated or overconsolidated clays.

The results presented illustrate the intricate relationship between the necessary face support pressure and the overburden, and the enormous effects of vertical alignment and overconsolidation on face stability conditions.

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