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On the face support of microtunnelling TBMs

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ABSTRACT

Face stability of microtunnelling TBMs is an important aspect for a safe and controlled project execution. Lack of proper face support can lead to sudden collapse with resulting large settlements. Guidelines for minimal and maximal support pressures in most codes do not take the infiltration of bentonite suspension in coarser soils into account. Infiltration lowers the effectiveness of the face support. In loose sands infiltration can lead to excess pore pressures and induce liquefaction, with possible catastrophic consequences. This paper investigates the influence of infiltration and gives some guidelines for a proper selection of bentonite suspensions based on soil gradation.

1. Introduction

Face support remains an important aspect for closed front microtunnelling. Without proper face support, the tunnel face may collapse suddenly or an uncontrolled and unobserved overexcavation may occur over parts of the project, both leading to possible large settlements at the surface. This is particularly the case in slurry supported TBMs in non-cohesive soils.

For large diameter TBM driven tunnels many authors have looked into face stability, where following the idea of Horn (1961) the wedge shaped limit equilibrium model has become very popular. Whereas Jancsecz and Steiner (1994) describes a basic implementation, Anagnostou and Kovári (1994) implements the infiltration of slurry during stand-still and Broere (2001) studies the effect of infiltration and excess pore pressures during excavation. The validity of these models is underscored by numerical and experimental work by Vermeer and Ruse (2000), Ruse (2004), Plekkenpol et al. (2006) and Kirsch (2009).

For microtunnelling, various authors have focussed their attention on the jacking forces during advance (Wilkinson, 1999; Chapman and Ichioka, 1999; Röhner and Hoch, 2010) and on the impact of lubrication to limit or control the jacking forces in various conditions (Shou et al., 2010; Barla et al., 2006; Pellet-Beaucour and Kastner, 2002) or on the interaction between jacking forces and soil response in difficult conditions (Broere et al., 2007).

Less attention is paid to the face stability requirements for microtunnelling. For instance, the Dutch code for pipeline systems NEN3650-1:2003 (2003) delegated the requirements for the execution of pipeline works using trenchless techniques to an Appendix, where less than a single page is dedicated to the specific requirements for slurry supported closed front machines, and these requirements remain non-quantifiable.

As given there these requirements include:

- The slurry pressure in the excavation chamber must be maintained within predetermined boundaries, to prevent face collapse or blow-out. (For example the minimal support pressure could be set at the active effective stress plus the water pressure plus 0.02 MPa; the maximum pressure as neutral effective stress plus water pressure.)
- Measures shall be taken to control that pressures exceed these bounds.
- The support fluid supply must be controlled with respect to pressure and discharge, in order to react immediately to changing circumstances.
- Especially within non-cohesive soils it is important to ensure that the support pressure does not induce lowered effective stresses in the soil.
- In uniform sands or layers that are prone to static liquefaction it is necessary to add bentonite [to the support fluid].

Although the need for a minimum and maximum support pressure is recognized, as well as the need for control of these boundary pressures, no calculation methods are give, apart from the example included in the guideline. Also, no specifications are given for what constitutes a support fluid, apart from the remark that in extremely problematic soils it should include bentonite.

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This ambiguous (from a technical point of view) definition of a support fluid in a slurry supported TBM opens up (from a legal point of view) the possibility to use plain water, without any additives, as the support fluid. If there is no need to use bentonite this slightly lowers the project cost, and for competitive reasons many tenders in the Netherlands have been made stating explicitly that no bentonite will be used, even in non-cohesive layers. This practise has contributed to several incidents with uncontrolled face collapse, overexcavation or extreme surface settlements (Bezuijen, 1996; Hölscher, 2008). This practise actually overlooks the requirement that the effective stresses in the soil should not be lowered, as discussed below.

Internationally, codes and design guidelines give similar limited attention to face stability. Japanese design guidelines documented by Osumi (2000) specify an effective support pressure of 20 kPa, irregardless of depth, diameter or soil conditions. German practice, as documented by Stein (2005), finds the calculation method for diaphragm walls by Walz et al. (1983) of sufficient accuracy and thereby implicitly uses the same model as Jancsecz and Steiner (1994), without considering the specific impact of smaller diameters.

This paper will look at the application of stability models for small diameter tunnels and at other face support requirements that can be introduced to ensure a safe and successful microtunnelling project using a slurry supported machine.

2. Face stability models

The slurry pressure applied at the tunnel face should be higher than the actual pore pressure and the horizontal effective stress in order to ensure stability of the face and to prevent excessive deformations. Field experience (Hölscher, 2006; Arends and Soons, 2004) as well as laboratory experiments (Chambon et al., 1991; Kirsch, 2009) and numerical simulations (Ruse, 2004) all show that the component of effective horizontal stress that needs to be countered is lower than the traditional (plane strain) active effective earth pressure, as derived by Rankine (1857). As is shown clearly in the experiments by Chambon et al. (1991), a significant influence of soil arching around and above the shield should be taken into account.

This is one of the reasons why the wedge stability models (see Fig. 1), such as proposed by Horn (1961), and often implemented as detailed in the paper by Jancsecz and Steiner (1994), are popular. These models allow the user to relatively straightforward incorporate the effects of arching, even though, as Kirsch (2009) shows, there are a number of somewhat arbitrary model choices that can be used to tune the model outcome. These include the exact arching formulation, whether a plane strain or fully three-dimensional silo description is used, how the shear stress on the side planes of the failing wedge is incorporated in the calculations and whether or not the effects of a layered non-homogeneous overburden are taken into account (Broere, 1998). The arching aspects have also been discussed in a recent paper by Anagnostou (2012).

In a homogeneous soil and for a given set of model choices, the normalized effective support pressure

$$N_D = \frac{s'}{\gamma'_w D} \tag{1}$$

can be precalculated, as done by Jancsecz and Steiner (1994) (tabulated there as K_{3D} values). Here s' is the effective slurry pressure (the difference between slurry pressure s and pore pressure p), γ'_w the effective volumetric weight of the soil and D the tunnel diameter.

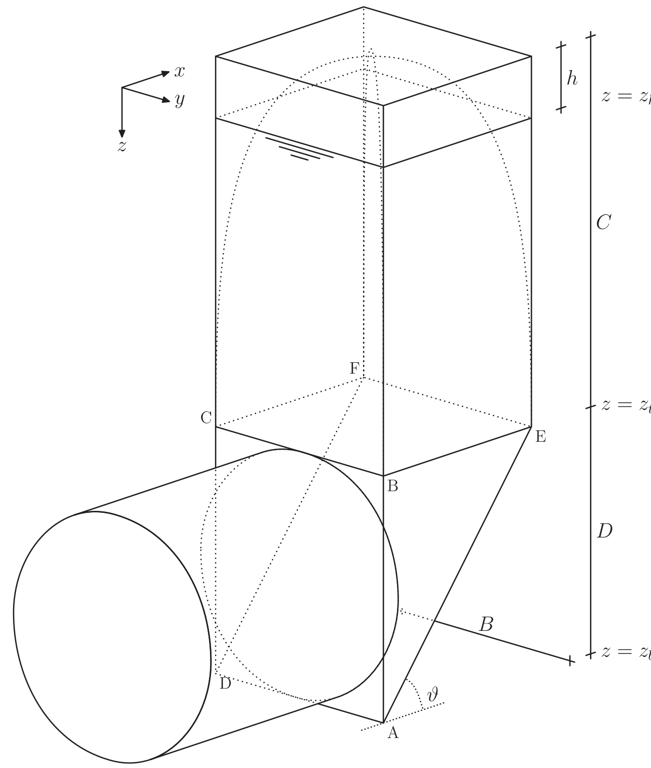

Fig. 1. Wedge stability model.

Tabulated values of N_D could then be used to calculate the required support pressure

$$s = s' + p = \gamma_1 N_D \sigma'_v + \gamma_2 p \tag{2}$$

including safety factors γ_1 and γ_2 for the effective stress and pore pressures respectively. In theory, this would provide a simple design method for the minimal face support pressure. As soon as effects of a multi-layered overburden, or even multiple layers at the tunnel face (Broere, 1998), are taken into account, the use of N_D becomes cumbersome, as its value depends on the stratigraphy. This is even more so when the effects of infiltration and excess pore pressures are considered. Anagnostou and Kovári (1994) incorporates the influence of a slurry infiltration zone in the wedge stability model. During stand-still, the slurry will infiltrate the grain skeleton until a maximum penetration depth is reached. This maximum penetration depth

$$e_{\max} = \alpha \frac{\Delta p \, d_{10}}{\tau_F} \tag{3}$$

depends on the pressure difference over the infiltration zone Δp, the characteristic grain size d_{10}, the yield strength of the slurry τ_F and a form factor α (Krause, 1987). As the support pressure is then transferred to the grain skeleton over this infiltration zone, instead of the ideal thin filter cake modelled by Jancsecz and Steiner (1994), the effectiveness of the support drops especially in coarse grained soils.

If, during excavation, the filter cake is completely excavated along with the soil by the cutter teeth of the TBM, the pressure difference s' between the support pressure in the excavation chamber of the TBM and the pore pressures in front of the TBM drives the infiltration of the slurry into the grain skeleton. This infiltration is quick at first and can be characterized using an infiltration half-time a as

$$e = \frac{a}{a+t} e_{\max} \tag{4}$$

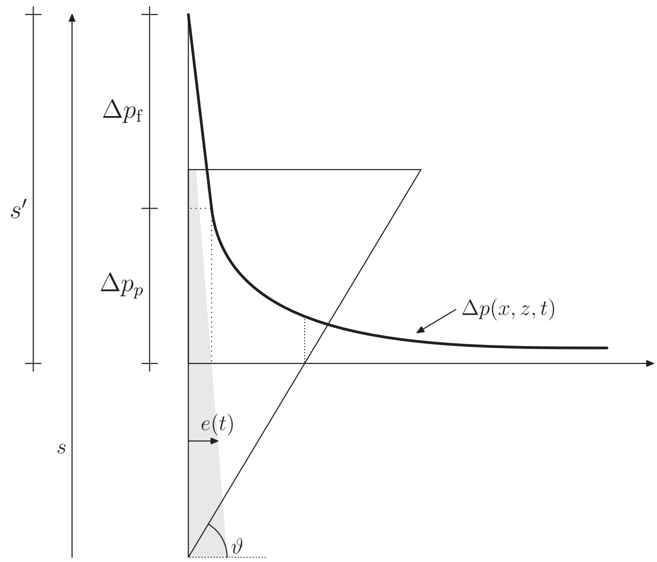

Fig. 2. Definition of pressure distribution over penetration zone and excess pore pressures.

with t the time since the start of infiltration. The half-time a depends strongly on the composition of the slurry, as found by Krause (1987). If further the soil is saturated, the infiltrating slurry must displace the volume of water already present in the pores, leading to excess pore pressures and a groundwater flow radiating away from the TBM. During excavation, the partially built-up filter cake is constantly removed by the cutter wheel and this infiltration continues. As the cutter wheel stops, the filter cake can penetrate to maximum depth and the infiltration stops. Excess pore pressures in front of the TBM can then dissipate. This process could be modelled as a time-dependent infiltration just in front of the TBM, as sketched in Fig. 2. The effective support pressure s' is partly transferred to the soil skeleton by drag forces in the slurry infiltration zone, resulting in a pressure drop Δp_f over this infiltration zone, estimated as

$$\Delta p_f = \frac{e}{e_{\max}} s' \tag{5}$$

and a remaining excess pore pressure Δp_p. This excess pore pressure in turn serves as input for a transient groundwater flow solution, with the exact solution $\Delta p(x, z, t)$ dependent on the geohydrological conditions of the site. In almost all cases no more than a single aquifer will be present at the tunnel face and a linear transient flow solution for a single semi-confined aquifer is sufficiently detailed. See Broere and van Tol (2001), Broere (2001) for full details.

The extent to which such excess pore pressures are generated in front of the TBM depends on the permeability of the soil, the grain size distribution, the properties of the slurry and the rotation speed of the cutter wheel, amongst others. In impermeable, fine grained soils there will effectively be no infiltration. In highly permeable, coarse grained soils the excess pore pressure will dissipate so quickly as to play no role. However, in medium fine soils, especially fine sands, excess pore pressures can develop. These will lower the effectiveness of the support pressure, as the difference between support pressure and the actual pore pressure drops, and lower the stability of the soil wedge, as the effective stresses are lowered by excess pore pressure and as a result the friction between the failing soil wedge and surrounding soil is lowered. Both effects require an increased support pressure to stabilize the face.

Whereas the N_D values derived by Jancsecz and Steiner (1994) are generally lower than the coefficient of active effective earth pressure (Rankine, 1857), when the influence of infiltration is taken into account the values can be as much 3 times higher and approach $N_D = 1$ for cases with limited cover and poor soil conditions (Broere and Hergarden, 2010). Generally, the required support pressure should be calculated for a specific project, taking the TBM dimensions, local soil conditions and stratigraphy into account, rather then estimated based on tabulated values for simplified cases.

2.1. Case studies

A limited series of case studies and parameter variations is included here to show the possible influence of infiltration and soil conditions. Four theoretical cases are used:

Case 1: the TBM is located in a homogeneous sand layer. The sand is assumed to have saturated volumetric weight $\gamma_s = 20\ \text{kN/m}^3$, angle of internal friction $\phi = 30^\circ$ and $d_{10} = 100\ \mu\text{m}$.
Case 2: the TBM is located in a coarse sand layer. Properties are as for case 1, except $d_{10} = 30\ \mu\text{m}$.
Case 3: the TBM is located in a loose packed sand layer. Properties are as for case 1, except $\phi = 20^\circ$.
Case 4: the TBM is situated in a sand layer just below a peat layer. properties of the sand layer are as for case 1, the peat layer has $\gamma_s = 11\ \text{kN/m}^3$.

In all calculations safety factors γ_1, γ_2 have been set to 1 and a volumetric weight of the bentonite slurry $\gamma_F = 10\ \text{kN/m}^3$ is used. This is a low, pessimistic, estimate for a bentonite slurry. In the infiltration model by Broere (2001) the excavation speed of the TBM plays a role. In these calculations a ratio $a/f = 5$ between the infiltration half-time a and the average time between cutter teeth passages f has been assumed. This corresponds to $a = 30$ s for a TBM wheel with 5 spokes and 2 rpm. For case 3 calculations have also been made for a poor quality slurry, with $a/f = 50$. In all cases a model factor $\alpha = 2.5$ has been used.

Figs. 3–6 show results for TBM diameters $D = 1$, 2 and 3 m and cover to diameter ratio $C/D = 0.5$, 1, 2, 3 and 4. In the model by Anagnostou and Kovári (1994) the yield strength of the slurry τ_F influences the results. Results given here are for the minimal slurry

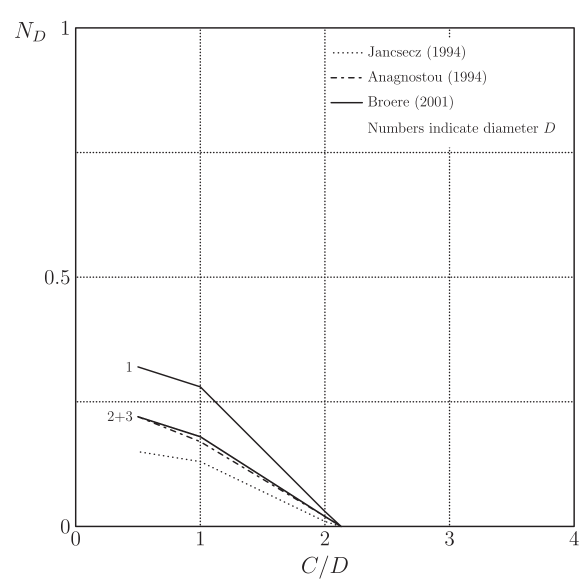

Fig. 3. Stability ratios N_D for case 1: TBM located in a sand layer.

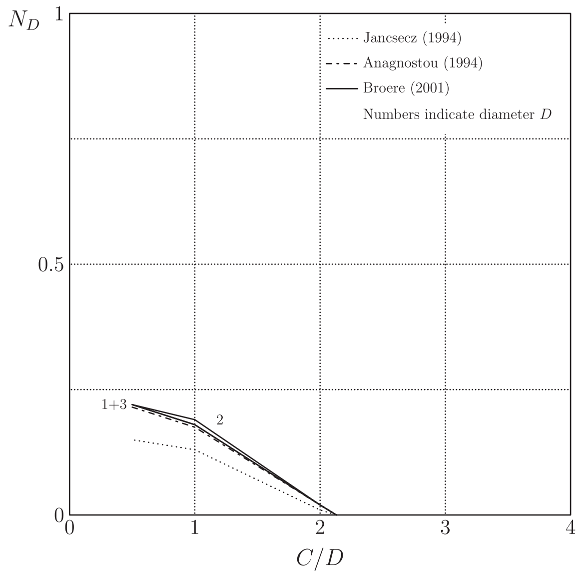

Fig. 4. Stability ratios N_D for case 2: TBM located in a coarse sand layer.

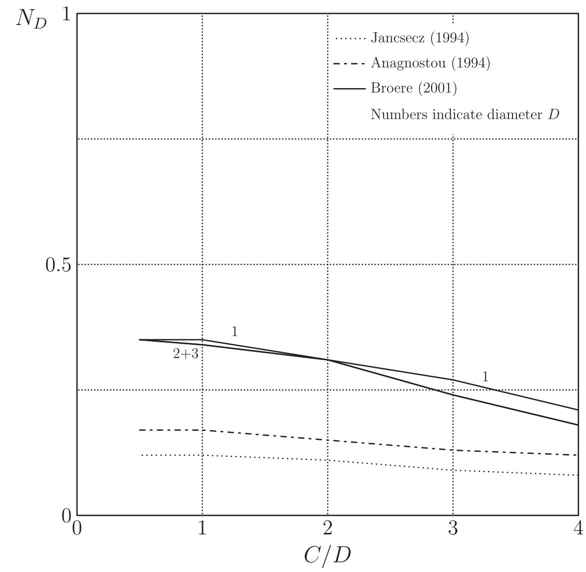

Fig. 6. Stability ratios N_D for case 4: TBM located just below a peat layer.

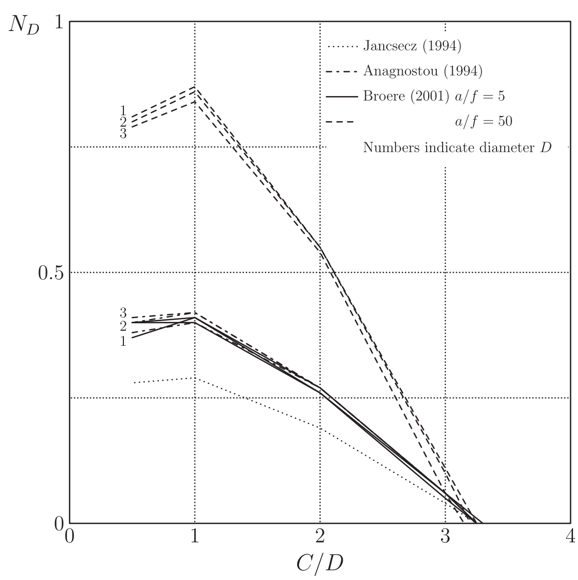

Fig. 5. Stability ratios N_D for case 3: TBM located in a loose sand layer.

quality obtained by their model for which the face can remain stable. In all cases this minimal yield strength is lower than that found from Eq. (6), discussed below, indicating that in these cases the infiltration mechanism from Anagnostou and Kovári (1994) is not governing the minimal required slurry quality.

The results in Figs. 3–6 show, for many small diameter tunnels ($D < 3$ m) with the tunnel situated in a homogeneous sand layer with a cover over diameter ratio C/D of more than 2 to 3, that arching dominates the global failure mechanism to such an extent that global face stability, at least theoretically, is ensured if the support pressure is equal to the pore pressure, i.e. N_D values drop to 0. This can be seen for example in Fig. 3 for $C/D > 2.1$ and in Fig. 5 for $C/D > 3.2$. If, on the other hand, the overlying layers are soft soils with limited shear capacity, e.g. peat or soft clay, arching does not occur as strongly and the calculated N_D values resemble those without such soft soil overburden at all (see Fig. 6).

3. Micro-stability and liquefaction

The conclusion one might draw from the previous section is that for relatively deep ($C/D > 4$) tunnels in sand, face stability is almost automatically ensured. This conclusion is not generally true, however, for a number of reasons.

First, part of the stabilizing force in the model taking infiltration into account, is derived from the drag force of the infiltrating medium into the soil. In theory it is indeed possible to stabilize a body of cohesionless sand by the drag force of a constant infiltration of water, as long as a sufficient gradient ($i \geqslant 2$) is maintained (van Rhee and Bezuijen, 1992). For this drag force to be present, and effectively act away from the TBM, the infiltration can never be interrupted. In a microtunnelling project where one has to periodically insert a new tunnel segment and elongate the water or slurry feeds, this is not practically achievable.

Secondly, one has to consider the microstability at the tunnel face, i.e. the stability of the individual soil grains (also known as "inner stability"). A single grain at the outer edge of a vertical wall of cohesionless material is inherently unstable. If it drops out of the matrix, the next grain is not stable and effectively grains would start to rain of the wall, slowly undermining the stability. To prevent this a (limited) amount of shear capacity or a drag force on the grains is needed. The minimum yield strength τ_F of the support fluid that will keep a vertical wall of cohesionless frictional material (sand) stable can be estimated based on Müller-Kirchenbauer (1977) and Kilchert and Karstedt (1984) as

$$\tau_F = d_{10}(1-n)\frac{\gamma'}{\tan\phi} \tag{6}$$

where n is the porosity and ϕ the angle of internal friction.

If the yield strength of the slurry is too low and sand grains start to rain off, the face will move away from the TBM. The speed at which the face will move can be estimated from the rate at which static liquefaction progresses

$$v = \frac{k}{\Delta n}\frac{\rho_k - \rho_w}{\rho_w}(1-n)\cot\phi \tag{7}$$

with k the permeability of the sand, Δn the porosity difference between the sand in the matrix and in the slurry, ρ_k the specific density of the grains and ρ_w the specific density of water (van den Berg et al., 2002). For medium dense to dense sand, this formula yields speeds of 1 mm/s or less, and a short interruption of the infiltrating drag force would not result in a serious disturbance at the face before operations are resumed. For a loose sand, however, Δn becomes small and the resulting speed high.

Although the risk of static liquefaction could now be determined based on the change in porosity, a more practical approach has been suggested for the recent update to NEN3650-1:2012 (2012). Field experience, at excavations and other types of construction works, as documented by CUR166 (2005), shows that static liquefaction becomes an issue for sand layers with a relative density $D_r < 55\%$. CUR166 (2005) estimates the relative density from a cone penetration test (CPT) based on the work by Schmertmann (1976). More recent work by Jamiolkowski et al. (2003) provides an updated relationship between cone resistance q_c and relative density D_r, which has been used to plot Fig. 7. This graph can be used for a quick evaluation of the liquefaction potential of sand layers in front of the TBM.

Where sand layers susceptible to liquefaction exist, with the susceptibility determined from a CPT, the updated NEN3650-1:2012 (2012) prescribes the use of a bentonite based slurry. For other site conditions, a minimum slurry quality according to Eq. (6) is prescribed if the characteristic grain size $d_{10} > 10\ \mu m$, and no conditions are set for finer graded soils. These requirements ensure that in sandy soils a minimum amount of fines needs to be present in the slurry and the use of clean water as a support medium is not allowed any more.

On the other hand, these new requirements do leave the established practice open, if mixed soil conditions exist at the face, and a sufficient clay fraction is excavated and kept in suspension at the face, to keep the fine fraction of the excavated material in suspension and pump it back to the face as a low-cost low-quality slurry. Implicit in this method is the need for the contractor to continuously check the resulting slurry quality on site. And although for fine sands this approach of re-use of excavated fines may be both practical and theoretically sound, it has clear limits in coarser material. Krause (1992) suggests that bentonite based suspensions have approximately 10 times higher yield strengths than non-swelling clay based suspensions of the same density and that for coarser sands the required yield strength, i.e. the required density, is so high that this is not practically attainable using non-swelling clays.

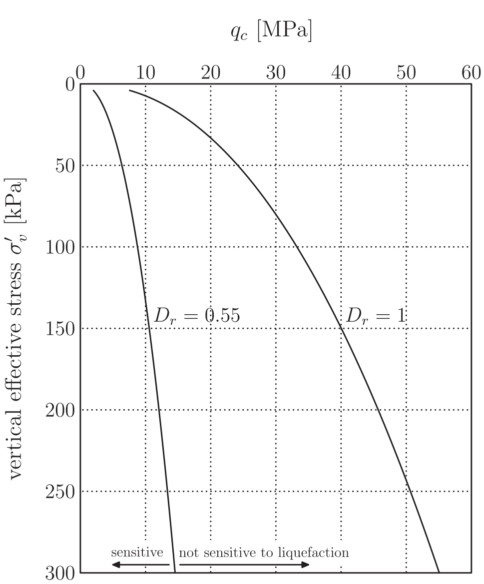

Fig. 7. Relationship between cone resistance q_c, effective vertical stress σ'_v and relative density D_r indicating the zone of loose packed sands (after Jamiolkowski et al., 2003).

4. Conclusions

When the influence of infiltration of the support medium into the soil during excavation, and the subsequent generation of excess pore pressures in front of the TBM, is taken into account, the required minimum support pressure in permeable non-cohesive soils can increase significantly. This is especially the case at low overburden, or where the overburden is composed of soil layers with low strength, i.e. peats and soft clays.

The generated excess pore pressures lower the effective stresses in the soil. This can be problematic in non-cohesive sand layers, where a lack of microstability of the individual grains can lead to a slow, gradual and ongoing collapse of the tunnel face. In loose sand at low relative density this mechanism can be relatively quick and give rise to static liquefaction of the soil in front of the TBM. This can result in initially undetected overexcavation and extremely large settlements at surface.

In order to prevent micro-instabilities or static liquefaction of loose sand layers, minimum requirements to the yield strength of the (bentonite) suspension used in slurry TBMs should be posed. The required minimal yield strength might be obtained by keeping sufficient fines, excavated at the face, in suspension by only partially cleaning the returns from the TBM, and reusing these as a low-cost low-quality suspension. The expectation is, however, that in practice the amount of fines present in coarser sand and gravel layers, as well as uniform fine sand layers, which soil types need a sufficient yield strength of the suspension to be stable, is too low to be practical. Combined with the fact that swelling clays like bentonite are an order more effective (by weight percentage) than non-swelling clays in building up a sufficient yield strength of the suspension and thereby preventing micro-instabilities, it is highly recommended to use bentonite based suspension in lieu of pure water to stabilize the tunnel face.

At the same time, a bentonite suspension will more effectively clog the pores at the excavation face and thereby limit the amount of (filtrate) water that flows from the excavation chamber into the soil. This infiltration water will generate excess pore pressures that lower the effective stresses of the soil, and thereby lower the global face stability.

Only where site conditions are such that infiltration and liquefaction are not an issue, face stabilization with water should be considered at all, and there it should be combined with a continuous control on the actual soil conditions and the actual quality and yield strength of the suspension present at the face.

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