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Influence of infiltration and groundwater flow on tunnel face stability

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ABSTRACT: During the excavation of tunnels in saturated soft soils, a pressurized bentonite slurry is often used to support the tunnel face. The slurry should form a filter cake, which is used to transfer the support pressure onto the soil. This filter cake is constantly removed by the cutter teeth of the TBM during excavation and slurry will infiltrate into the soil while the filter cake rebuilds. This results in excess pore pressures directly in front of the tunnel face. To quantify the influence of these excess pore pressures on the stability of the tunnel face, a simple groundwater-flow model has been incorporated in a wedge stability analysis. For a tunnel in fine sand layers, the pore pressure distribution calculated with this model has been compared with field measurements. It is shown that the minimal support pressure is significantly higher, compared with full-membrane calculations.

1 INTRODUCTION

During the excavation of a tunnel in soft water bearing soils, it is often necessary to provide a temporary support to prevent collapse of the working face. To estimate the minimal support pressure needed a number of models has been proposed over the years. Based on observations made in field and laboratory tests Broms & Bennermark (1967) derived a stability ratio for purely cohesive soils. Davis et al. (1980) obtained a set of overburden dependant stability criteria based on the limit analyis of a purely cohesive (Tresca) material, which corresponded well with the results of centrifuge tests in clay. Using a similar approach Atkinson & Potts (1977) proposed an expression for the minimal support pressure in a cohesionless frictional material and Leca & Dormieux (1990) presented a number of upper and lower bound solutions for the more complex case of a cohesive-frictional material. The common factor in all these models is that they do not incorporate the effects of pore pressures or heterogeneities of the soil body.

In permeable soils below the water table the pore pressures are often the main contribution to the forces that have to be supported by the shield. In order to incorporate the pore pressures in a stability analysis, Jancsecz & Steiner (1994) suggested a wedge shaped failure mechanism loaded by a soil silo, as sketched in Figure 1. This approach is similar to the one outlined by Horn (1961) and has parallels with the calculation methods used for slurry filled trenches. Jancsecz & Steiner used this wedge model to calculate the minimal support pressures for a tunnel in a purely frictional material and showed the effects of varying overburden.

In the model of Jancsecz & Steiner it is assumed that the entire support pressure is transferred onto the soil skeleton by an infinitely thin membrane at the working face. In slurry shield tunneling bentonite, other clays or aggregates are often added to the working chamber to provide such a filter cake. This cake should seal the working face to prevent the slurry from infiltrating the soil and should ensure an efficient transfer of the support pressure onto the soil skeleton. In fine grained soils a filter cake is indeed formed on top of the working face and acts as an impermeable membrane.

In coarse soils however, the slurry will infiltrate into the soil to a certain extent, see e.g. Müller-Kirchenbauer (1977), and the full-membrane assumption is not valid. The effectiveness of the support decreases as the slurry penetrates the soil which undermines the stability of the tunnel face with increasing slurry penetration distance. Anagnostou & Kovári (1994) presented a wedge stability model which included this effect of slurry infiltration. They assumed that the penetration distance of the slurry would always be greater than the cutting depth of the tunnel boring machine (TBM) and reasoned that a constant penetration distance would then establish itself during boring. They showed that in coarse soils the support pressure calculated with a full-membrane wedge model may well suffice during the actual excavation, but that during stand-still the slurry penetrates into the soil and gradually undermines the stability of the tunnel face, until collapse of the face occurs.

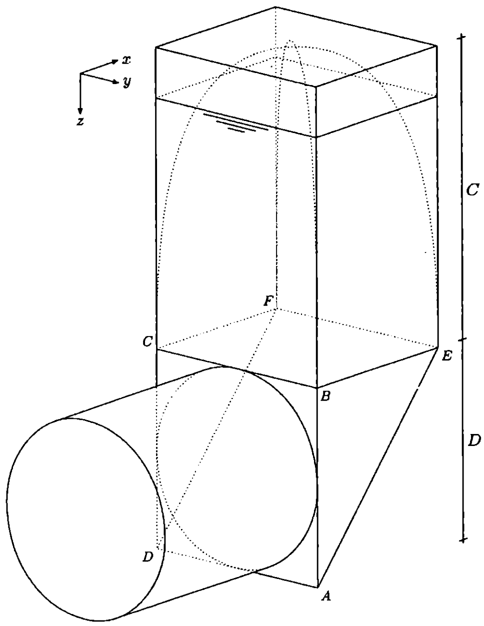

Figure 1. Wedge and silo model.

Although the wedge stability model as presented by Anagnostou & Kovári included the influence of the equilibrium pore pressures as well as the bentonite penetration process, it does not deal with the development of excess pore pressures in front of the TBM. Excess pore pressures may occur if the outflow of ground water in the layers before the TBM is confined, for example, when tunneling in sand lenses or layers overlaid by impermeable clay and the filter cake at the tunnel face is somehow disturbed or too permeable.

Such a disturbance of the filter cake can be of major importance in fine to medium granular sands. During stand-still a filter cake will establish itself at the face and the assumption that the face is sealed by an impermeable membrane is reasonably valid. During excavation however, it may well occur that the cutting depth of the teeth on the cutter wheel of the TBM is so large that the entire filter cake is removed by each passage of the cutter arms. After the filter cake is removed, the slurry will infiltrate into the soil and a new filter cake will build. During this process large amounts of filtrate water will flow into the soil, resulting in excess pore pressures in front of the TBM.

This combination of effects adversely affects the stability of the tunnel face in a number of ways. Firstly the removal of the filter cake reduces the effectivity of the support pressure, as described by Anagnostou & Kovári. Unless the soil is coarse however, this is a relatively minor effect. Secondly the increase of pore pressures leads to a reduction of the effective stresses and thereby a reduction of the friction capacity of the failure wedge. And the increased pore pressures effectively cancel part of the support pressure. To compensate for this combination of effects a higher minimal support pressure is needed, as compared to a full-membrane model. This higher support pressure will in turn lead to an increased filtration rate and higher excess pore pressures.

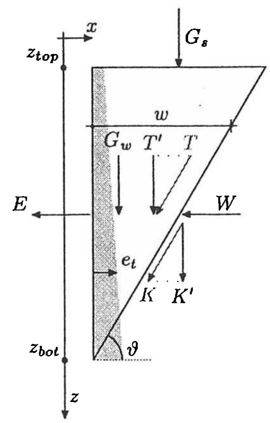

Figure 2. Definition of forces acting on the wedge and infiltration zone (shaded area).

The Authors have incorporated a simple ground water model in a wedge stability analysis and used the resulting model to quantify the influence of the slurry infiltration process and excess pore pressures on the stability of the tunnel face. The results have been compared with field measurement obtained at the site of the Heinenoord tunnel in the Netherlands.

2 WEDGE STABILITY MODEL

The collapsing soil body in front of the face can be schematised as a triangular wedge loaded by a soil silo, as sketched in Figure 1. Forces acting on the wedge are the effective weight of the wedge itself, the vertical weight of the silo, the shear forces acting along the slanted failure plane (AEFD) as well as the triangular side planes, the load resulting from the pore water and the support force of the slurry. For a given value of the angle ϑ between the horizontal and the slip surface, see Figure 2, the resulting equilibrium equations can be solved and the required support pressure is found iteratively by maximising the resulting support pressure. Jancsecz & Steiner (1994) have presented the resulting minimal support pressures for a tunnel in homogeneous non-cohesive soil.

As the development of excess pore pressures is most likely to occur in heterogeneous, stratified soils, we will need to extend the basic equilibrium equations of the failure wedge to include soil stratification. This will result in a set of equilibrium equations for each layer that intersects the wedge. As shown in Broere (1998) it is possible to reformulate this problem in terms of integrals over the entire wedge.

To this end we define $w(z)$ the width of the wedge at depth z and introduce the shorthand notation ζ^+ and

ζ^- for the goniometric relations

$$\zeta^+ = \tan\varphi\sin\vartheta + \cos\vartheta \tag{1}$$
$$\zeta^- = \tan\varphi\cos\vartheta - \sin\vartheta. \tag{2}$$

It should be noted that the angle of internal friction φ generally depends on the depth z, i.e. $\varphi(z)$, especially when different soil layers occur within the failure wedge. This also holds for the other soil properties used in this Paper. However, to keep the formulae readable this depth-dependance has not been written explicitely. Of special interest is the fact that the slip angle ϑ may vary gradually or discretely within different layers, as this allows the introduction of different failure shapes into the stability analysis. For the numerical examples presented in this Paper the simplification has been made that ϑ is constant for all z, as this significantly reduces the number of iterations needed to determine the minimal support pressure.

Now the vertical components of the friction forces on the slip surfaces of the wedge can be written as

$$K' = D\int_{z_{bot}}^{z_{top}} \frac{c}{\zeta^- \sin\vartheta}\mathrm{d}z, \tag{3}$$
$$T' = \int_{z_{bot}}^{z_{top}} \left[c + K_y\sigma_v'\tan\varphi\right]\frac{w}{\zeta^-}\mathrm{d}z, \tag{4}$$

where K' results from the friction on the slip plane AEFD and T' from plane ABE. Here c denotes cohesion, K_y is the coefficient of horizontal effective stress and σ_v' is the vertical effective stress. The effective wedge weight G_w is simply the integral of the effective volume weight of the soil γ' times the wedge width w times tunnel diameter D and the vertical load at the top of the wedge resulting from the soil silo is

$$G_s = D^2\cot\vartheta\,\sigma_v'(z_{top}), \tag{5}$$

where the effects of soil arching as well excess pore pressures have to be incorporated in the calculation of σ_v'.

For a given value of slip angle ϑ the resulting horizontal wedge force E can be found from

$$E = -D\frac{G_s + G_w + K' + 2T'}{\int_{z_{bot}}^{z_{top}} \frac{\zeta^+}{\zeta^-}\mathrm{d}z}. \tag{6}$$

Now the support force $S = E + W$ for a given angle ϑ can be calculated from the force exerted by the water at the slip plane AEFD,

$$W = D\int_{z_{bot}}^{z_{top}} p_0(z) + \Delta p(w(z),z)\mathrm{d}z. \tag{7}$$

In order to calculate W the pore pressures at rest p_0 and the excess pore pressures $\Delta p(x,z)$ are needed. To establish the distribution of excess pore pressure we will take a closer look at the effects of the slurry infiltration and groundwater flow.

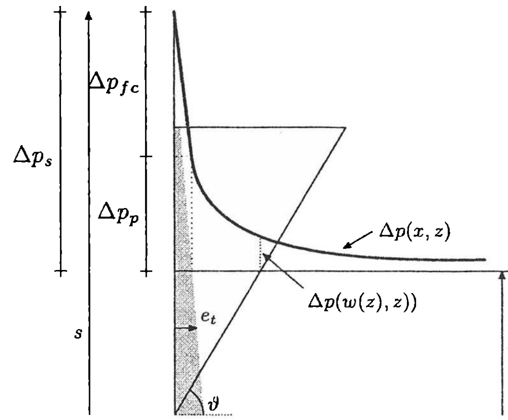

Figure 3. Pressure drop over penetration zone and excess pore pressure distribution.

2.1 *Slurry Infiltration*

If the slurry pressure exceeds the pore pressures in the soil, as is needed to support the face in non-cohesive soils, the slurry will infiltrate the soil and form a filter cake. The influence of this process on the stability of vertical trenches has been studied by a.o. Müller--Kirchenbauer (1977) and Kilchert & Karstedt (1984). From their work we find that the maximal penetration distance e_{max} depends primarily on the total pressure difference over the filter cake Δp_s, which is the difference between the support pressure s and the pore pressure in rest p_0.

$$e_{max} = \frac{\Delta p_s d_{10}}{\alpha\tau_F} \tag{8}$$

Here d_{10} is the characteristic grain diameter at which 10%mass passes the sieve, τ_F the shear strength of the bentonite and α describes the relation between the grain size and the effective radius of a flow channel, and normally takes values between 2 and 4, see Krause (1984) or Kilchert & Karstedt (1984).

When during excavation the cutter teeth largely or completely remove the filter cake at the face, the slurry will again start to infiltrate and a new cake will be built over time. During this build-up process, we can no longer assume that the support pressure is transferred onto the soil skeleton by a membrane, but seepage forces from the slurry and filtrate water will stabilize the soil. Krause (1987) has shown that the time-dependant infiltration distance e_t can be described by

$$\frac{e_t}{e_{max}} = \frac{t}{a+t}. \tag{9}$$

The timespan a is the time at which half the maximim penetration distance is reached. This value can for example be determined from a column infiltration test.

By linearising the dependance between the penetration distance e_t and the pressure drop over the filter

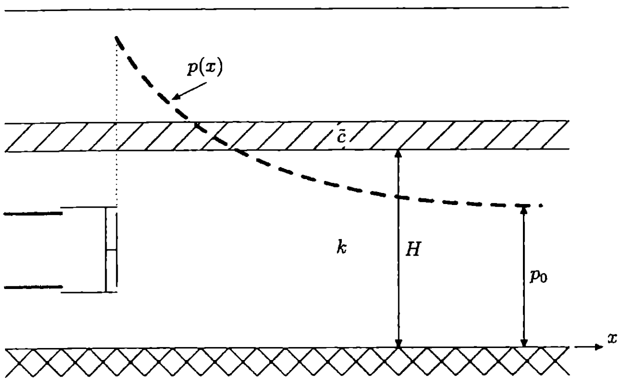

Figure 4. Schematisation of semi-confined aquifer.

cake Δp_{fc}, we can invert (8) to find the pressure drop over the partial filter cake at each moment in time as

$$\Delta p_{fc} = e_t \frac{\alpha \tau_F}{d_{10}}. \tag{10}$$

In practice the infiltration distance will vary over the face with the time passed since the cutting wheel has removed the filter cake. For this model however a mean pressure drop over the partial filter cake is assumed, which can be found by selecting a proper mean infiltration distance e_t, i.e. a proper value for the time span t since the removal of the filter cake. Field observations suggest t approximately equal to half the time span between subsequent passages of the cutter arms. We assume that the resulting pressure drop equals the seepage forces exerted by the infiltrating slurry on the soil skeleton and the remaining excess pressure

$$\Delta p_p = \Delta p_s - \Delta p_{fc} \tag{11}$$

is used to compute the hydraulic head which will be used as input for the groundwater flow model.

2.2 *Groundwater Flow*

Essentialy the inflow of the filtrate water into the soil is a non-stationary ground water flow problem. Using the filtration velocity of the slurry, which can be easily derived from (9), the discharge of filtrate water from the working chamber can be estimated. Field observations suggest that in situations where the build-up of excess pore pressures significantly influences the stability of the face, the discharge of filtrate water is large enough that an almost stationary flow will be reached in the vicinity of the face during the excavation process. When this is the case, the ground water flow can be approximated as a stationary flow problem, which will yield to a safe upper bound value for the excess pore pressures.

The basic equations of stationary flow are described in many text books on groundwater mechanics, see e.g. Verruijt (1970). In the current application we will schematise the soil in front of the the TBM as a one-dimensional semi-confined aquifer, see Figure4, as this is appropriate for the case history considered in the next Section. In situations where no impermeable or semi-permeable layers are present the solution of an unconfined radial flow might be more appropriate.

Using the hydraulic head at the far side of the partial filter cake Δp_p as the necessary boundary condition, the excess pore pressure distribution in the semi-confined aquifer can be established as

$$\begin{aligned} \Delta p(x,z) &= p(x,z) - p_0(z) \\ &= \Delta p_p(z) \exp(-x/\lambda) \end{aligned} \tag{12}$$

where λ is the leakage factor

$$\lambda = \sqrt{kH\bar{c}}. \tag{13}$$

The leakage factor can be computed from the permeability k and height H of the aquifer and the hydraulic conductivity $\bar{c}$ of the overlying aquitard, see also Figure 4.

2.3 *Solution procedure*

For a given value of the slip angle ϑ the resulting wedge force E can be found if the distribution of excess pore pressures $\Delta p(x,z)$ is known. This distribution in turn depends on the excess support pressure Δp_s, which can be calculated if the water force W and wedge force E are known. In general an iterative solution procedure will be needed to solve this interdependance, in which an estimated support pressure is used as input for the ground water calculation and a new estimate of the support pressure is calculated. This process is repeated until an equilibrium of the excess pore pressure profile is reached.

When the support pressure for a given slip angle has been found, a second iteration procedure is needed to find the angle ϑ for which the wedge force E is maximal. The resulting support pressure is the required minimal support pressure.

3 FIELD MEASUREMENTS

The influence of excess pore pressures on the face stability will be illustrated with measurements taken at the monitoring field of the Second Heinenoord tunnel. This 8.55m diameter tunnel has been constructed under the River Oude Maas in the vicinity of Rotterdam in soft Holocene and Pleistocene layers. As the building site is relatively close to open sea a tidal influence is observed in the river and a damped and retarded tidal variation is also observed in deeper sand layers. Table 1 provides an overview of the (simplified) soil stratification and parameters (Bakker, 1996). Other parameters which have been used in the calculations are listed in Table 2.

Table 1. Soil stratification and parameters for Second Heinenoord Test Field North.

Description	Top [m]	$\gamma_{wet}(\gamma_{dry})$ [kN/m^3]	c [kPa]	φ [°]	K_0	k_h [m/s]	$\tilde{c}$ [s]
Mixture of sand and clay	+2.50	17.2 (16.5)	3	27	0.58		
Sand, clayey	−1.50	19.5	0	35	0.47		
Sand, clay layers	−5.75	19	0	30	0.47		10^5
Sand, clayey in places	−10.0	20.5	0	36.5	0.45	$1 \cdot 10^{-4}$	
Sand, gravelly	−17.5	20.5	0	36.5	0.50	$2 \cdot 10^{-4}$	
Clay, sandy in places	−20.75	20	7	31	0.55		

Table 2. Parameter values.

Tunnel axis	NAP -13.2m
Water table	NAP +0.0m
Bentonite shear strength	$\tau_F \approx 5$ Pa
Characteristic grain size	$d_{10} = 4\ \mu$m
Excavation speed	2 rotations/min (5-spoke wheel)

Using a full-membrane model to calculate the minimal support pressure s at the tunnel axis, we find a representive value $s = 148$kPa. This is only 16kPa over the pore pressure at that depth $p_0 = 132$kPa and compares well with findings from saturated centrifuge tests (Bezuijen 1997). When the influence of excess pore pressures is included in the calculations however, as outlined in this Paper, a minimal support pressure $s = 212$kPa is found, approximately equal to the mean support pressure used. The required difference between p_0 and s has risen to 80kPa, a five-fold increase compared to the ful–membrane model.

As part of a research program coordinated by the Centrum Ondergronds Bouwen (COB), a number of test fields were set up on both sides of the river. The layout of the test fields has been previously described by Leendertse et al. (1997). Of special interest are the piezometers installed in the track of the TBM. These were positioned at such a depth that they were roughly aligned with the center of the cutter wheel and were destroyed as the TBM passed the gauge. Up to that point they provided a clear picture of the excess pore pressure distribution in front of the TBM, which has been plotted in Figure 5.

In Figure 5 the measured pore pressures have been plotted against the distance to the tunnel face. In this representation the piezometric head during stand-still, approximately 120kPa, is given by the lowest point on the downward spikes that can be observed at 1.5m intervals. A small tidal influence can be observed in these measurements during stand-still. The remainder of the plotted line represents the excess pore pressures measured. These are only measured during the actual excavating process and immedeately afterwards. As soon as the cutter wheel is stopped a drop in the excess pore pressures is observed and the remaining excess pore pressures dissipate.

From the support pressure $s = 212$kPa a mean pressure drop over the partial filter cake $\Delta p_{fc} = 48$kPa has been calculated and the remaining excess pressure $\Delta p_p = 32$kPa has been used as input for the groundwater calculations. The resulting excess pore pressures have also been plotted in the Figure 5. As can be seen prediction and measurement are in good accordance.

The Authors conducted a parameter study which showed that the calculated excess pore pressures are highly sensitive to the leakage factor λ of the semi-confined aquifer. Furthermore there is a strong dependency on ratio of the excavation speed to the slurry infiltration velocity, as the angular speed of the cutter wheel influences the mean infiltration distance used in the calculations.

These observations are confirmed by measurements taken during the construction of another tunnel in the vicinity of Rotterdam. This tunnel has been bored in somewhat coarser sands at location where no clearly definable impermeable overlaying strata are present. Again using the calculations outlined above, excess pore pressures up to 5kPa were expected in the vicinity of the tunnel face, decreasing strongly with distance. This prediction corresponded very well with pore pressure measurements obtained.

4 CONCLUSIONS

When during excavation with a slurry shield the filter cake at the face is removed, the slurry will infiltrate the soil to build a new filter cake and excess pore pressures will develop in front of the TBM. These phenomena have been included in a face stability model and lead to an implicite set of equations, which has been solved iteratively.

Such excess pore pressures have been shown to be a major influence on the stability of the tunnel face as they reduce the effectiveness of the support force from the slurry and lower the effective stresses in the soil, reducing the friction capacity of the soil. To compensate for these effects a higher support pressure is needed. A significant reduction of the face stability is most likely to occur in heterogeneous soils and depends strongly on the permeabilities of the dif-

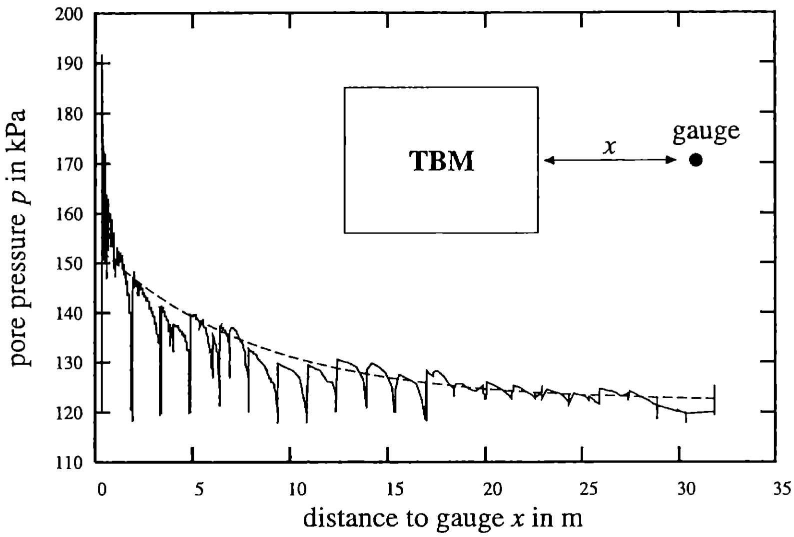

Figure 5. Excess pore pressure measurements taken at the Second Heinenoord Test Field North, adapted from Bezuijen (1998), compared to calculated pore pressure distribution (dashed line).

ferent soil layers and the mean infiltration distance of the slurry, which can be controlled by the excavation speed of the cutting wheel.

The minimal support pressure and excess pore pressures calculated with the presented stability model correspond well with field measurements and experience.

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