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Influence of excess pore pressures on the stability of the tunnel face

W. Broere
A. Broere's Aannemingsmij. BV, Amsterdam, The Netherlands
Geotechnical Laboratory, Delft University of Technology, The Netherlands

ABSTRACT: In shield tunnelling a bentonite slurry or a foam may be injected by the TBM into the soil. These injections will displace part of the pore water before the TBM and, especially in fine sands, will cause excess pore pressures in front of the TBM. These excess pore pressures increase during boring and dissipate during stand-stills. They decrease the margin between the minimal and maximal allowable support pressure. Using a time-dependant groundwater flow model this behaviour has been included in a limit equilibrium model, in order to predict the pore pressures measured in front of the TBM. The results have been compared to field measurements.

1 INTRODUCTION

The most common tunnel boring techniques used in soft, water bearing soils are the slurry shield and the earth-pressure balance (EPB) shield. In a slurry shield a pressurised slurry is used to stabilise the tunnel face. This slurry is normally injected into the working chamber at a pressure higher than the pore water pressure in the soil. Due to the pressure difference the slurry will infiltrate the soil and form a filter cake at the face (Krause 1987). This filter cake will then seal the face, help to transfer the slurry pressure onto the soil skeleton and protect against micro-collapses at the face.

During excavation however, the filter cake is constantly removed by the cutter bits and re-established by renewed infiltration of slurry into the soil. As a result there is a continuous inflow of filtrate water into the soil, resulting in excess pore pressures in front of the TBM. These excess pore pressures lower the effective stresses and thereby the stability of the tunnel face.

In an EPB shield the excavated soil is used to support the tunnel face, often conditioned with additives like bentonite slurry or foam. These additives are injected into the working chamber at pressures above the pore water pressure and will infiltrate the soil in front of the TBM, displacing the pore water present there. Part of the effectiveness of the foam treatment of the soil rests in the fact that the foam replaces part of the pore water, lowering the water content of the soil (Maidl 1995). As a result of this infiltration process excess pore pressure are generated in front of the face.

Anagnostou & Kovári (1996) have shown that the minimal required support pressure for an EPB shield can be calculated using a similar wedge stability model as used for slurry shields. Although the stability of the face is in general not considered a problem in EPB shield tunnelling, an accurate indication of the minimal required support pressure is required to control the surface settlements and to prevent a face collapse in the event of a partially filled working chamber. The effects of the slurry or foam infiltration on the stability of the face can be calculated using a stationary groundwater flow model and incorporating the results in a wedge stability model (Broere & van Tol 2000).

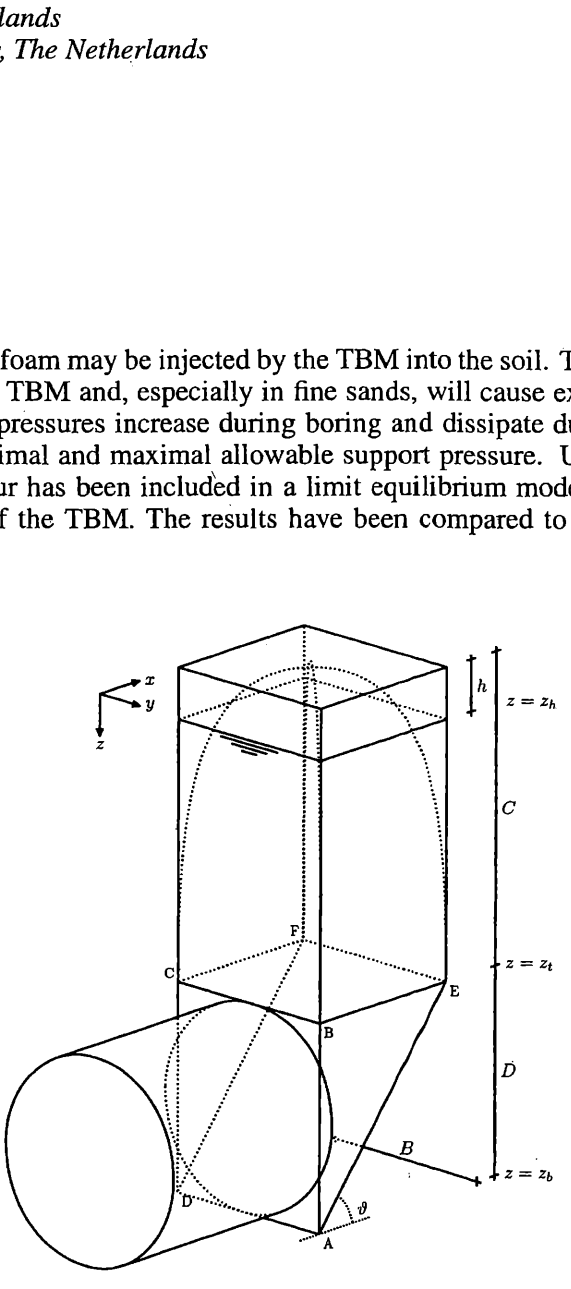

Figure 1. Wedge and silo model.

The implicit assumption made when a stationary groundwater flow model is used is that the infiltration and groundwater flow reach equilibrium quickly after the start of excavating. In many cases where the infiltration has a significant effect on the stability of the face, the permeability and specific storage of the aquifers are such that this assumption is not valid.

In the case of foam injection in a silty sand layer, presented as a case study later in this article, the time required to reach equilibrium flow conditions is larger than the 30 to 60 minutes it takes to excavate a lining section. Also the generated excess pore pressures will not always be fully dissipated when the next excavation cycle starts.

These effects can be modelled using a transient groundwater flow model coupled to the wedge stability calculations. With the resulting model the build-up of excess pore pressure during excavation can be predicted, as well as the dissipation of these excess pore pressures during stand-still. With this model the effects of the infiltration process on the stability of the tunnel face will be quantified.

2 WEDGE STABILITY MODEL

The basic wedge stability model is a limit equilibrium analysis, in which the collapsing soil in front of the TBM is schematised as a triangular wedge, loaded by a soil silo (see figure 1). This wedge is assumed to be a rigid body, loaded its effective weight and the overburden resulting from the soil silo. On the side planes of the wedge the cohesive-frictional forces are taken into account, derived from the horizontal effective stress. A possible shear force at the top of the wedge is generally not taken into account.

Equilibrium of these forces results in an effective earth force acting towards the tunnel face, which has to be countered by the effective support force S'. This is the difference between the total support force S and the water force W that results from the pore pressure. For a given wedge angle θ the resulting earth force E can be calculated. The minimal support pressure can be found by iterating over the angle θ and maximising E.

As sketched here the model is similar to that presented by Jancsecz & Steiner (1994) and suitable for homogeneous soils only. It has been shown by Broere (2001) that the wedge stability model can easily be extended to stratified soils. In this form the analysis assumes that the full support force S is transferred onto the soil skeleton at the tunnel face. In the case of a slurry supported face it is, therefore, necessary that the bentonite forms a filter cake and seals the face, so that the slurry cannot infiltrate the soil. In medium to fine sands this is normally no problem during stand-still.

However, during excavation the cutter bits constantly remove the filter cake. When the filter cake is removed, the slurry immediately starts to infiltrate the soil, in order to form a new filter cake. In this process filtrate water from the slurry infiltrates the soil and generates excess pore pressures in front of the TBM. These excess pore pressure lower the effective stresses in the soil, and thereby the frictional forces acting on the wedge. They also increase the water force W and this lowers the effective support force S'. To counter these effects a higher support force S, i.e. a higher support pressure, is needed. Unfortunately this also increases the infiltration rate of the slurry and the resulting excess pore pressures.

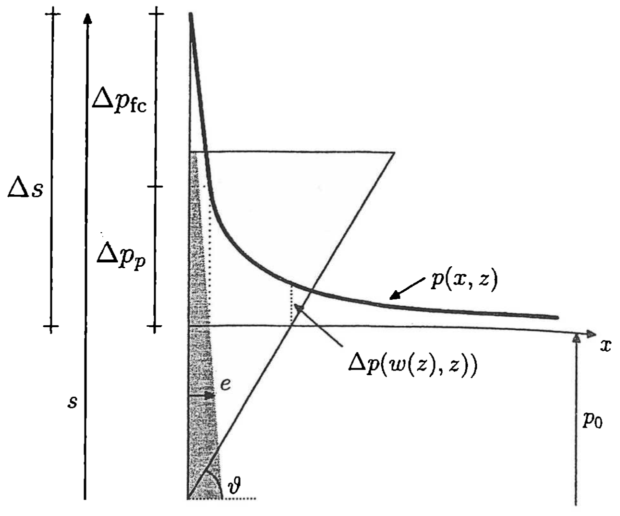

Figure 2. Pressure drop over the slurry infiltration zone and excess pore pressure distribution.

The effect of the constantly excavated and rebuilding filter cake has been modelled by Broere & van Tol (2000) as a pressure drop Δp_{fc} over a partial filter cake and included in a wedge stability model. Although the infiltration length of this filter cake varies over the face, an average infiltration length e, depending on the average infiltration time, has been adopted. The pressure drop over this partial filter cake has been substracted from the excess support pressure Δs (the difference between the total support pressure s and the pore pressure at rest p_0), to obtain the remaining excess pore pressure Δp_p, see figure 2. This excess pore pressure Δp_p has been used as input for a stationary groundwater flow problem.

The solution to this groundwater flow problem can be adapted for the hydrogeological conditions at the construction site, but for most cases the simplification to a single one-dimensional semi-confined aquifer is acceptable. In that case the excess pore pressures at depth z and as a function of the distance from the face x are given by

$$\Delta p(x, z) = \Delta p_p(z) \exp\left(\frac{e - x}{\lambda}\right) \tag{1}$$

for $x > e$, where λ is the leakage length of the aquifer.

From this excess pore pressure profile the reduction of the friction forces acting on the wedge, resulting from the lower effective stresses, can be calculated.

The increase in the water force W is calculated by integrating the excess pore pressures at the slanted front plane of the wedge. Also taken into account is an uplift force at the top of the wedge, which reduces the overburden load. As shown by Broere (2001) the impact of these effects on the stability of the tunnel face can be severe. Especially in medium to fine grained sands, with permeabilities between $10^{-3} < k < 10^{-5}$m/s, the required excess pore pressure Δs may be four times higher than predicted with an impermeable filter cake.

2.1 *Transient Groundwater Flow*

The assumption made in adopting a stationary groundwater flow model is that the excess pore pressures reach equilibrium shortly after boring has started. This assumption is not always valid, as the time required to reach stationary flow conditions may be larger than the average time needed to excavate the length of a single tunnel lining ring. In that case the excess pore pressures will not reach the level predicted by (1), and the effect on the stability of the face is overestimated.

A better prediction of the excess pore pressures in front of the face can be obtained using a transient groundwater flow model. The basic differential equation for transient flow in a semi-confined aquifer is given by Strack (1989) as

$$\nabla^2 \Phi = \frac{\Phi}{\lambda^2} + \frac{S_s}{k} \partial_t \Phi \tag{2}$$

where Φ the potential in the aquifer defined as $\Phi = kH(\varphi - \varphi_0)$, φ the piezometric head, S_s the coefficient of specific storage and k the permeability.

For the simple case of a single semi-confined aquifer of height H, with a discharge Q at $x = 0$ and has a constant head φ_0 at infinity, the solution to (2) has been given by Bruggeman (1999) as

$$\varphi - \varphi_0 = \frac{Q\lambda}{4kH} \left[\operatorname{erfc}\left(\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda} \right) \exp\left(\frac{x}{\lambda} \right) - \operatorname{erfc}\left(\frac{xu}{2\sqrt{t}} - \frac{\sqrt{t}}{u\lambda} \right) \exp\left(-\frac{x}{\lambda} \right) \right] \tag{3}$$

with $u = \sqrt{S_s/k}$ and $\operatorname{erfc}(x)$ the complementary error-function.

To predict the excess pore pressures in front of the tunnel face, an estimate of the discharge Q is needed. In a slurry shield this could be obtained from the average infiltration rate of the bentonite, in the same manner as the average infiltration length e was determined. Mohkam (1985) on the other hand reports that the amount of water displaced by the infiltrating slurry is roughly equal to the porosity of the excavated material. This leads to an estimated discharge per unit area of the tunnel

$$q = -nv \tag{4}$$

with n the porosity and v the advance rate of the TBM. In a foam-conditioned EPB machine the amount of water displaced depends on the foam injection rate, but as the amount of foam injected is often of the same order as the porosity of the excavated material, and displaces the pore water originally present, the above relation can also be used to get an estimate of the discharge in this case.

This piezometric head Φ can be used instead of (1) in the wedge stability analysis. In that way the minimal required support pressure, at time t after boring has started, can be calculated, as well as the pore pressures generated in front of the face. If the discharge Q has been estimated correctly, the excess pore pressures predicted by (3) will approach those given by (1) over time.

2.2 *Dissipation during Stand-still*

In those cases where the excess pore pressures do not reach equilibrium shortly after boring has started and a transient flow model is needed to accurately describe the build-up of excess pore pressures, the time required for the excess pore pressures to fully dissipate will also be large. If in an undisturbed boring process the stand-still period, between the boring of two subsequent tunnel rings, is of the same order as the time needed to excavate a tunnel ring, part of the pore pressures generated during the first excavation period will remain when the boring of the next ring is started. To correctly assess the excess pore pressures that remain at the start of boring, a transient groundwater flow model describing the dissipation of excess pore pressures can be used.

If it is assumed that the excess pore pressures in the aquifer have reached equilibrium at the time boring is stopped, the excess pore pressures are initially given by (1). If it is further assumed that soon after boring is stopped a filter cake forms and seals the face, or that the foam significantly lowers the permeability at the face, so that no more water enters or leaves the working chamber, the excess pore water can only flow away from the face. In that case the excess pore pressures remaining at time t after boring has stopped can be estimated from (see Bruggeman 1999)

$$\varphi - \varphi_0 = \frac{\Delta p_p}{2\gamma_w} \left[\operatorname{erfc}\left(\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda} \right) \exp\left(\frac{x}{\lambda} \right) + \operatorname{erfc}\left(-\frac{xu}{2\sqrt{t}} + \frac{\sqrt{t}}{u\lambda} \right) \exp\left(-\frac{x}{\lambda} \right) \right] \tag{5}$$

2.3 *Predicted Pore Pressures at the Face*

Alternating (3) and (5) during boring and stand-still, the time-dependant excess pore pressures in the aquifer resulting from the infiltration of slurry or foam can be predicted. This is illustrated in figure 3, for a fine, silty sand layer with $\lambda = 9$m, $S_s = 7 \cdot 10^{-4}\text{m}^{-1}$,

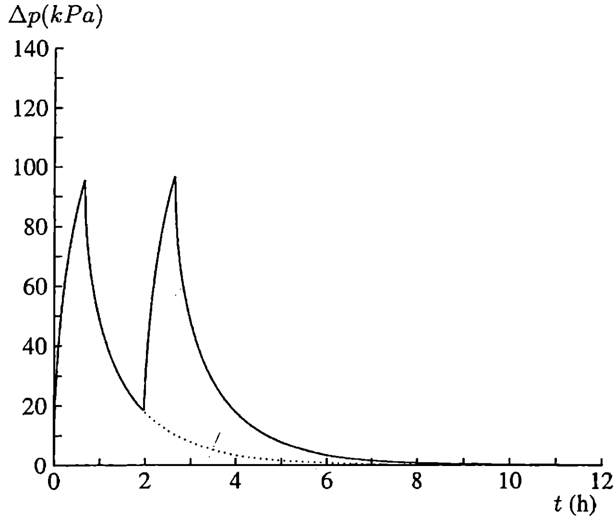

Figure 3. Excess pore pressures generated at the face due to infiltration.

$k = 10^{-5}$m/s and $n = 0.4$. For the TBM an average advance rate $v = 5$cm/min has been chosen and $H/D = 1.33$, with D the diameter of the TBM.

It is assumed that the excavation of a single tunnel ring takes 45 minutes and the stand-still between subsequent excavation periods is 75 minutes. For two consecutive rings the excess pore pressure at the face has been plotted as a function of time. It can be clearly seen that the time needed for the generated excess pore pressures to fully dissipate is longer than the time between subsequent excavation periods. At the start of the second excavation approximately 20% of the generated excess pore pressures remains.

3 LOSS OF SUPPORT MEDIUM

The infiltration and resulting excess pore pressures also have an effect on the maximal allowable support pressure. Generally the maximal allowable support pressure is derived from the passive failure of the soil body at the face, using e.g. the cone shaped failure mechanisms described by Leca & Dormieux (1990). The allowable support pressures derived from such a model are very high.

In case the support medium infiltrates the soil, failure may occur, however, at a lower support pressure. When the pressure in the pore fluid is larger than the contact forces between the grains, a hydraulic fracture can develop in the soil. In that case a flow channel may be created from the excavation chamber of the TBM to the surface by which the support medium can flow off. The effect is almost as sudden as a blow out, although not as explosive, and the loss of support medium through this flow channel can lead to a drop in the support pressure and a subsequent failure of the tunnel face. In cases where this failure mechanism can occur it should be included in determining the maximum

Table 1. Simplified stratigraphy and soil parameters at MQ1, Botlek Rail Tunnel.

Layer	Top (m+NAP)	$\gamma(\gamma_d)$ (kN/m^3)	c (kPa)	φ (°)	k (m/s)
sand fill	+5.0	19(17)	0	30	
clay, silty	-0.4	17	7.5	27.5	
clay, sandy	-2.4	19	15	27.5	10^{-7}
sand, fine	-4.2	19	0	32.5	10^{-6}
clay	-14.0	19	15	27.5	10^{-7}
sand, fine	-14.6	19	0	32.5	10^{-6}
peat, clayey	-17.2	15	10	25	10^{-9}
sand	-17.6	20	0	35	$3\cdot10^{-4}$

allowable support pressure.

From laboratory tests by Bezuijen (1996) it follows that hydraulic fracturing can occur if the excess pore pressure

$$\Delta p \geq \sigma_3' + 2c_u \tag{6}$$

with c_u the undrained shear strength and σ_3' the lowest effective principal stress.

In cases where the TBM is situated in a permeable layer below soft, impermeable layers, it is often not sufficient to check this condition at the crown of the TBM only. The excess pore pressure can build up below the impermeable layers, similar to the air build up leading to blow out described by Babendererde (2000). This can lead to a flow channel being created, initially from the top of the permeable layer to the surface, and thereafter extending and widening through erosion.

To prevent this secondary failure mechanism, it is also necessary to calculate the build up of excess pore pressures at the bottom of the impermeable layers. A similar simplified groundwater flow model as presented in equation (3) can be used for design purposes. In situations where this mechanism may be normative a full three-dimensional groundwater flow calculation could be used for a more detailed analysis, as demonstrated by Bezuijen (2001).

4 BOTLEK RAIL TUNNEL

The Botlek rail tunnel is a 9.65m diameter twin-tunnel bored through Holocene and Pleistocene sand, clay and peat layers in the vicinity of Rotterdam, the Netherlands. It has been constructed using a foam-conditioned earth-pressure balance shield and is of interest as at several points along the tunnel alignment piezometers have been placed in the projected path of the TBM, as part of an extensive research program (COB 2001).

On the south bank of the river Botlek a piezometer is placed in a Holocene sand layer. Above this sand layer lies approximately 9m of soft Holocene clays and sand fills. See table 1 for a simplified overview of the stratigraphy at this location. In the lower Pleistocene sand layer an average head of 0.2m+NAP is measured, and a tidal variation of 0.3m is observed, resulting from

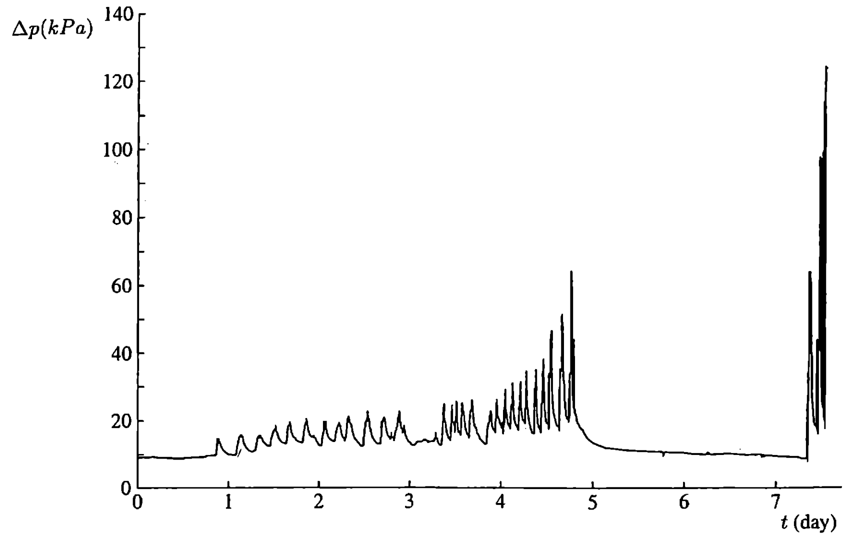

Figure 4. Pore pressure measurements for Botlek Rail Tunnel MQ1 as function of time (BTC/NS-RIB 2000).

the tidal variation observed in the river. The head in the Holocene layers is approximately 3m+NAP and shows no tidal variation. At this location the tunnel axis is 7.4m below NAP and the piezometer is located roughly one metre below the projected tunnel axis.

As the TBM approached the piezometer, excess pore pressures were measured. Measurements made during the last $7\frac{1}{2}$ days these are plotted in figure 4. The peaks correspond with the excavation periods and the subsequent drops with the periods of ring building and maintenance. The measurements made at day 4 show clearly that the excess pore pressures are not fully dissipated as the excavation of the next tunnel ring starts. Only during the stop on day 5 the TBM is halted long enough for the excess pore pressures to fully dissipate.

The excess pore pressure measurements for this weekend stop have been compared to (5) in figure 5. At this time the TBM is halted at x =4.5m before the piezometer; the other parameters are the same as used in the prediction in section 2.3. It is clear that, even though a strongly simplified groundwater flow model is used, the agreement between the predicted and measured dissipation is good. The fact that the excess pore pressures dissipate quicker than predicted is positive, as lower excess pore pressures remain when boring is started. The influence of an improved groundwater flow calculation on the required minimal support pressure will be negligible however.

The measurements are also plotted in figure 6 as a function of the distance between the piezometer and the TBM. In this way they can be compared with the predicted excess pore pressures from (3) at 45 minutes after boring has started, and from (5) 90 minutes after boring has stopped. With regard to the stability calculations the excess pore pressure distribution in the first few metres from the face is most important.

With the excess pore pressure distribution from figure 6 the influence on the required minimal support pressure can be calculated, using the wedge model outlined earlier. Due to the reduction of effective stresses the required effective support pressure s' (the difference between the support pressure and the actual pore pressure) increases to 18kPa, instead of the 9kPa predicted when excess pore pressures are disregarded. As shown the actual pore pressures to be countered also rise, so that a far higher excess support pressure (the support pressure minus the pore pressure at rest) of 81kPa is calculated.

In a slurry shield this increased excess support pressure would lead to a direct increase in the required support pressure. In the closed faced EPB shield used here, part of the support is derived from mechanical support by the cutter head and part from the foam injection. The division between mechanical and foam support could not be determined accurately, however.

5 CONCLUSIONS

During excavation with a slurry shield or a foam-conditioned EPB shield in a medium or fine sand layer, the infiltration of bentonite slurry or foam into the soil will generate excess pore pressures in front of the face. These excess pore pressures lower the effective stresses in the soil as well as the effectiveness of the support medium. This lower the stability of the face and a significantly higher effective support pressure may be required to prevent a face collapse.

In order to calculate the minimal required support pressure, the effect can be included in a wedge stability analysis, using a transient groundwater flow model to predict the build up of excess pore pressures over time. Especially in fine sands a transient flow model is needed to accurately predict the pore pressures in front of the TBM during excavation, as the time required to reach equilibrium flow can be larger than the time needed to excavate a single tunnel ring.

The influence of infiltration and dissipation on the

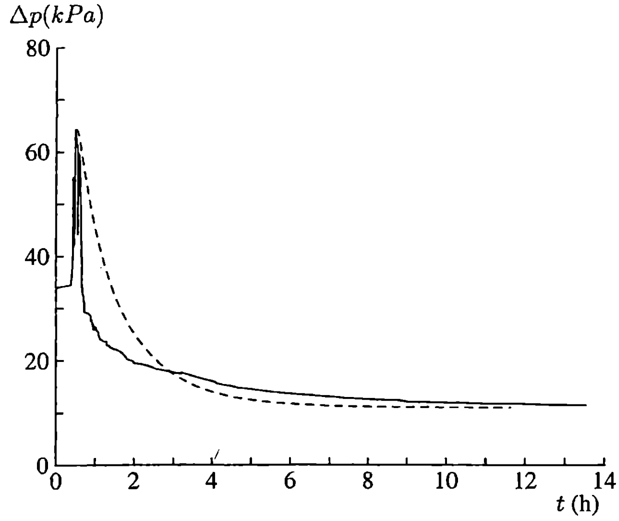

Figure 5. Pore pressure measurements for MQ1, ring 67, compared with (5) for $x = 4.5$m.

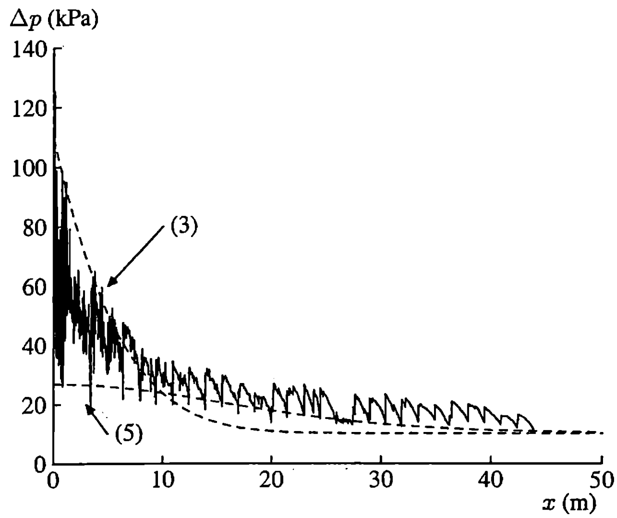

Figure 6. Excess pore pressure profiles according to (3) at $t = 0.75h$ and (5) at $t = 1.5$h, compared with measurements for MQ1.

stability of the face can be predicted using relatively simple transient groundwater flow models. These show that close to the face the excess pore pressures will increase quickly after boring has started and generally take longer to completely dissipate. For a presented case approximately 20% of the excess pore pressure generated remained after 75 minutes. This behaviour was very similar to that observed in field measurements in front of a foam-conditioned earth pressure balance shield in a fine sandy soil.

Excess pore pressures due to infiltration may also lower the maximal allowable support pressure, as hydraulic fracturing can occur at far lower pressures than required for a passive failure of the soil. Hydraulic fracturing can create a flow channel from the excavation chamber to the surface, leading to a loss of support pressure and a subsequent face collapse.

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