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Monitoring and modelling during tunnel construction

Surveillance et modélisation au cours de la construction de tunnel

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ABSTRACT
Tunnelling projects are often technologically challenging projects. Therefore it is not uncommon to perform quite some measurements during the execution of these projects. Measurements are performed to control settlements and/or control of the drilling process. Modern TBMs record all kind of data on the drilling process. This paper shows that analyzing the results of the measurements and modelling with relatively simple calculation models can lead to new insights in the tunnelling process and possible failure mechanism. Examples are presented, investigating the pressure distribution in front of, or at, the tunnel face and back-fill grouting.

RESUME
Les projets des tunnels creusés sont souvent technologiquement des projets de challenge. C'est pourquoi des mesures s'effectuent souvant pendant l'exécution de ces projets. Des mesures sont effectuées pour contrôler les tassements et/ou contrôler le processus de forage. Les tunneliers (TBM) modernes enregistrent tout genre de données durant le processus. Cet article montre que l'analyse des résultats des mesures et la modélisation par des modèles relativement simples peut mener à des nouvelles comprehensions du creusement des tunnels et des mécanismes de rupture possibles. Des exemples sont présentés, étudiant la distribution de pression au devant, ou au front de taille de tunnel et le remblayage par injection.

1 INTRODUCTION

The construction of bored tunnels started only recently in The Netherlands, in the nineties of the last century. Up to then it was expected that the soft soil in The Netherlands was not suitable for a cost effective construction of bored tunnels. With the start of the first bored tunnel projects it was decided to perform monitoring campaigns during each project. This research was initiated by the Dutch Ministry of Transport and Public works and the COB (the Centre for Underground Construction). These campaigns included prediction of the values that can be expected during the monitoring using state of the art calculation models, measuring before, during and after the passage of the TBM and evaluation of the data. This method has proven to be quite effective to acquire knowledge of the processes involved. The paper describes some measurements that led to a new or better description of processes that occur during tunnelling.

Measurements and modelling during 3 tunnel projects will be dealt with: the 2nd Heinenoord Tunnel, the first bored tunnel in The Netherlands, the Botlek Rail Tunnel and the Sophia Rail Tunnel. As will be described in the paper, insight was gained by prediction of the outcome of the measurements or by analyzing the measurements and performing additional laboratory testing. This paper shows some of the measurements and describes briefly the mechanisms involved. A full description of the models used is not possible within the limits of this paper; reference is made in the literature for these models.

2 2ND HEINENOORD TUNNEL, PORE PRESSURES

Excess pore pressures have been predicted and measured in front of the tunnel face during drilling of the 2nd Heinenoord tunnel in saturated sand.

Before these measurements it was generally assumed that the bentonite slurry plasters the tunnel face. This is true after a stand still of several minutes, but not during excavation in saturated sand. The parameters presented in Table 1 were used in the predictions.

Table 1. Soil conditions and slurry parameters during the drilling of 2nd Heinenoord tunnel.

Parameter	Value	Dimension
d_{15}	100	μm
permeability	1.10^{-4}	k/s
porosity	0.41	-
viscosity slurry	$18*10^{-3}$	Kg/(ms)
yield stress slurry	0.01	kPa
face pressure above pore press.	50	kPa

The set-up of the measurements is shown in Figure 1. Some pore pressure gauges are 'eaten' by the TBM. The original function of these pore pressure gauges was to investigate the influence of the cutting elements on the pore pressures in the sand.

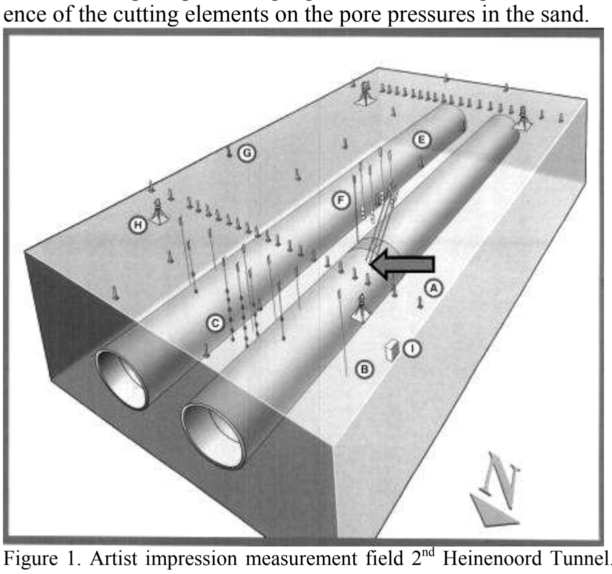

Figure 1. Artist impression measurement field 2nd Heinenoord Tunnel. The arrow indicates the pore pressure gauges in front of the TBM. Results of the gauge in the middle are used this paper. Drilling was from North to South (Bakker et al. 2003).

The predictions showed however, that a penetration depth of 0.05 m is needed for full plastering and that between two passages of the elements (which take about 60 s) there could be no further penetration than 0.015 m, see Bezuijen et al. (2001). The lack of plastering of the tunnel face results in an excess pore pressure in front of the TBM. The course of the excess pore pressure on the tunnel axis was estimated assuming that specific discharge is the same all over the tunnel face. This is an approximation, in reality the discharge will be smaller in the center of the tunnel face compared to the areas further away from the tunnel axis. Although an approximation, it appeared that using the measured excess pressure at the tunnel face, the resulting formula could simulate the course of the excess pore pressure in front to the TBM very well, see Figure 2.It was realized that the measured excess pore pressure can influence the face stability (Bezuijen et al, 2001, Broere, 2001). This result had practical consequences during the construction of the Groene Hart Tunnel (a 15 m diameter tunnel for high speed trains), where at one location a surface load was applied to prevent a blow out (Aime et.al 2004).

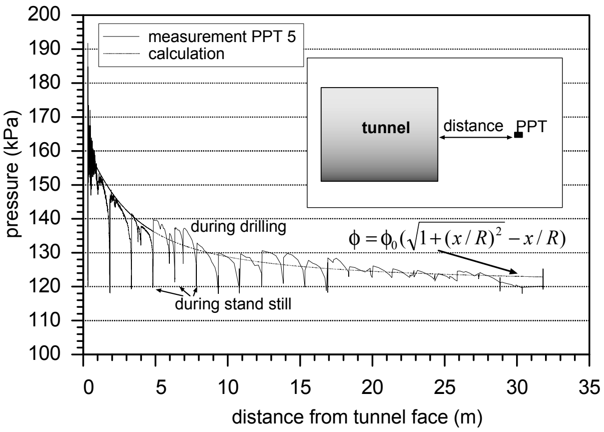

Figure 2. Measured excess pore pressure in front of a slurry shield and approximation.

3 BOTLEK RAIL TUNNEL, TUNNEL FACE EPB

The Botlek Rail Tunnel, the second bored tunnel in The Netherlands, was made with an Earth Pressure Balance (EPB) shield TBM. The principle of such a TBM is shown in Figure 3. The soil is removed from the pressure chamber by a screw conveyor. The pressure drop from a few bars to atmospheric pressure is regulated with the screw conveyor and a valve or pumps at the end of the screw conveyor. The TBM can work without additives in clayey soils, but in sandy soil, as was present at the location of the Botlek Rail tunnel, it is necessary to condition the soil with additives. This is often done with foam. By injection of foam from the cutter head into the soil, the porosity of the sand is increased to a value above the maximum porosity, which facilitates excavation and also reduces the permeability (Bezuijen, 2002). Reduction of the permeability was of importance since the Botlek Rail tunnel passes on its deepest point through permeable Pleistocene sand (k = 3.10^{-4} m/s).

An important aspect for the stability of the tunnel face and the limitation of surface settlements is the average pressure and the pressure distribution at the tunnel face. Therefore this pressure was measured at 9 locations on the pressure bulkhead.

A non-hydrostatic static pressure distribution was measured over the tunnel face where a hydrostatic pressure distribution was predicted based on results measured for a slurry shield, see for an example Figure 4. This figure shows two pressure distributions measured at different times and compares these with two hydrostatic pressure distributions. Clearly there are deviations from the hydrostatic distribution. There are differences between the pressures measured with the instruments on the right side of the TBM compared with pressures measured on the left side. This difference was attributed to the direction of rotation of the cutter head (Bezuijen et al. 2005[b]).

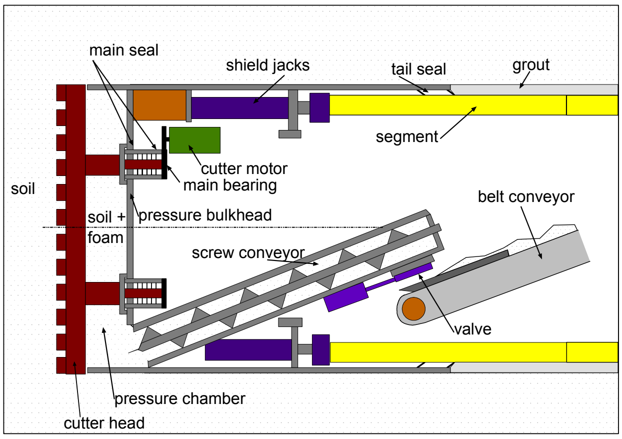

Figure 3. Principle of EPB TBM.

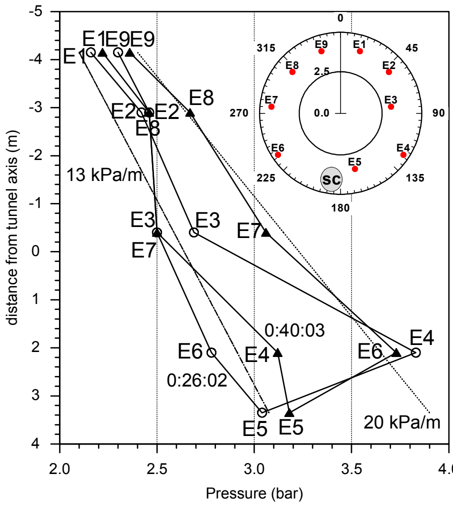

Figure 4. Example of non-hydrostatic pressure distribution measured at the Botlek rail tunnel. The rotation direction of the cutter head was reversed between the 2 measurements shown. The inset shows the position of the instruments and the position of the screw conveyor (SC) in the pressure bulkhead of the TBM.

It was found that the difference in vertical hydraulic gradient is likely to be caused by the yield strength of the muck in the pressure chamber. With no or hardly any yield stress in the muck the pressure distribution at the tunnel face is hydrostatic as was measured for a slurry shield TBM (Bakker et. al, 2003). In presence of cohesion in the muck and adhesion to the TBM the vertical gradient can be written as:

$$\frac{dP}{dx} = \rho_m g \pm 2\frac{\tau_a}{L} \qquad (1)$$

Where P is the pressure, ρ_m the density of the mixture, g the acceleration of gravity, τ_a the adhesion between the muck and the TBM and L the distance between the cutter head and the pressure bulkhead. Depending on the flow direction the pressure gradient can be $2\tau_a/L$ higher or lower than the pressure gradient corresponding to the density of the mixture. In case of a flow with a horizontal component, as can be expected in the pressure chamber between E6 and E5 as well as between E4 and E5, the influence of the adhesion becomes even bigger. Density in the pressure chamber was measured by taking samples through the bulkhead during drilling. The densities found are shown in Figure 5. Laboratory experiments have shown that the adhesion 1 one to a few kPa, with the densities measured and a L of approximately 1 m, this means that pressure gradients from 7 up to more than 20 kPa/m are possible.

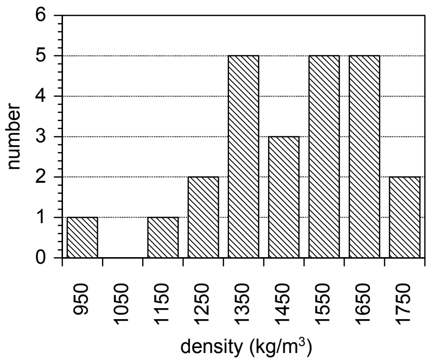

Figure 5. Distribution of muck densities measured in the pressure chamber (24 samples).

Due to the foam injection that increases the porosity to values above the maximum porosity, there are no grain stresses in most of the pressure chamber, but it was found that there can be some grain stress close to and in the entrance of the screw conveyor due to drainage of the muck in that area (Bezuijen et al. 2005[b]). This allows even negative pressure gradients, see Figure 4.

Analysing the measurements showed the influence of adhesion on the pressure distribution and the influence of drainage. These results mean that the pressure distribution on the tunnel face depends on more than the density of the slurry and that changes in this pressure distribution during the drilling process cannot be avoided.

4 SOPHIA RAIL TUNNEL, GROUTING

Another important part in the tunnelling process is the grouting of the tail void to fill up the space between the lining and the soil, see also Figure 3. The quality of the grouting process determines the position of the lining and is of major importance on the surface settlements. To get a better understanding of the grouting process, the grout pressures were measured in 2 rings during the boring of the Sophia Rail Tunnel. Soil conditions are rather uniform along a large part of this tunnel, see Figure 6.

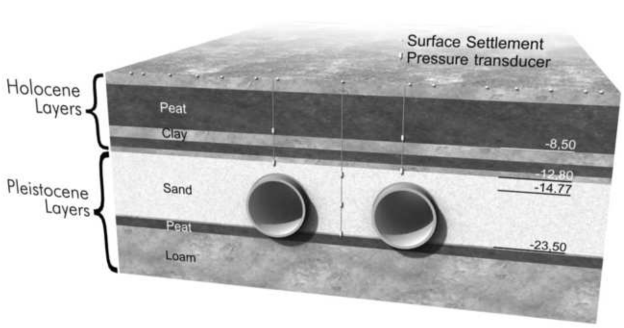

Figure 6. Cross-section of the Sophia Rail Tunnel and soil layering at the measurement location.

In The Netherlands it is usual to prescribe the grouting pressures that have to be applied during the tunnelling in order to avoid excessive surface settlements. Furthermore it was tried to match the grout pressures to the total stress that exist in the soil before tunneling also to minimize settlements.

A calculation model that describes the pressure distribution in the direct vicinity of the TBM was available before the start of the measurements. From calculations with this model it was possible to find the relation between injection strategy, yield stress of the grout mortar and the pressure distribution directly behind the TBM (Talmon et al. 2001). It was further realized that at a certain distance from the TBM the sum of the forces on the lining has to be zero and therefore the average pressure gradient is determined by the weight of the lining, more or less independent from the injection strategy.

A typical result from the grout pressure measurements is shown in Figure 7.

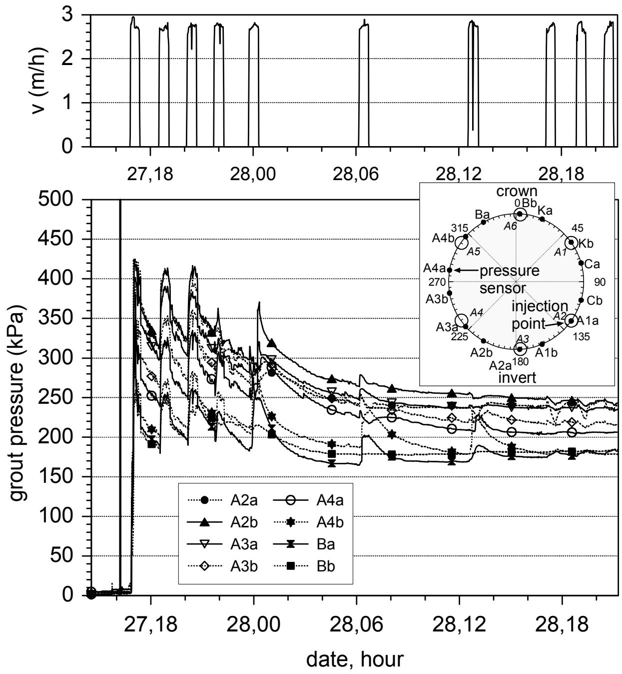

Figure 7. Sophia Rail Tunnel. Measured grout pressures and boring velocity. For clarity only the pressures measured on the left side are shown. The pressures increase during boring and decrease during stand still.

Figure 7 shows the pressure increase during drilling and a decrease during stand still. Conform to the expectations the vertical hydraulic gradient decreases from nearly 19 kPa/m to below 7 kPa/m (Figure 8). This last value is close to the gradient that corresponds to the average weight of the tunnel lining and the auxiliary train (Bezuijen et al. 2004) and the higher gradients measured close to the TBM correspond with the values calculated with the flow model (Talmon et al. 2001, Bezuijen et al. 2004).

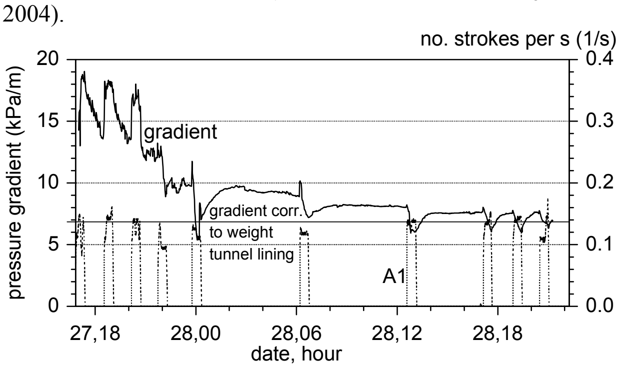

Figure 8. Measured vertical pressure gradient in the grout as a function of time, together with the no. of strokes per second for one of the grout pumps (these pumps are activated during boring).

Analyzing the results, it was noticed, as mentioned already, that the grout pressure increases during boring and decreases during stand still. The reason for this appeared during grout consolidation tests to investigate how the grout mortar behaves under the applied pressure. Grout loses 3 to 10% of its volume when loaded with effective stresses up to 100 kPa. A grout consolidation test was developed to investigate the consolidation of grout, see Figure 9 and Figure 10. A test result is shown in Figure 11.

After pressurizing the vessel the valve is opened and the grout starts to consolidate (sometimes described as bleeding). In the first part of the consolidation process the volume loss increases with the square root of time (Bezuijen &Talmon, 2003), see also Figure 11. This assumption is valid as long as the grain stress close to the impermeable plate is still negligible. When grain stresses develop, leading to a decrease in the measured pore pressure, the consolidation decreases to reach an end value. The time necessary for grout consolidation is in most cases shorter than the time for hardening of the grout and then the consolidation is the dominant process for the increase of the yield stress in the grout over time.

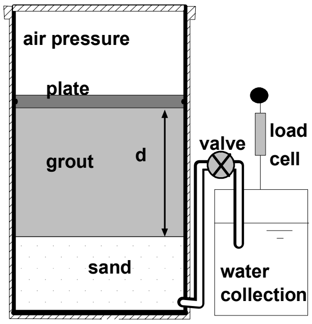

Figure 9. Grout consolidation, measurement principle.

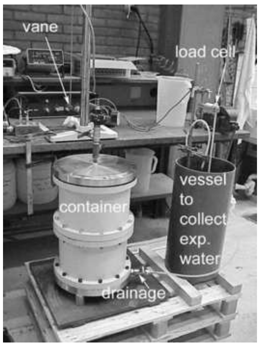

Figure 10. Grout consolidation, experimental setup.

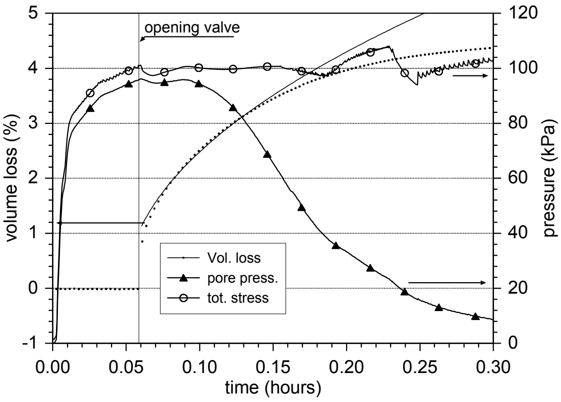

Figure 11. Result of consolidation test. Pore pressure and total pressure are measured on the impermeable plate.

In Figure 7 this decrease in grout pressure during consolidation is shown for one tunnel, but it is measured for a lot of tunnels that are bored in sand (Hashimoto et al., 2004). Consolidation of the grout leads to an unloading of the sand around the tunneland because the sand reacts stiff during unloading, some volume loss leads to a significant pressure drop (Bezuijen & Talmon, 2003). Pressures restore however when drilling recommences. Pressures decay with distance from the TBM. This is caused by fluid loss, and is governed by yield stress and the thickness of the grout cake (Talmon & Bezuijen 2004). The grouting conditions directly behind the TBM are critical with respect to settlements. Here grout pressures and injected volume of grout are the governing parameters. Soil reacts less stiff during loading compared to unloading. Therefore different grout injection volumes lead to the same final grouting pressure, but to different settlements. From this it might be concluded that in controlling surface settlements it is more appropriate to control the volume of injected grout than the grout pressure once the injection pressures are within certain bounds. .

The measured vertical hydraulic gradient also implies a loading on the lining. The longitudinal loading on the lining can be calculated using the beam equation (Bezuijen et al, 2005[a]). Close to the TBM, where the pressure gradient is not yet in equilibrium with the weight of the tunnel, there will be buoyancy forces. It was found that these forces can reach critical values when the length over which the vertical gradient is higher than corresponding to the weight of the tunnel is too high. This can happen when the yield stress of the grout is too low and consolidation or hardening take too much time compared to the progress of the tunneling process.

5 CONCLUSIONS

Several examples were shown where the combination of measurements and analyzing of the results led to new insight in some of the mechanisms of importance during the tunnelling process:

1. Excess pore pressures can occur at the tunnel face during the excavation process. These excess pore pressures decrease the stability of the tunnel face.
2. Pressure gradients at the tunnel face of an EPB are not only determined by the density of the slurry, but also by the yield stress of the slurry and the rotation direction of the cutter blade. This limits the possibilities to control the pressure gradient, which can be of importance when evaluating the soil deformation caused by the tunnel face.
3. The cause of the grout pressures and the grout pressure gradient as measured during tunnelling could be explained. Injection strategy determines the grout pressures just behind the TBM. At a larger distance these are dominated by the weight of the tunnel. Consolidation of the grout leads to a reduction in grout pressures at some distance from the TBM and the longitudinal loading on the lining can be coupled quantitatively to the grout properties.

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