Special lectures

Geotechnical Aspects of Underground Construction in Soft Ground – Ng, Huang & Liu (eds)

Processes around a TBM

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ABSTRACT: Processes that occur around a TBM during tunnelling have been investigated while tunnelling in saturated sand. The pore pressure in front of the TBM increases due to a lack of plastering during drilling. This has consequences for the stability of the tunnel face, or the soil in front of the tunnel. A bentonite flow is likely alongside the TBM from the tunnel face, and/or grout flow from the back. It seems that virtually no investigation has been made of this part of the TBM, but it is important to understand the volume loss that occurs around a tunnel. The lining is constructed behind the TBM and the tail void grout is applied. Pressures measured in the tail void grout will be discussed, as well as the consequences for loading on the soil and the lining. Most of the results described are based on field measurements performed at various tunnels constructed in the Netherlands.

1 INTRODUCTION

Dutch experience of using TBM tunnelling is relatively recent. The first TBM tunnel was constructed in the Netherlands between 1997 and 1999 (the Second Heinenoord Tunnel). In the early 1990s, Dutch engineers were uncertain whether the soft saturated soil in the western parts of their country was suitable for TBM tunnelling. The decision was therefore taken to include a measurement programme in the first tunnelling projects. An overview of this programme and some results are presented by Bakker & Bezuijen (2008). In the programme, results from the measurements were predicted using existing calculation models. The measurement results were analysed at a later date, and discrepancies with the predictions were explained where possible.

An important part of the measurement and analysis programme was dictated by the processes that occur around the TBM. This paper deals with some of these processes. It does not cover all aspects of TBM tunnelling as this would not fit within the limits of this paper (see Bezuijen & van Lottum, 2006, for more information). The paper focuses on certain areas where ideas concerning the mechanisms involved have changed over the last decade, and where a better understanding is now apparent.

In order to structure this paper, we 'walk' along the TBM. We start with a process at the front of the TBM: the creation and stability of the tunnel face under the influence of excess pore pressures. The paper then discusses what happens next to the TBM. The last part of the paper deals with the tail void grout that is injected at the end of the TBM. The paper describes the current state of the art of these processes, and discusses how knowledge gained about these processes may influence the design of a TBM tunnel in soft soil.

2 PORE PRESSURES IN FRONT OF A TBM

2.1 *Flow in coarse and fine granular material*

During TBM tunnelling, it is essential that the tunnel face is stabilised by pressurised slurry (slurry shield) or muck (EPB shield). The pressure must be adapted to the ground pressure to stabilise the front. If pressure is too low, this will lead to an instable tunnel front resulting in collapse of the tunnel face. If pressure is too high, a blow-out will occur. Various calculation methods have been proposed to calculate the stability of the tunnel face. Most of these methods do not take the influence of pore water flow into account. It is assumed that the bentonite slurry or muck at the tunnel face creates a perfect seal that prevents water flow from the face into the soil. Experience with tunnels built in areas where the subsoil contains gravel has shown that the bentonite slurry can penetrate into the subsoil over more than 7 m (Steiner, 1996). Steiner advises that the sand and fines should be retained in the slurry (instead of removing them in the separation plant), and that sawdust should be used in the bentonite (Steiner, 2007). Anagnostou & Kovari (1994) propose a calculation method for such a situation. However, this method only takes the viscous behaviour of the slurry into account, and not the stiffening that occurs during standstill. The results of this calculation method may therefore lead to the prescription of bentonite

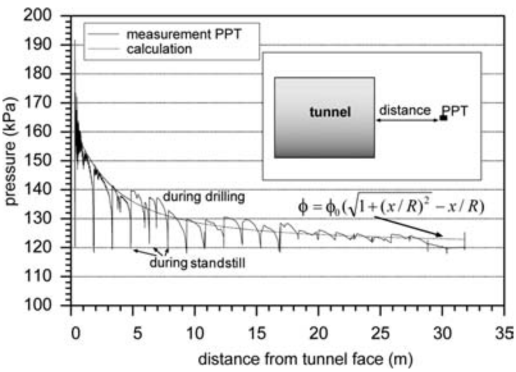

Figure 1. Measured excess pore pressure in front of a slurry shield and approximation.

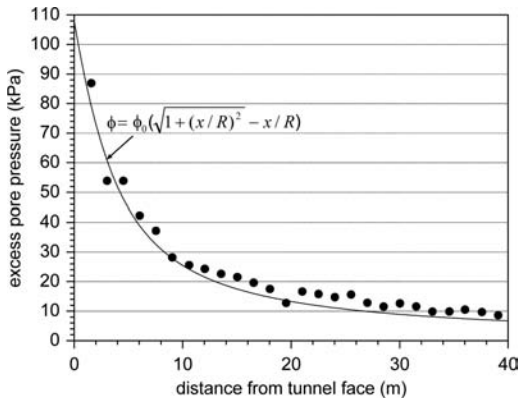

Figure 2. Measured excess pore pressure in front of an EPB shield (●) and approximation (Botlek Rail Tunnel, MQ1 South). Relatively impermeable subsoil.

with viscosity that is too high (Steiner, 2007). The state of the art for such a situation involving coarse granular material is still trial and error, but the trial can be performed in the laboratory to avoid errors in the field.

Usual tunnelling conditions in the Netherlands are a saturated sandy soil in medium-fine sand. In such soil conditions, the groundwater flow influences the plastering. There will be virtually no plastering of the tunnel face by the bentonite or the muck during drilling, because the groundwater in front of the TBM prevents water in the bentonite slurry or muck flowing into the soil. Plastering will only occur during standstill of the TBM process.

Figure 1 shows measured pore pressure in front of a slurry shield as a function of the distance from the TBM front. Plastering occurs during standstill, resulting in a pressure of 120 kPa (the hydrostatic pressure). Higher pore pressures were measured during drilling, because the TBM's cutter head removes a cake before it can form at the tunnel face.

Figure 2 shows the same phenomenon measured in front of an EPB shield. Here, only the pressure during drilling was recorded.

Bezuijen (2002) shows that the amount of excess pore pressure measured in the soil in front of the TBM (apart from pressure at the tunnel face) also depends on soil permeability, the quality of the bentonite or muck, and the drilling speed. Where EPB drilling takes place in sand with a low permeability ($k = 10^{-5}$ m/s), the pore pressure measured in sand in front of the TBM is virtually equal to pressure in the mixing chamber. The pressure is lower in sand with higher permeability ($k = 3 \cdot 10^{-4}$ m/s), because some plastering of the face occurs during drilling. Soil permeability also influences the foam properties. Muck in the mixing chamber will be dryer in sand with a higher permeability. Where the permeability of the sand is lower, the water content in the muck is nearly entirely determined by water in the soil and much less by the foam properties (also see Bezuijen, 2002).

Figure 1 and Figure 2 also show a theoretical curve (Bezuijen, 2002):

$$\phi = \phi_0(\sqrt{1+(x/R)^2} - x/R) \tag{1}$$

Where ϕ_0 is the piezometric head at the tunnel face, ϕ the piezometric head at a distance x in front of the tunnel face, and R the radius of the tunnel. This relationship is valid for situations where the permeability of soil around the tunnel is constant. In the Netherlands, the sandy layers used for tunnelling are sometimes overlain with soft soil layers of peat and clay with a low permeability. In such a situation, the pressure distribution in the soil can be evaluated as a semi-confined aquifer. This is described by Broere (2001).

2.2 *Influence on stability*

Bezuijen et al (2001) and Broere (2001) have shown that the groundwater flow in front of the TBM implies that a larger face pressure is necessary to achieve a stable front. According to Bezuijen et al (2001), the difference is approximately 20 kPa for a 10-m-diameter tunnel constructed in sand, where the top is situated 15 m below the ground surface.

Knowledge of this groundwater flow appeared essential during the Groene Hart Tunnel (GHT) project, not to prevent collapse of the tunnel face but to prevent a form of blow-out (Bezuijen et al, 2001). This tunnel enters a deep polder where the piezometric head in the sand layers underneath the soft soil layers is higher than the surface level (see Figure 3). As a result, the effective stresses beneath the soft soil layers are extremely small. The calculated excess pore pressure

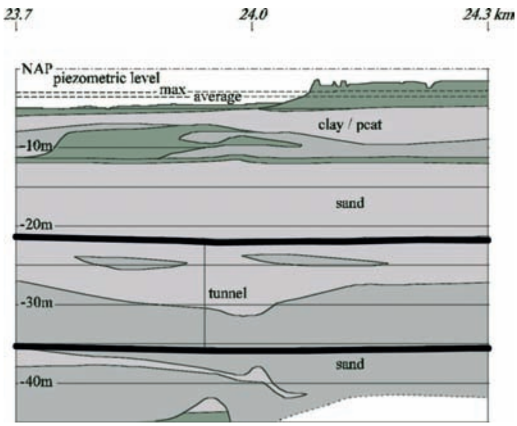

Figure 3. Geotechnical profile GHT tunnel in polder. Tunnel is drilled from right to left in this picture.

in the sand layer induced by the tunnelling process could cause 'floating' of the soft layers. The contractor made detailed numerical calculations (Aime et al, 2004). As a result of these calculations, a temporary sand dam was constructed at the point where the tunnel entered the polder. This dam delivered the necessary weight to prevent lifting of the soft soil layers due to excess pore pressure generated at the tunnel face during drilling.

3 FLOW AROUND THE TBM

3.1 *Calculation model*

Until recently, only limited attention has been given to pressure distribution and flow around the TBM shield. It was assumed that the soil was in contact with the TBM shield across the shield. During drilling of the Western Scheldt tunnel, however, it appeared that the TBM deformed at large depths and high water pressures (the tunnel is constructed up to 60 m below the water line). This could not be explained by the concept of a TBM shield in contact with the soil. Furthermore, tunnelling technology has advanced to a level where the ground loss due to tunnelling is less than the volume difference caused by tapering of the TBM. TBMs are usually tapered, with a slightly larger diameter at the head compared with the tail. This allows the TBM to manoeuvre and to drill with a certain curvature. Table 1 shows the volume difference due to tapering for different TBMs.

The volume losses measured during these projects varied, but negative volume losses were sometimes measured in all the projects (there was actually heave). It is clear that the measured volume loss can be less than the volume loss due to tapering. This leads to

Table 1. Percentage of tapering of the TBM in 3 tunnel projects in The Netherlands.

Tunnel project	Tapering %
Second Heinenoord	0.95
Botlek	0.77
Sophia	0.79

the idea (Bezuijen, 2007) that the soil is not in contact with the TBM all over the TBM. Overcutting at the tunnel face can lead to bentonite flow over the TBM shield from the face towards the tail. Grout pressure during grout injection is usually higher at the tail than the soil pressure. The soil is therefore pushed away from the TBM, and grout will flow from the tail over the shield. It is possible to describe flow on the shield, if it is assumed that both the bentonite and the grout are Bingham liquids, that the yield stress is dominant in the flow behaviour, and that there is linear elastic soil behaviour. A more or less conceptual model is developed, assuming a cylindrical symmetrical situation around the tunnel axis. Changes in the soil radius for such a situation can be described as (Verruijt, 1993):

$$\Delta\sigma = 2\frac{\Delta r}{r}G \qquad (2)$$

Where $\Delta\sigma$ is the change in pressure, Δr the change in radius, r the radius of the tunnel and the grout, and G the shear modulus of the soil around the tunnel.

The flow around the TBM shield can be described as:

$$\Delta P = \alpha\frac{\Delta x}{s}\tau_{\gamma} \qquad (3)$$

Where ΔP is the change in pressure due to the flow, Δx a length increment along the TBM, s the gap width between the tunnel and the soil, and τ_γ the yield stress of the grout around the TBM. α is a coefficient indicating whether there is friction between the soil or bentonite and the grout ($\alpha = 1$) only, or also between the TBM and the grout or bentonite ($\alpha = 2$). Viscous forces are neglected in this formula. This is permissible due to the low flow velocities that can be expected.

With no grout or bentonite flow around the TBM, tapering will lead to an effective stress reduction proceeding from the TBM's face to the tail according to equation (2). The grout and bentonite flow will change this pressure distribution. In order to calculate the pressure distribution under flow, the flow direction of both the bentonite and the grout must be known. These flow directions can vary during the tunnelling process (Bezuijen, 2007). On average, however, the TBM

Table 2. Input parameters used in calculation with bentonite and overcutting.

Length TBM shield	5	m
Diameter	10	m
Diameter reduction	0.2	%
Overcutting	0.015	m
Asymmetric (1) or symmetric (2)	2	
Grain stress	150	kPa
Grout pressure	400	kPa
Pore pressure	200	kPa
Pressure on tunnel face	250	kPa
Shear modulus (G)	90	MPa
Yield stress grout	1.6	kPa
Yield stress bentonite	0.01	kPa

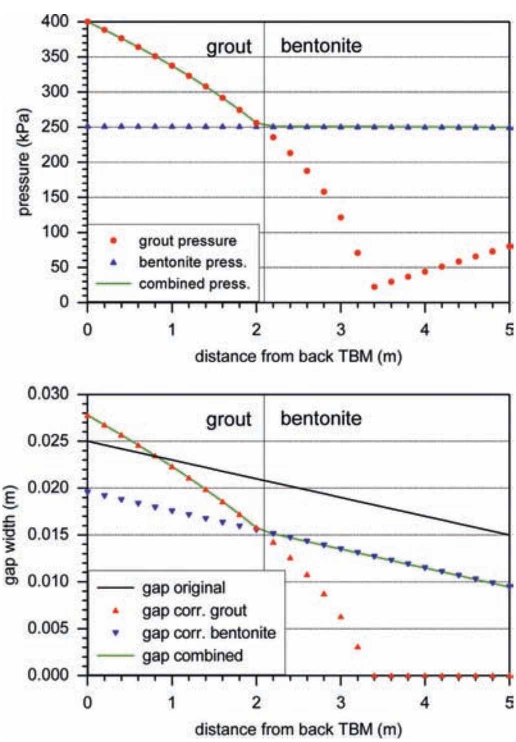

Figure 4. Pressures and gap width along a TBM. Grout pressures and bentonite pressures. Parameters see Table 2. Plots show pressures and gap width for the bentonite and grout pressure separately and the combined result.

advances and therefore the bentonite and grout front must also advance in the same direction to achieve a stable situation. This means that grout and bentonite only move with respect to the soil, and not with respect to the TBM. Therefore $\alpha = 1$ for both the bentonite and the grout. The result of an example calculation using the parameters given in Table 2 is shown in Figure 4.

The Figure shows that the gap width for a completely stiff soil mass would increase from 0.015 m at the front to 0.025 m at the tail of the TBM. If there were only grout pressures, the gap width would be 0.028 m at the tail of the TBM, due to the grout pressure that is larger than the total stress. However, the gap would close at 3.4 m from the tail. If the influence of the bentonite is included, there is still a gap width of 0.01 m at the tunnel face (5 m from the tail). The line through the triangles presents the gap width due to the combined effects of both the bentonite and the grout. The plot above presents the pressures in the same way.

3.2 *Consequences and status*

The model shows that the volume loss is not determined by tapering of the TBM (as suggested for example by Kasper & Meschke, 2006), but is influenced by the pressure distribution of the bentonite and grout. With sufficient grout pressure, it is possible to have a 'negative' volume loss (the surface level rises after the TBM passes). It also explains that bentonite is sometimes found in the tail void, and grout is found in the pressure chamber. The first situation occurs when bentonite pressure is relatively high and grout pressure is low (we will see that it is quite difficult to control grout pressure, especially during ring building). The second situation occurs when grout pressures in the tail void are relatively high (which may occur during drilling).

Contrary, however, to the model described for the pore pressures in front of the TBM and the grout pressure, to be described in the following sections, the experimental evidence for this model is still limited. To our knowledge, pressure distribution around the TBM shield has never been measured. The shield was perforated during construction of the Western Scheldt tunnel but no grout was found between the shield and the soil (Thewes, 2007). The fact that no grout was found during this investigation may be caused by the fact that, in reality, the TBM will not be placed as symmetrically in the drilled hole as suggested in this simple model. The TBM must be in contact with the soil at some point to maintain mechanical equilibrium. There will be no grout around the shield at that location.

Guglielmetti (2007) rightfully argues that more research is needed in this field, because: 'The topic (flow of bentonite and grout around the TBM) is definitely one of the most important in the field of mechanised tunnelling, being the management of the void around the shield of a TBM as one of the major sources of concern for both designers and contractors involved in urban tunnelling projects'.

There is some evidence from the results of extensometer measurements carried out at the Sophia Rail Tunnel. The results of the extensometers (shown in Figure 5) are presented in Figure 6 during passage of

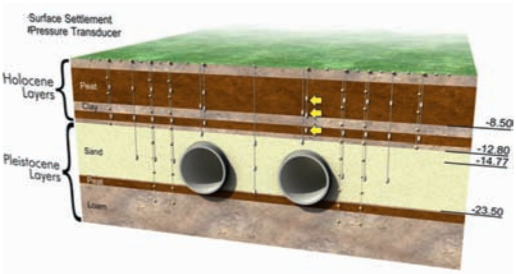

Figure 5. Sophia Rail Tunnel, soil stratification and location of extensometers at the measurement location (picture Arne Bezuijen).

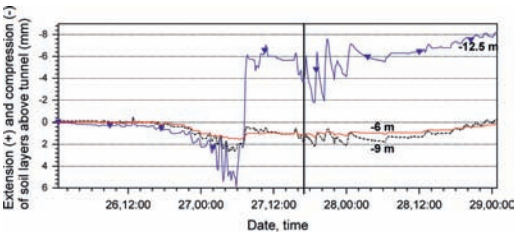

Figure 6. Extensometer results. The vertical line shows when the tail of the TBM passes. Soil above the TBM is already compressed before the tail passes.

the TBM. The results show that there is initially some extension of the soil in front of the TBM due to the relatively low stresses at the tunnel face. However, the soil above the tunnel (see the extensometer at −12.5 m) is compressed several rings before the tail of the TBM passes (the vertical line) indicating heave, and there is therefore no settlement due to the tapering. When the TBM has passed, the extensometer at −12.5 m follows the course of the grout pressures measured around the lining. This will be discussed in more detail in the next section, and shows that a change in grout pressure indeed leads to a change in soil deformation.

We are currently working on the possibility of measuring pressures around the shield.

4 TAIL VOID GROUTING

4.1 *Introduction tail void grouting*

Coming at the end of the TBM, the tail void grouting process is important. The process determines the loading on the soil and on the lining.

The pressure distribution caused by tail void grouting has been studied during construction of the Sophia Rail Tunnel (Bezuijen et al, 2004) and the Groene Hart Tunnel. Here, we will describe the fundamental mechanisms using measurements from the Sophia Rail

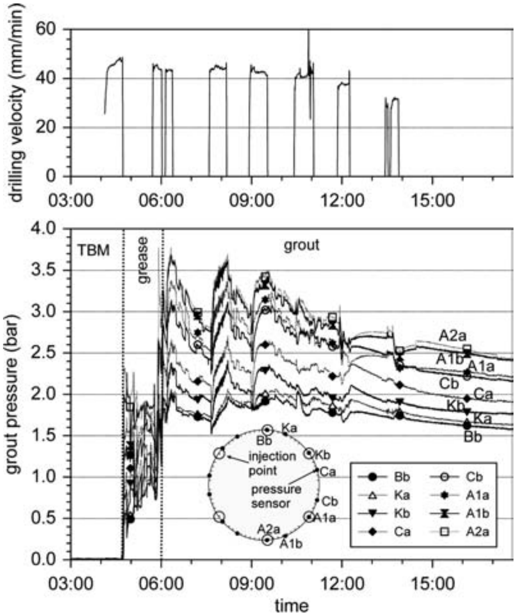

Figure 7. First tube Sophia Rail Tunnel: drilling velocity and measured grout pressures at the right side of the tunnel as a function of time.

Tunnel, as they have provided the most complete data set until now.

The study of grout pressures was initiated by earlier measurements performed at the Second Heinenoord Tunnel and the Botlek Rail Tunnel. These measurements did not match the generally accepted assumption at that time – at least in the Netherlands – that the vertical pressure gradient in liquid grout must be dictated by the density of the grout, and that the pressure distribution after hardening must reflect the K_0 (the ratio between the horizontal and vertical soil pressure). In reality, the vertical pressure gradient was lower and the influence of K_0 could not be detected.

4.2 *Measurements*

The Sophia Rail Tunnel was constructed in sandy subsoil overlain with soft soil layers (see Figure 5). The water table is close to the surface. During construction of the Sophia Rail tunnel, two rings in the lining were each equipped with 14 pressure sensors. The pressures measured with one of these instrumented rings are shown in Figure 7.

These measurements are discussed in detail in Bezuijen et al (2004): we will only describe the main phenomena here. The upper plot in Figure 7 shows the drilling velocity, when drilling occurs, and when there was a standstill for ring building. It can be seen that an increase in pressure is measured as soon as the pressure gauges (built into the lining elements) moved from the grease into the grout. Pressure increases as long as drilling continues, and decreases when drilling stops during ring building.

4.3 *Grout pressures*

The mechanism that leads to these pressure variations is explained in Bezuijen & Talmon (2003). Grout bleeding or consolidation of the grout leads to a volume loss of grout. Experiments showed that this volume loss is between 3% and 8%, depending on the type of grout (Bezuijen & Zon, 2007). This consolidation leads to stress reduction in the relatively stiff sand layer. This stress reduction is measured as a reduction of grout pressure. The effective stresses will ultimately be very small: the minimum stress that is necessary to keep the hole in the ground open. Leca & Dormieux (1990) calculate this for a tunnel opening in sand. They calculate that a cylindrical cavity in the ground remains open when effective stresses of only a few kPa are applied.

The consequence is that grout pressures around the lining will decrease to values that are only a few kPa above the pore water pressure. It is therefore clear that the original K_0 can no longer be found in the grout pressures. The pressure decrease due to volume loss in the grout has changed the original stress state, and unloading of the soil leads to much lower stresses. Since the stresses in the sand around the tunnel decrease, the sand reaction will be the reaction of a very stiff material. Only a small volume decrease in the grout will lead to a large decrease in stresses. Calculation methods quite often still use the original in-situ stresses to calculate loading on the lining. For a tunnel in sand, this leads to a calculated loading that is much too high, as shown by Hashimoto et al (2004).

For slow hardening or non-hardening grouts, the strength increase in the grout is caused by grout bleeding or consolidation. It should be realised that this strength increase is only present when the tunnel is drilled through a permeable soil. When drilling takes place through less permeable soils such as clay, this consolidation will be much lower and the grout will be in liquid form over a greater part of the tunnel's length. This has consequences for loading on the lining, as we will discuss later.

4.4 *Pressure gradients*

The vertical pressure gradient over the tunnel lining is important when calculating the longitudinal loading on the lining. The vertical pressure gradient that was measured during construction of the first tunnel tube of the Sophia Rail Tunnel is shown in Figure 8. The pressure gradient starts at nearly 20 kPa/m and decreases to values under the pore water pressure gradient of

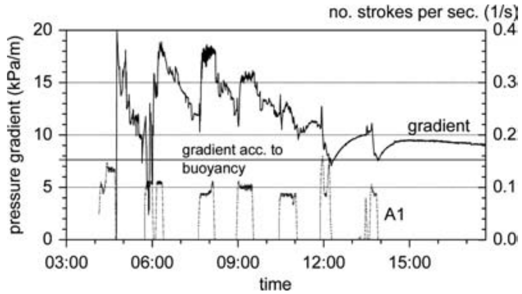

Figure 8. First tube Sophia Rail Tunnel: pressure gradient over the tunnel lining at one location, and pump activity for one of the injection points (A1) as a function of time.

9.81 kPa/m. The tail void grout used for this tunnel had a density of 2190 kg/m^3. If the vertical pressure were to increase with depth in accordance with this density, the pressure gradient should be 21.5 kPa/m. Results showed that the measured vertical density is always lower. This is because the grout is a Bingham liquid, with a viscosity and a yield stress. The grout has to flow downwards if more grout is injected in the upper half of the tunnel. This downward flow needs a driving force to overcome the yield stress, and the pressure gradient will therefore be less than the gradient that is calculated from the density. Talmon et al (2001) developed a numerical program to calculate the pressure distribution in the tail void due to injection. We only describe some of the consequences here. If the viscosity is not taken into consideration, the maximum pressure gradient (*dP/dz*) that can be expected is:

$$\frac{dP}{dz} = \rho_{gr} g - 2\frac{\tau_y}{s} \tag{4}$$

Where ρ_{gr} is the density of the grout, g the acceleration of gravity, τ_y the yield strength of the grout, and s the width of the tail void gap between the soil and the lining. If the yield stress in the grout is low, the vertical pressure gradient is determined by the grout density (21.5 kPa/m for the Sophia Rail Tunnel, slightly higher than the maximum value measured in Figure 8). Consolidation or hardening of the grout leads to a higher yield stress, and thus to a lower gradient.

A complicating factor is that the maximum shear stress that can be developed is a vector. If the maximum shear stress is developed in one direction, there will be no shear stress perpendicular to that direction. When drilling starts for a new ring and the grout pumps are activated, the elastic soil reaction will lead to an increase of the tail void and grout will therefore flow backwards from the TBM. Ring shear stresses barely develop in this situation, and the vertical gradients therefore increase during drilling. They decrease again when drilling stops (Figure 8).

Further from the TBM, the vertical gradients decrease and become equal to the gradient according to the buoyancy forces. This has to be the case, because the total force on the lining far away from the TBM must be zero. The vertical pressure gradient therefore compensates for the weight of the lining. As a result, the gradient becomes lower than the gradient in the pore water. This is because the average density of the lining is lower than the density of pore water. One remarkable result is that the vertical pressure gradient at some distance from the TBM (at 12:00 in Figure 8, 5 rings behind the TBM) decreases during drilling. The flow no longer has any influence at this point, but drilling and grout injection lead to higher gradients in the first part of the lining and therefore to higher buoyancy forces. The first rings have the tendency to move upwards, which must be compensated by the TBM and the rings further away. This partly compensates for the weight of the rings further from the TBM, so that the effective weight of these rings and also the vertical gradient is less.

5 INFLUENCE ON PORE WATER PRESSURES

Section 2.1 describes how no plastering occurs at the front when drilling takes place in fine to medium-fine saturated sand, because the bentonite filter cake is destroyed by the cutting wheel before it is able to form. As a result, water flows from the tunnel face into the soil. Section 4.3 describes how consolidation of the grout also leads to a water flow from the tunnel lining into the soil, because water expelled from the grout will flow into the surrounding soil. A grout cake will form however, because the consolidated grout is no longer disturbed. It is therefore reasonable to assume that examination of the variation in pore pressure in soil next to a tunnel under construction will show pore pressures that are dominated by pressures existing at the tunnel face. This theory was tested at the Groene Hart Tunnel. Pore pressure transducers (PTTs) were installed as close as 0.75 m from the tunnel lining. The PTTs were placed in one plane, with the grout pressure gauges on Ring 2117 of the tunnel (see Figure 9).

Figure 10 shows the measurement results. The grout pressure gauges on Ring 2117 give no signal before they are in the grout. The PPTs show a slight increase during drilling due to the excess pore pressure generated at the tunnel face. As drilling stops, the pore pressure reduces to the hydrostatic pressure. The various construction cycles can be seen. There is a sharp increase in grout pressure when Ring 2117 leaves the TBM, followed by a decrease due to consolidation. It is remarkable however that this has virtually no influence on the measured pore pressures at less than a metre from these gauges. This result is confirmed by numerical calculations. The quantity of water expelled

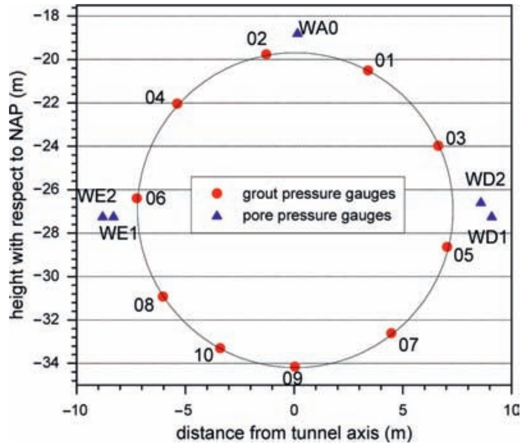

Figure 9. Position of pore pressure gauges and grout pressure gauges at ring 2117 of the GHT.

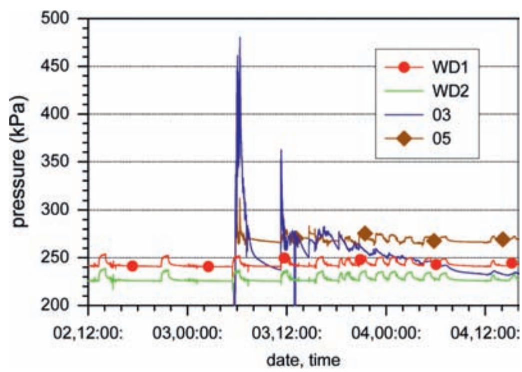

Figure 10. Pore pressures and grout pressures measured at GHT (also see text).

from the grout is far less than the water flow from the tunnel face. The latter dominates the pore pressures.

The measurements show another remarkable feature. Grout pressure gauge 05 follows the water pressure after 3.20:00, but this is not the case for gauge 03. This may indicate that there is no 'sealing' grout layer around gauge 05, so that it is possible to measure the pore water pressure.

6 LOADING ON TUNNEL LINING

We have seen in Section 4.4 that vertical pressure gradients exist in the zone where the grout is not yet consolidated or hardened which are higher than corresponds to the weight of the lining. Measurements at the Sophia Rail Tunnel showed that the gradient decreases more or less linear with the distance (see Figure 11). As a result, that part of the lining is pressed upwards by

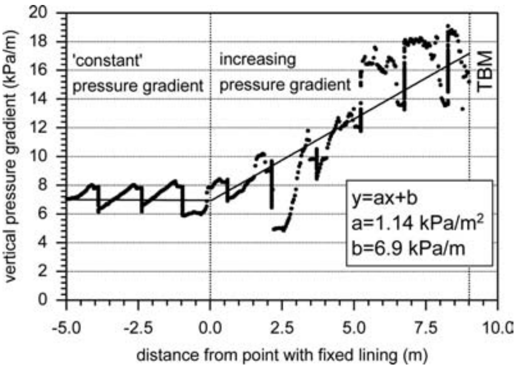

Figure 11. Example of gradient in the grout pressure as a function from the distance (0 on the X-axis represents the point where the lining is more or less fixed. The TBM is at 9 m). Results from Sophia Rail Tunnel (Bezuijen et al, 2004).

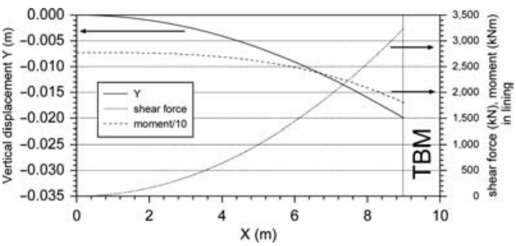

Figure 12. Calculated shear force and moment in the lining, and displacement where the grout has not yet hardened. Calculated moments are divided by 10.

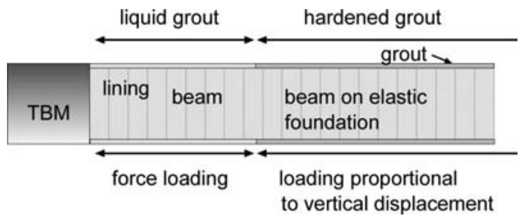

Figure 13. Boundary condition for beam calculation.

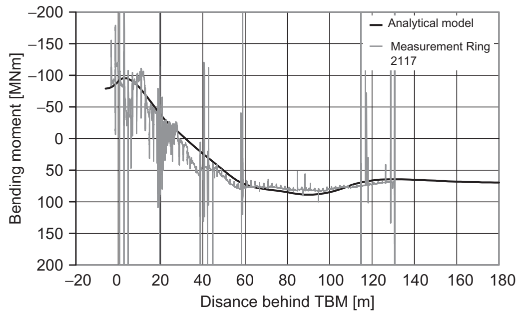

Figure 14. Bending moment ring 2117, measurement and calculation. Groene Hart Tunnel (Hoefsloot, 2008).

the buoyancy forces. It is necessary to mobilise shear forces from the TBM to achieve a stable tunnel lining. This will lead to moments in the lining.

Bezuijen & Talmon (2005) have shown that the moments in the liquid grout zone increase backwards from the TBM (see Figure 12). A positive moment means here that the force on the lower part of the tube is higher than on the upper part. At the TBM, this moment is created by the TBM itself. This is because face pressure is higher at the bottom due to larger soil stresses.

At the Groene Hart Tunnel the bending moment in the lining was measured for a large distance behind the TBM using strain gauges installed in the lining segments. There is an increase in the moment for a few rings, in accordance with the calculations previously mentioned. There is subsequently a decrease, with the moments becoming negative at a greater distance from the tunnel. Bogaards & Bakker (1999) and Hoefsloot (2008) argue that the remaining bending moment is a result of the staged construction of the tunnel. They developed a calculation model to take into account the different stages in construction.

However, Talmon (2007) has shown that such a 'staged' calculation is not necessary to find the same results.

According to Talmon, the negative moment appears at some distance from the TBM because the reaction force to compensate the buoyancy in the fluid grout zone is situated further from the TBM than the buoyancy force itself. The tunnel lining is 'pushed' a bit higher in the soil than in the equilibrium situation far behind the TBM.

Hoefsloot and Talmon both model the tunnel lining as a beam on an elastic foundation, except for lining elements inside the TBM and lining elements in the liquid grout zone, see Figure 13. The exact boundary conditions and the transition between liquid and solid grout are still the subject of debate.

Although example calculations have been presented that show good correlation with measurements (see Figure 14), there are still uncertainties with this type of calculation that need further research:

– An important input parameter is the moment and shear force that is transferred from the TBM to the lining. While the moment can be derived from the jack forces, the shear force is not determined.
– With generally-accepted parameters for the lining stiffness and the soil's elastic parameters, the calculated movement of the lining is much smaller than the measured movement.
– The grout pressures are only measured when the grout is more or less in the liquid phase. This results

Table 3. Specification of grout mixtures used in fracture tests (WCR = water-cement ratio). Coclay D90 Ca activated bentonite is used.

Mixture	WCR	Bentonite %	k (m/s)
1	1	7	5.10^{-8}
2	10	7	6.10^{-0}

in loading on the tunnel lining as shown in Figure 11. However, loading on the lining in situations where the grout has hardened is less known. This is because the instruments used were not suitable to measure pressures when grout has hardened.

Conclusions that can be drawn from this type of calculations are:

– The length of the liquid grout zone and the density of the grout are extremely important parameters when calculating bending moments in the lining. If this length is too long, loading will be too high and tunnelling will not be possible (also see Bezuijen & Talmon, 2005).
– The shear force that is exerted on the lining by the TBM is an important parameter. It is therefore worthwhile to measure this shear force.

7 COMPENSATION GROUTING

Grout consolidation also appeared to be important when describing compensation grouting. Experiments (Gafar et al, 2008) showed that the fracturing behaviour in compensation grouting depends on the specification of the grout. If more cement is added, the permeability of the grout is higher and there will be more consolidation and leak-off during grout injection. Gafar et al describe how this influences the fracturing behaviour. Recent tests carried out as part of the research project on compensation grouting present proof of the suggested grout consolidation mechanism. At Delft University, the density of grout bodies made in two compensation grouting experiments was analysed in a CT-scan. Such a CT-scan can be used to determine the density of the material tested. The grout mixtures used in the experiments are shown in Table 3.

The results of the CT-scans are shown in Figure 15 and Figure 16. The results of the first grout mixture clearly show an increase in density at the boundary of the grout body. Grout at the boundary of the sample is consolidated. The grout body made with the second mixture has a more constant density across the fracture (the middle section). In the second experiment, the CT-scan was performed while the grout body was still in the sand. The more homogeneous density of

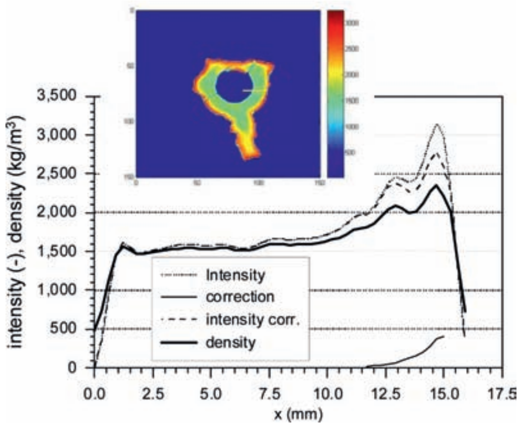

Figure 15. Density measured with a CT-scan. Raw data (inset) and density. Correction for beam hardening effect and calculated value of the density of the grout along the line shown in inset. Mixture 1 in Table 3.

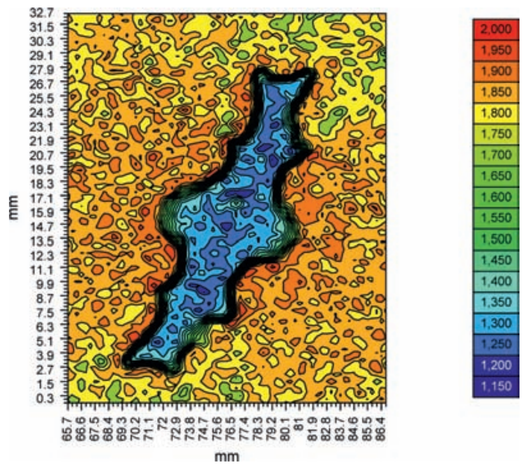

Figure 16. Grout density in a fracture measured with a CT-scan. Mixture 2 in Table 3.

the grout body in the second test is understandable if the permeabilities of the grout are considered. The lower permeability of the second grout sample results in much less grout consolidation within the limited injection time. The grout density in the fracture therefore does not increase at the boundary of the grout as is the case for mixture 1.

The permeabilities were determined using the procedure suggested by McKinley and Bolton (1999), a form of oedometer test with drainage on one side. This procedure can also be used to test the consolidation properties of tail void grout. However, the thickness of the grout layer in the test should be identical to that in the field. This is to avoid scaling effects that occur because hardening of the grout is independent of the sample size (Bezuijen & Zon, 2007).

8 DISCUSSION

The research described above has increased understanding of the processes that occur around a TBM during tunnelling. This has already had consequences for practical aspects of tunnelling. Examples are the excess pore pressures in front of the TBM: extra sand was added locally above the planned tunnel trajectory of the Groene Hart Tunnel to prevent a blow-out, and the grout was changed in a tunnel project in London where it appeared that the liquid zone of traditional grout for a tunnel drilled in clay with no possibility of consolidation was too long to achieve the desired drilling speed. However, the authors believe that the results can make an even greater contribution to improving shield tunnelling. Knowledge about the influence of excess pore pressures on face stability can improve definition of the pressure window at the tunnel face, so preventing a blow-out due to excessively high pressures and instability caused by pressures that are too low. In combination with research on EPB tunnelling in clay (Merrit & Mair, 2006), foam research for EPB tunnelling in sand can lead to better control of the EPB process. It has already been discussed how flow around the TBM is important for TBM design, and that more experimental evidence is needed. Research into grouting can lead to smaller settlement troughs and optimisation of loading on the lining. This last aspect may lead to cheaper lining construction.

The results must be discussed with tunnel builders and contractors if improvements to the shield tunnelling process are to be achieved. Discussion about certain aspects has already started, but we hope that this paper will stimulate the involvement of more parties.

9 CONCLUSIONS

To understand the processes that are important when tunnelling with a TBM, the flow processes around a TBM must be considered: groundwater flow at the tunnel face, bentonite and grout flow around the TBM, and grout flow and grout consolidation around the tunnel lining. The research described in this paper has brought about progress with regard to these flow processes during tunnelling in soft ground:

- The groundwater flow at the tunnel face is described.
- The muck in the mixing chamber is described as a function of drilling speed and permeability.
- A conceptual model for the flow of bentonite and grout has been developed. Although this model must still be verified using the results of measurements, it shows some promising results.
- Considerable information has been obtained about the grouting process and the resultant lining loading.

Although not unusual, it is interesting to see that this research also raises new questions: what is the exact position of the TBM during the tunnelling process, what is the interaction between the TBM and the lining, are the predicted pressures around the TBM correct, and what are the consequences for our design methods? Even in a relatively simple beam calculation for calculating loading on the lining in a longitudinal direction it appears that uncertainties in the boundary conditions determine the outcome of the calculation. As long as these uncertainties remain, more sophisticated numerical calculations will present the same uncertainties.

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REFERENCES

Aime, R., Aristaghes, P., Autuori, P. & Minec, S. 2004. 15 m Diameter Tunneling under Netherlands Polders. *Proc. Underground Space for Sustainable Urban Development (ITA Singapore), Elsevier.*

Anagnostou, G. & Kovári, K. 1994. The face stability of Slurry-shield-driven Tunnels. *Tunelling and Underground Space Technology* 9(2): 165–174.

Bakker, K.J. & Bezuijen, A. 2008. 10 years of bored tunnels in the Netherlands. *Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

Bezuijen, A. 2002. The influence of soil permeability on the properties of a foam mixture in a TBM. *3rd. Int. Symp. on Geotech. Aspects of Underground Construction in Soft Ground, IS-Toulouse*.

Bezuijen, A. 2007. Bentonite and grout flow around a TBM. *Proc. ITA 2007, Prague*.

Bezuijen, A., Pruiksma, J.P. & Meerten, H.H. van. 2001. Pore pressures in front of tunnel, measurements, calculations and consequences for stability of tunnel face. *Proc. Int. Symp. on Modern Tunneling Science and Techn. Kyoto*.

Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel. *Proc ICOF 2003 Dundee.*

Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements of grout pressures during tunneling of the Sophia Rail tunnel. *Soils and Foundations* 44(1): 41–50.

Bezuijen, A. & van Lottum, H. (eds). 2006. *Tunnelling A Decade of Progress*. GeoDelft 1995-2005, Taylor and Francis/Balkema, Leiden, ISBN 0 415 39113 4.

Bezuijen, A. & Zon, W. van der. 2007. Volume changes in grout used to fill up the tail void. *Proc. ITA 2007, Prague*.

Bogaards, P.J. & Bakker, K.J. 1999, Longitudinal bending moments in the tube of a bored tunnel. *Numerical Models in Geomechanics Proc. NUMOG VII:* 317–321.

Broere, W. 2001. *Tunnel Face Stability & New CPT Applications*. Ph.D. Thesis, Delft University of Technology, Delft University Press.

Gafar, K., Soga, K., Bezuijen, A., Sanders, M.P.M. & van Tol, A.F. 2008. Fracturing of sand in compensation grouting. *Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

Guglielmetti, V. 2007. *Tunnels and Tunnelling International*, October, P32.

Hashimoto, T., Brinkman, J., Konda, T., Kano, Y. & Feddema, A. 2004. Simultaneous backfill grouting, pressure development in construction phase and in the long term. *Proc. ITA Singapore*.

Hoefsloot, F.J.M. 2008. Analytical solution longitudinal behaviour Tunnel lining. *Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

Kasper, T. & Meschke, G. 2006. On the influence of face pressure, grouting pressure and TBM design in soft ground tunnelling. *Tunn. and Undergr. Space Techn.* 21: 160–171.

Leca, E. & Dormieux, L. 1990. Upper and lower bound solutions for the face stability of shallow circular tunnels in frictional material. *Géotechnique* 43: 5–19 (in French).

Merritt, A.S. & Mair, R.J. 2006. Mechanics of tunnelling machine screw conveyors: model tests. *Geotechnique* 56(9): 605–615.

McKinley, J.D. & Bolton, M.D. 1999. A geotechnical description of fresh cement grout – Filtration and consolidation behaviour. *Magazine of Concrete Research* 51(5): 295–307.

Steiner, W. 1996. Slurry penetration into coarse grained soils and settlements from a large slurry shield tunnel. *Proc. Geotech. Aspects of Underground Construction in Soft Ground*, London, Mair and Taylor (eds). Balkema, Rotterdam, ISBN 9054108568: 329–333.

Steiner, W. 2007. Private communication.

Talmon, A.M. 2007. *Notes on analytical beam model.* Delft Hydraulics report Z3934/Z4145.

Talmon, A.M., Aanen, L. Bezuijen, A & Zon, W.H. van der. 2001. Grout pressures around a tunnel lining *Proc. Int. Symp. on Modern Tunneling Science and Techn. Kyoto*.

Thewes, M. 2007. Private communication.

Verruijt, A. 1993. *Soil Dynamics*. Delft University of Technology, b28.