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# Field measurements on grout pressures during tunnelling

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ABSTRACT: Grout pressures were measured on a tunnel lining of approx. 9.5 m diameter during drilling. 14 pressure gauges were mounted in one ring of the tunnel lining. Measurements were performed for 15 hours, during this time the final hardening of the grout did not occur. The start of the drilling process after completion of the ring resulted in the lowest pressures. Results show that the grout pressure distribution is dominated by the injection during drilling, but this distribution changed when drilling stops and buoyancy forces get an influence. Farther away from the injection points the influence of buoyancy forces increases.

## 1 INTRODUCTION

Since 1996 the Centre for Underground Construction (COB) in the Netherlands has initiated a substantial amount of practical oriented field and desk research (Bakker et al, 2001). One of the main issues in this research was settlement control and prediction. Given the fact that pressure control at the front of slurry type TBM's is generally controlled properly nowadays, one has to concentrate on the improvement of process control regarding backfill grouting in the annular gap in order to increase settlement control. The outcome of the research so far has shown an evident qualitative correlation between the quality of the grouting process in the annular gap at the back of the TBM and settlements. However, this correlation has not led to a quantified relationship yet. With a metro tunnel under the historic city center of Amsterdam coming up next years (Kaalberg et al, 2001) the autonomic government policy for ongoing tunneling research is additionally supported.

Therefore it was decided by COB to define additional field and desk research at the Sophia Rail Tunnel (3,6 km twin bore tunnel, external diameter 9,4 m). One of the main issues in this research is to increase knowledge regarding the injection, flowing and hardening process of the backfill grouting, in order to obtain a more constant, reliable and predictable settlement control for future projects in an urban environment worldwide.

Another main topic in the research is the behaviour of the tunnel lining during construction. It is not seldom seen that damage occurs at segmental joints, which originates primarily from joint design, but presumably also from the (in homogenously distributed) loads of the fluid grout in the annular gap.

In order to tackle both issues in one field research two complementary ways of monitoring the grouting process were defined. The backside of the TBM is provided with 10 circumferentially installed grout pressure sensors. And one cross section is defined where both building, surface and subsurface instrumentation is installed as well as two tunnel rings (two tunneling directions) are circumferentially instrumented with grout pressure sensors in the lining.

When the TBM is passing the instrumented cross section both ways of grout pressure data capturing can be compared. The aim is to analyse the grout pressure distribution around the tunnel instantaneously behind the TBM and at a greater distance and investigate the correlation. This information will be used as an input later on for 4D FE models (Dijk & Kaalberg 1998) to further calibrate and validate settlement prediction and lining calculations.

In this paper the monitoring data of the sensors in the lining for the first TBM passage are described and analysed and compared with a flow prediction model.

## 2 THEORY

Before hardening grout can be described as a Bingham liquid. However, the properties change with time. Viscosity and yield stress will increase. Talmon et al (2001) presented a flow model to describe the pressure distribution around a tunnel lining caused by the grout flow and as a function of the discharge through the injection points.

However, a Bingham liquid can withstand shear stresses without flow (what a Newtonian fluid cannot). This means that also without flow not necessarily the hydrostatic pressure distribution is present. In the case of grout around a tunnel lining, the lining shall move upwards due to buoyancy forces. It can only move if the grout is pressed down. This will be prevented by the shear stresses between the soil and the grout. From our experiments and analysis of field measurements, Talmon et al (2001) we have concluded that the shear stress between concrete and grout is much smaller than the shear stress between soil and grout and therefore the former can be neglected). The relation between shear stress and pressure caused by that shear stress can be derived from Figure 1, and results in:

$$\Delta P = \tau \frac{R}{s} d\alpha \tag{1}$$

Where ΔP is the pressure change caused by the yield stress τ, R the radius of the hole made by the tunnel, s the joint width and α the angle indicated in Figure 1. Assuming that the grout flows to the bottom of the tunnel from a location with a certain angle α, the total pressure drop by the yield stress is:

$$P = \tau \frac{R}{s} \alpha \tag{2}$$

This excess pressure that increases with α will result in a downward directed force on the tunnel that compensates the buoyancy force K. For a small segment it can be written:

$$dK = PR \cos \alpha d\alpha \tag{3}$$

Combining Equations (2) and (3) and integration over the two half circles leads to:

$$\tau = \frac{Ks}{D^2} \tag{4}$$

With this last equation it is possible to calculate what yield stress must be present to prevent upward moving of the tunnel lining until it hits the ceiling of the hole drilled due to buoyancy force.

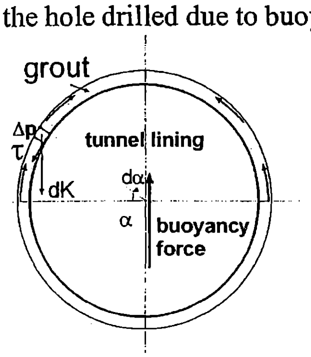


Figure 1: Scheme to calculate the pressure distribution due to buoyancy forces.

If the actual yield stress τ is larger than the result of Equation (4), then there will be only limited movement of the tunnel and Equation (2) using the result of (4) gives the influence on the pressure distribution. Without shear stress the pressure will increase according to a hydrostatic pressure distribution. Equation (2) leads to a pressure highest on top of the tunnel and thus counteracting to the hydrostatic pressure. The resulting pressure gradient will therefore be less than the hydrostatic gradient.

## 3 MEASUREMENT PROCEDURE

Several segments in one ring of the tunnel consist of an instrumented annulus where pressure sensors were installed. The positions of the sensors in the ring are shown in Figure 2.

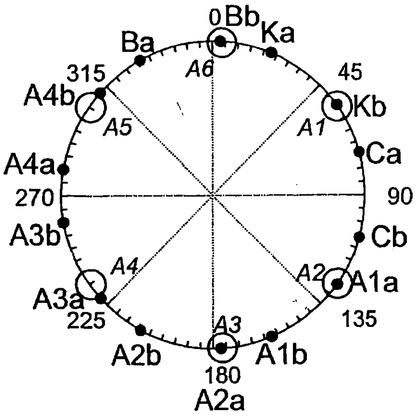


Figure 2: Distribution of pressure sensors in the lining (•) and grout injection points on the TBM (O) with the code used in the graphs.

These pressure sensors, in stainless steel housing, have been permanently placed using epoxy resin in holes drilled in the concrete segments and can be used for measurements even after initial pressure monitoring. These segments then underwent a competency and waterproofing test by applying a pressure of 1Mpa for 2 hours.

The pressure sensor is the Labom type DE1680-A4001-L19 that has a measuring range of 1 MPa and an accuracy of 0.25 % full-scale (2.5 kPa). The sensor consists of a flat membrane (50.8 mm diameter) placed on its outer surface. The grouting liquid is therefore in direct contact with the membrane during the measurement. The membrane is placed 4 cm below the outer surface of the segment to prevent damage by the steel brushes at the end of the TBM shield as it passes by. To prevent airlocks the space above the membrane is automatically filled with the same grease as used to seal the tail end.

The sensors were sampled at 25Hz using a data acquisition system based on an internal PC card (Keithley DAS 1802). This card controls the sensors and collects and archives the measured data.

Data is recorded while the segment is still within the TBM shield and measurements are continuously logged for 15 hours.

The relative high frequency sampling of 25 Hz was used to measure possible high frequency events. Analyzing the data including Fourrier transform showed that such events were not available. Therefore a considerable data reduction of 1 of 500 samples could be used without loss of significant data.

## 4 RESULTS

### 4.1 *Pressures versus time and displacement*

Measured grout pressures are shown in Figure 3 as a function of time together with the drilling velocity. This figure shows that grout pressures increase during drilling when grout is injected, but decrease during periods of stand still, probably due to dewatering of the grout into the soil.

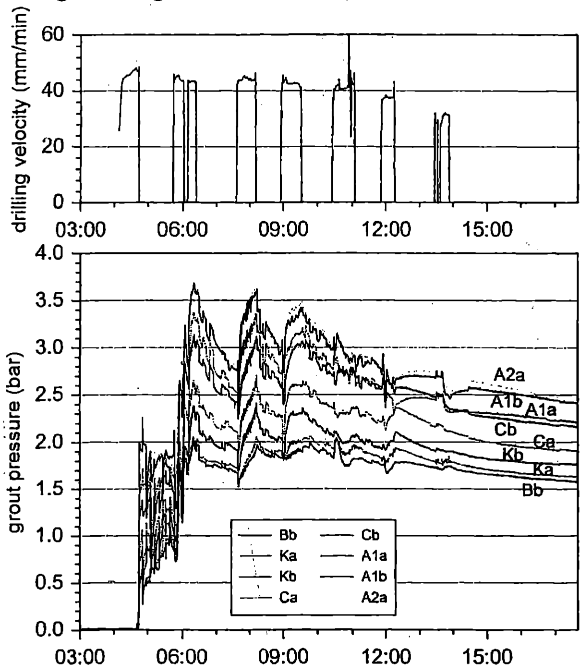


Figure 3. Drilling velocity and measured grout pressures at the right side of the tunnel as a function of time (see for the lettering also Figure 2).

Only halve of the instruments is shown because the number of lines would lead to confusion if the results of all transducers were shown. Just before 5:00 the grout pressure transducers pass trough the tail brush. At approximately 6:00 they come into the grout. After the completion of the drilling for that ring the grout pressures decrease until at approximately 7:30 the drilling for the next ring started. In the decreasing pressure some sharp rise or fall in pressure can be distinguished. Analyzing the measurements it was shown that the ring building process caused this. The rise or fall could be associated with the movement of the plungers of the main jacks during the process of placing segments of the tunnel lining.

Remarkable is the dip in the grout pressure, measured with all transducers just before the drilling for a new ring started. The reason for this dip can be seen in Figure 4. The figure shows the drilling velocity and the number of strokes of the grout pumps per second as a function of the displacement in the upper plot. It can be seen that when the drilling started (the velocity jumps from zero to a certain value) the grout pumps do not start immediately and therefore the first drilling is done with too little grout leading tot a sharp decrease in the grout pressure. After that the pumps started and the grout pressure increases again.

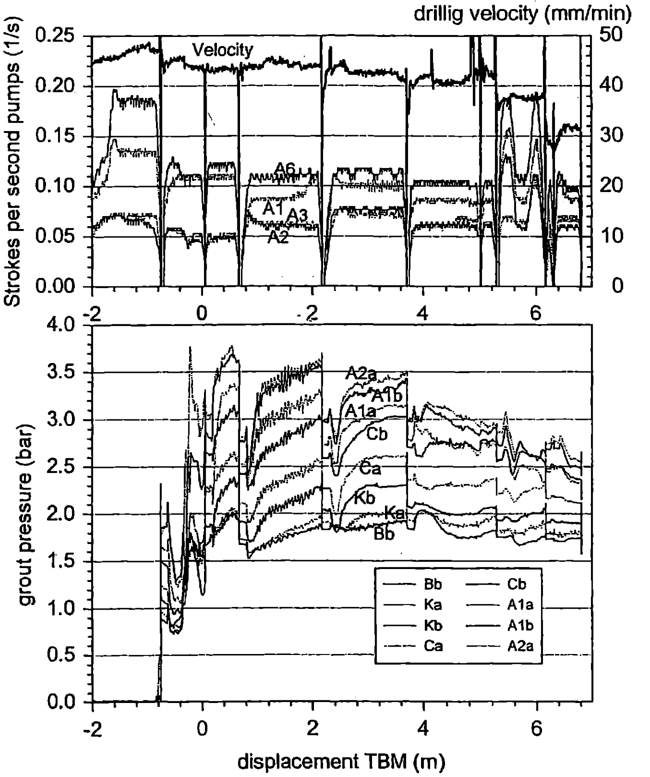


Figure 4. Drilling velocity, strokes per second of the grout plunger pumps and grout pressures as a function of the displacement. (a moving average over 25 points was taken for the data on the plunger pumps to reduce noise on the data).

### 4.2 *Polar plots and gradients*

Figure 5 shows the measured pressures at different times in a polar plot and the distribution of pressures as a function of height. Both plots showed that the pressure distribution is as could be expected. The pressure increases more or less linearly with depth. The pressure distribution is more or less the same on both sides of the tunnel, although there is some difference between both sides for the 7:00 measurement. Remarkable is that the pressure gradient in vertical direction is not constant in the measurements. The slope of the line in the lower plot of Figure 5 changes with time. The results shown in this paper are the results of measurements on one ring. This means that after installation there will be the same grout present during the measurement. If only the hydrostatic pressure in the grout would determine the pressure distribution, then the gradient should be constant. However, more aspects play a role as was mentioned in the Theory section and will be elucidated further in the discussion. To investigate what gradients did occur, the pressure gradient in vertical direction was calculated using the least square method over all measurement points. The result in shown in Figure 6 together with the pump activity of the pump for grout injection line A1 (The pump for A1 was chosen to have a reference when the pumps are on. It could have been also one of the other pumps) The results show that in general the vertical pressure gradient decreases in time and thus also as the distance between the TBM and the instrumented lining increases.

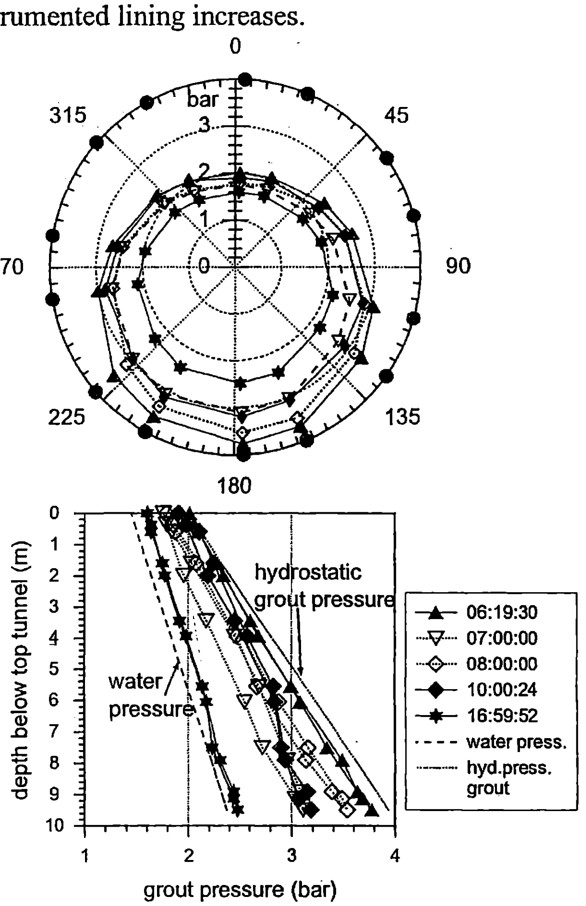


Figure 5. Polar plots and pressure as function of height at various time steps. The lower plot also shows the water (pore) pressure and the hydrostatic pressure in the grout starting from 2 bar at the top as a reference. Drilling at 6:19:30 and 8:00:00 stand still for the other points.

Remarkable is that up to 12:00 drilling (and therefore activity of the grout pump A1) leads to a sharp increase in the pressure gradients. For the last rings there is just a decrease. Possible explanations will be dealt with in the Discussion Section.

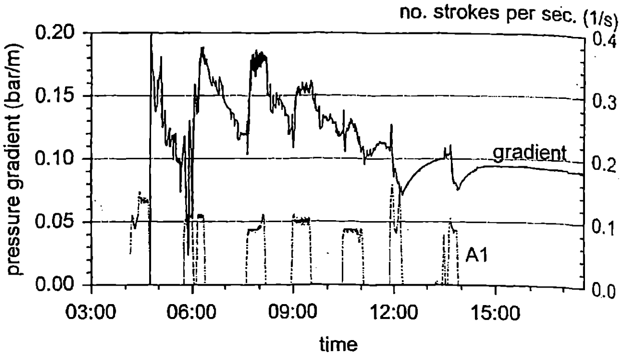


Figure 6: Pressure gradient over the tunnel lining at one location and pump activity for one of the injection points (A1) as a function of time.

## 5 DISCUSSION

### 5.1 *Pressures*

The estimated effective pressure on top of the tunnel lining was at the location of the instrumentation approximately 200 kPa (2 bar) (Bezuijen & Talmon 2001). This is close to the grout pressure applied during drilling on top of the tunnel (instrument Bb in Figure 3 and Figure 4). When drilling stops the grout pressures decreases. This decrease is largest at the pressure gauges at the lower part of the tunnel. This will be the reason that the reported settlements at this ring were only limited.

The lower plot in Figure 5 shows that the grout pressures are above the pore pressures at that location although the difference is only small for the last measurement (time = 16:59:52). It also shows clearly that the measured pressures are always lower than the hydrostatic pressure in the grout would be applying a 2 bar grout pressure at the top.

### 5.2 *Gradients*

The grout pressure on top of the tunnel is an important parameter that influences the surface settlement. The vertical pressure gradient is of importance to understand the grouting process, but also determines, with the absolute pressure, the stress distribution in the soil, which is of importance to understand soil-structure interaction for adjacent foundations.

The volumetric weight of the grout used was 21.5 kN/m$^3$. In case the hydrostatic pressure determines the vertical pressure gradient, the gradient should be 21.5 kPa/m. As can be seen from Figure 5 and Figure 6, this value is never reached during these measurements. A likely reason for this discrepancy are the buoyancy forces as was explained before. However, also when there is still a grout flow the pressure distribution will not be a hydrostatic distribution (Talmon et al 2001). During drilling, close to the TBM the pressure distribution is governed by the grout flow from the grout injection points around the tunnel lining. In case of a downward directed flow, the pressure, corrected for hydrostatic pressures, has to decrease to maintain the flow.

The flow model described by Talmon et al (2001) was used to simulate the grout pressures. The parameters used are summarized in Table 1.

Table 1. Parameters used in grout flow calculation.

| Parameter | value | dim. |
|---|---|---|
| Diameter lining (D=2R) | 9.45 | m |
| Thickness grout layer (s) | 0.16 | m |
| Density grout ($\rho_{gr}$) | 2190 | kg/m$^3$ |
| Yield stress grout start | 0.9 | kPa |
| Yield stress grout after 7.5 hour | 1.8 | kPa |
| Viscosity parameter | 5 | Pa s |
| Grout pressure at top | 2 | bar |
| Drilling velocity | $7.2*10^{-4}$ | m/s |

The density and rheological parameters of the grout were determined from samples taken from the construction site. The distribution of grout injection flow rates over the injection ports has been read from the strokes made by the grout delivery pumps, see Figure 4. Our vane-measurements revealed that up to a grout age of 5.5 hour the yield stress was 900 Pa, at this age the yield stress began to increase. At an age of 7.5 hours the yield stress was 1.8 kPa, and increasing with time. The results are shown in Figure 7 and showed good agreement with measurements obtained during drilling.

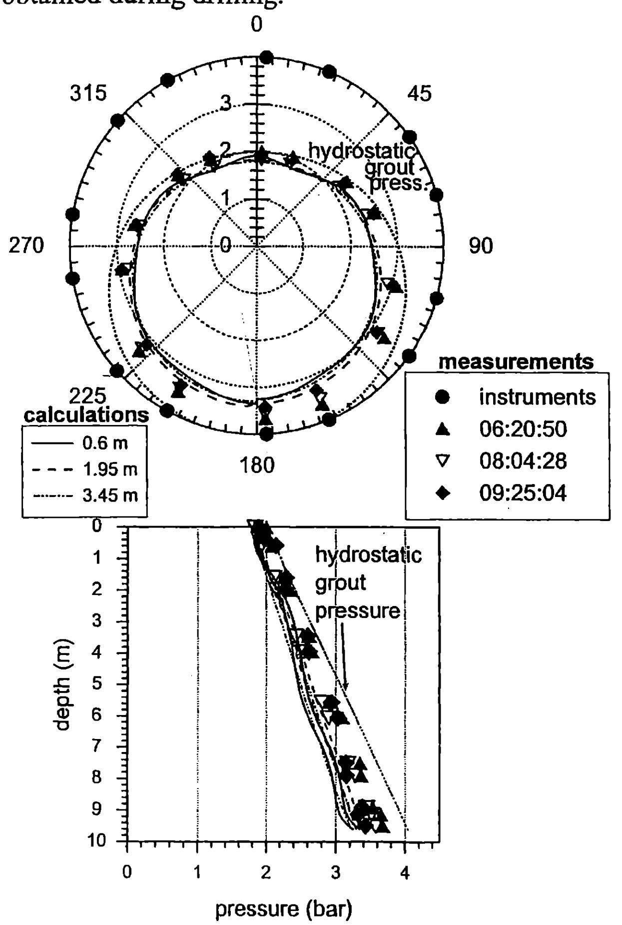


Figure 7: Measured and calculated grout pressures compared during drilling.

The model described to calculate the influence of the buoyancy forces on the pressure distribution does not predict a constant vertical pressure gradient as was measured. The gradient is assumed to be constant over the circumference (Equation (1) showed that $dP/d\alpha$ is constant) and therefore would vary over the tunnel height. The maximum gradient is found at 90 and 270 degrees (see Figure 5, upper plot). At these points the gradient can be written as:

$$\frac{dP}{dz} = \rho_{gr} g - \frac{\tau}{s} \tag{5}$$

With $\rho_{gr}$ the density of the grout and g the acceleration of gravity.

The additional parameters used in this calculation to calculate the gradient are presented in Table 2. The grout pressure at the top is lower now, because this pressure decreases when grouting stopped.

Table 2. Additional parameters used in calculations.

| Parameter | value | dim. |
|---|---|---|
| Thickness lining (sl) | 0.4 | m |
| Density concrete | 2400 | kg/m$^3$ |
| Grout pressure at top | 1.8 | bar |

Using these parameters it can be calculated that the buoyancy force K = 1370 kN/m. This resulted with Equation (4) in a shear stress $\tau$ of 2.45 kPa between the soil and the grout. With these results and Equation (5) the maximum gradient is found to be 6.1 kN/m if only buoyancy forces determine the gradient. This is less than the minimum gradient measured and therefore it can be concluded that the measured gradient is also influenced by moments in the lining because it is fixed by the TBM on one side and the hardened grout on the other.

These buoyancy calculations led to another result. The calculated shear stress is more than the yield stress measured (Table 1). This would mean that the tunnel would not be stable and would be lifted by buoyancy forces. Another way to look at the results is to determine the yield stress in the grout from the measurements. In case the rheological properties of the grout would be the same as for water, gradient would be always 21.5 kPa/m with or without drilling. Assuming that the tunnel wants to move in upward direction due to buoyancy forces, then the minimum measured gradient results with Equation (5) in a mobilized shear stress. This can be less than the yield stress, but not more. From using Equation (5) it can be concluded that a measured gradient of 0.12 bar/m (12 kPa/m) around 8:00 (see Figure 6) corresponds with a minimum yield stress of 1.5 kPa. The minimum in the gradient measured just after 12:00 of 0.076 bar/m means that the yield stress is then 2.2 kPa or more. These values for the yield stress are higher than measured with the vane. Possible reasons for these discrepancies are:

1. The reological parameters are determined by atmospheric pressure. Dewatering in the field situation can lead to higher yield stress.
2. Based on flow experiments the shear stress along the tunnel lining and the grout is neglected. It is possible that this changed in no-flow conditions.

From the measurements presented here it appeared too simple to state that the pressure distribution close to the tunnel is governed by the TBM and that buoyancy is dominant further away. Close to the TBM the pressure is dominated by the grout flow during drilling, but is more influenced by the buoyancy during stand still. The model presented on the buoyancy is purely 2-D. It only takes into account one ring and does not take into account the possible bending moment along the tunnel axis. This appeared to have a large influence close to the TBM. However, further away from the TBM the influence of the bending moment in the lining decreases and the gradient approaches the value calculated with Equation (5).

During drilling around 12:00 and 14:00 the pressure gradient is the lowest. This is during drilling but the TBM is that far away from the instrumented lining that the grout flow has hardly any influence on the pressure distribution. In such a case forces exerted by the TBM on the lining can lead to movements of the lining and this can lead to a change in the pressure distribution. For example, if larger jack forces are used in the bottom part of the TBM to compensate its tendency to go too deep caused by its weight. These forces will induce a moment in the lining that lead to movement of the lining until the pressures on top of the lining have increased to compensate for this moment.

## 6 CONCLUSIONS

The grout pressure measurements presented in this paper and the comparison with calculations lead to the following conclusions:
1. Reliable measurements were obtained for the grout in the liquid phase. Measurements were too short to give data for the hardening phase of the grout. It is also not certain whether or not the instrumentation used would survive the hardening phase.
2. The grout pressures are not only influenced by the injection strategy and flow, but also by forces exerted on the lining during placing of segments or drilling.
3. The drilling started before start of the grout injection pumps. This led tot a decrease in grout pressure at the beginning of the drilling. This pressure decrease could be measured up to 5 m behind the TBM.
4. Injection strategy can influence the pressure distribution close to the TBM (<5 m) during drilling. Here account has to be given to changes in rheological properties at larger distances because the final set of the grout commences. The influence of injection decreases with distance. During standstill and further away from the TBM buoyancy forces and the bending moments in the lining govern the pressures. This results in pressure gradients considerably lower than according to a hydrostatic pressure distribution in the grout.

## 7 FUTURE PERSPECTIVE

After the second passage of the TBM next summer the TBM sensors will be evaluated and compared with the next ring of lining sensors. Parallel to that (sub)surface settlements will be evaluated in order to further validate the 4D prediction models. The observed TBM process parameters along a tunnel track of ca. 100 m will act as an important 4D input. On top of that damage assessment of several houses above the tunnel is anticipated.


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