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

Backfill grouting research at Groene Hart Tunnel

A.M. Talmon
Deltares (Delft Hydraulics) & Delft University of Technology, Delft, The Netherlands

A. Bezuijen
Deltares (GeoDelft) & Delft University of Technology, Delft, The Netherlands

ABSTRACT: Measurements on back-fill grouting at Groene Hart Tunnel (GHT) are described and analysed. The research utilises TBM data, monitored grout pressures and measured rheological properties of tunnelling fluids. Grout injection strategy and vertical grout pressure distribution behind the TBM were found to agree with established theory, like found in other tunnelling projects. However, on some occasions different smaller vertical grout pressure gradients were found, which might have been caused by an incidental upward movement of the tunnel. It is shown that the consolidation of grout determines the decay of grout pressures. The associated net hydraulic resistance of surrounding soil is substantially higher than according to radial flow theory. This higher resistance is attributed to bentonite originating from the face of the TBM, having invaded the grout-soil interface.

1 INTRODUCTION

The Groene Hart Tunnel (GHT) in the Netherlands is part of a high speed railway line between Amsterdam and Brussels. The location of the tunnel is about 20 kilometers South-West of Amsterdam. The length of the tunnel is 7.16 km. The tunnel is situated at a depth of about 30 meters. Hydro-geotechnical conditions are described by Aime et al. 2004. This double track tunnel has a diameter of 14.5 meters. Aristaghes et al. 2002 reported on the design of the tunnel.

Grouting research at GHT was conducted as a cooperation between Delft Institutes, HSL project organisation and COB-research. The study focused on the interaction of soil, grout and tunnel construction at the injection of grout. Three specific aims were: 1) Analysis of grout pressures measured at GHT. 2) Quantification of the consolidation of grout and its influence on grout pressures. 3) Modelling of the grout pressure distribution; specifically from grout injection at the TBM to the buoyancy dominated region further behind the TBM. Novel conditions at the time were the impressive diameter of the tunnel: 14.5 m, in contrast to the 10 m diameter tunnels monitored up to then, the critical hydro-geotechnical soft soil conditions and a different type of grout. The geotechnical profile is shown in Fig.1.

Laboratory characterisations were conducted on the grout: vane testing for rheological properties, consolidation experiments, and tests to determine to

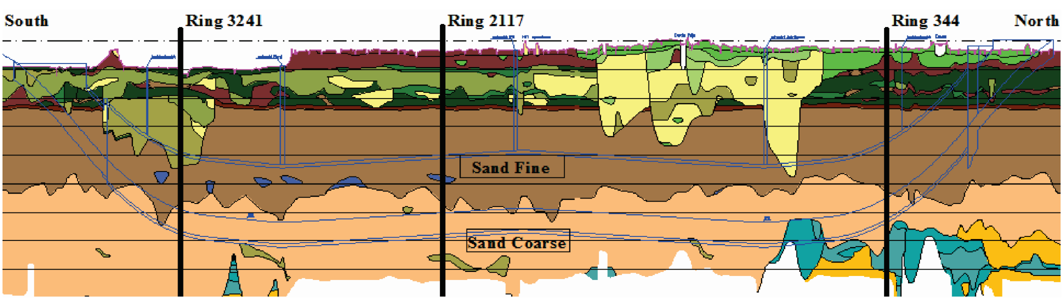

Figure 1. Geotechnical profile Groene Hart Tunnel and measurement locations: Instrumented plot no 1 at ring 344 (29/03/2002), COB-passage at ring 2117 (03/06/2003) and Noordplaspolder at ring 3241 (05/11/2003).

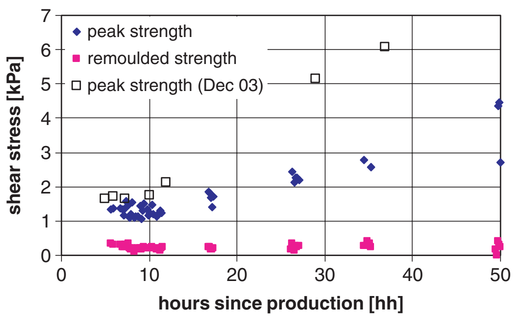

Figure 2. Peak shear strength and remoulded strength of GHT tail void grout, determined by vane testing.

what degree of grout consolidation the grout pressure transducers used in the lining will give reliable results. Also the rheological and consolidation properties of bentonite that could surround the grout were measured.

The construction of Groene Hart Tunnel was monitored on a number of locations. Three locations were available for Delft Institutes research: "Instrumented plot no 1", a so-called "COB-passage" and a passage in "Noordplaspolder", see Figure 1.

In each of these locations grout pressures on the tunnel lining were measured. The COB-passage had additional instrumentation: pore water pressure measurements, axial and tangential strain measurement in the tunnel lining, convergence measurements, vertical position measurements of the tunnel lining and tilt measurements of the tunnel lining. This paper focuses on grout pressures.

2 PROPERTIES OF TUNNELING GROUT

2.1 *Rheology of grout*

Yield stresses are governing grout pressures behind a TBM. Shirlaw et al. (2004) describe the ingredients of the cement-less grout employed at Groene Hart Tunnel (GHT): sand, flyash, lime and chemical additives. They also address some of its physical properties. Fresh grout samples were taken (27/03/2002) from a supply container located in the TBM. The grout was tested in Delft. The density of the grout is 1850 kg/m^3, the water content is 0.201.

Peak shear strength and remoulded shear strength were measured with a vane apparatus as a function of time. Figure 2 shows the results. The peak shear strength of fresh grout is about 1 to 1.5 kPa. The tests also show that peak shear stresses increase slowly in time. This is caused by puzzolanic reaction of the flyash present in the grout. The remoulded shear strength, on the contrary, is nearly constant in time. The grout was fine grained, though no sieve tests were conducted for quantification. Similar tests were conducted on grout collected in December 2003. Also the shear modulus of grout was determined. This property is needed to calculate the transition to elastic behaviour (=input to DCLong: Talmon & Bezuijen 2005).

The peak shear strength, instead of the remoulded strength, is here the relevant parameter, since through continued consolidation the failure surfaces are closed continuously, and it is here that the peak strength has to be surmounted continuously.

2.2 *Consolidation of grout*

In porous soil conditions elevated grout pressures, compared to the pore water pressure at the same location, lead to consolidation of grout, or grout bleeding. The consolidation of grout, in turn, influences the effective stress distribution some distance behind the TBM. Consolidation starts at the grout-soil interface, and proceeds slowly into the grout layer that fills the tail void (this layer is called a grout cake). Consolidation properties of GHT grout were determined in Delft by means of element tests such as described by Bezuijen & Talmon (2003). Permeability and porosity change of the grout are the governing parameters. Relevant results to the calculation of grout consolidation in the tail void are: permeability grout cake $k = 2.4*10^{-8}$ [m/s], porosity fresh grout $n_i = 0.31$ and porosity of the consolidated grout cake $n_e = 0.24$.

3 LABORATORY TESTING OF GROUT PRESSURE SENSORS

Pressure sensors (SenSym/ICT 19C 100 P) have been mounted in the tunnel lining segments. A cavity (D = 82 mm, h = 23 mm) filled with tail brush grease separated the sensor membrane from the grout (this construction is comparable to that at Sophia Rail Tunnel, which is described by Bezuijen et al. 2004).

Since at the GHT beam action of the tunnel lining was the primary subject of the COB-research, it had to be sure that the grout pressure data was reliable, specifically under consolidated/hardened conditions of the grout. The functioning of these grout pressure sensors was tested in the laboratory, Bezuijen (2004). A sketch of the test cell is shown in Figure 3. A pressure sensor was placed in a grease filled cavity position, as at GHT.

The grout in the test cell was loaded by increasing the air pressure in the top of the cell. The resulting grout pressure was the same as in the field.

Up to about one hour, this sensor adequately measured externally imposed variations of total pressure, see Figure 4. After that the pressure readings were in

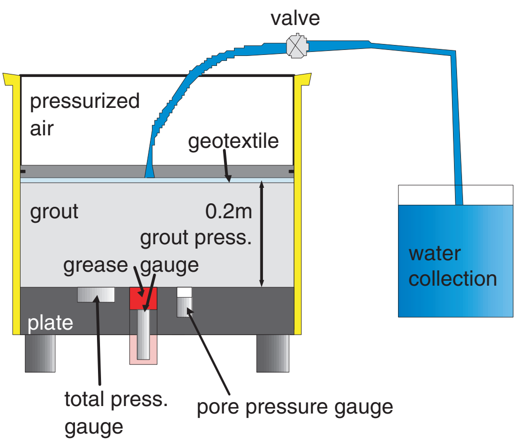

Figure 3. Laboratory testing of grout pressure sensor.

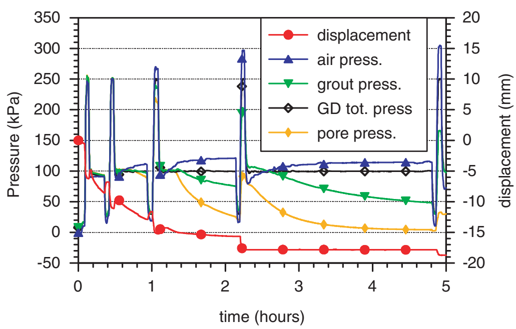

Figure 4. Example of pressure sensor response.

between the imposed total pressure and the pore-water pressure.

During this process another flush mounted pressure sensor with very stiff membrane, continued to measure the total stresses. It is concluded that after one hour consolidated grout reached the pressure sensor. Subsequently grout-arching might have occurred in front of the GHT pressure sensor when loaded dynamically. As a consequence, pressure readings at GHT have to be interpreted with care. It should however be mentioned that these tests might have been too stringent; the imposed pressure variations are not likely to occur in the field. Instead there is a slow decay of grout pressure with time, given arching less chance to occur. Furthermore the sensor is located at the impermeable lining. In Section 6 it will be shown that in a tunnelling process it can take more than 25 hours before the grout close to the lining is consolidated

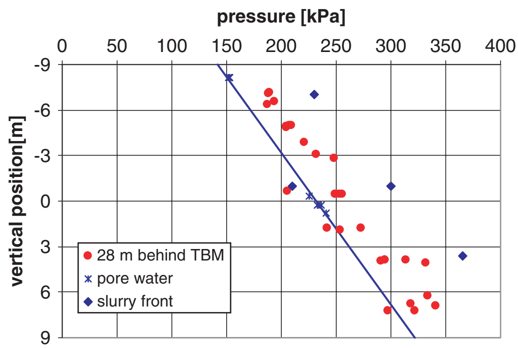

Figure 5. Pressures measured on tunnel lining at 28 m behind TBM.

4 GROUT PRESSURES MEASURED AT GHT

4.1 *Measured grout pressures at "Instrumented plot no.1"*

Twelve grout pressure sensors were utilised in "Instrumented plot no 1". The time-series were relatively short: the data covered at max 3 rings behind the TBM. The wireless FM-data transfer of the sensors suffered interruptions and malfunctioning.

The grout was injected through six injection pipes. The pressure drop over the pipes was about 3 bar. This pressure drop is equivalent to a wall shear stress of 1 kPa. This is significantly higher than found for cementious grouts such as at Sophia Rail Tunnel and Botlek Rail Tunnel, where about 0.2 kPa was found (Talmon et al. 2001).

Pump-stroke counts showed that about 60% of the grout was injected through the three injection ports in the upper half of the TBM. Thus a net downstream of grout is created at the back of the TBM.

Despite the sparse set of tail void pressure data, a picture emerged that the vertical grout pressure gradient behind the TBM ($\leq$10 kPa/m) could have been rather small in comparison to earlier tunnelling projects such as Sophia Rail Tunnel (Bezuijen et al. 2004).

4.2 *Measured grout pressures at COB passage*

At COB passage, tail void grout pressures were measured by a total of 32 pressure sensors. Ring 2117 and 2118 were each circumferentially equipped with 10 pressure sensors. The remaining 16 sensors were equally distributed over ring 2119, 2120 and 2121.

The grout pressure profile, 28 m behind the TBM, is shown in Figure 5. At this point it approaches the hydrostatic pressure distribution. Immediately behind the TBM, measured pressures are about 100 kPa above pore water pressure. Only two sensors located in the

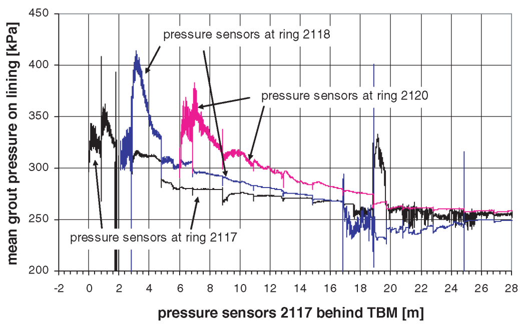

Figure 6. Mean grout pressure of ring 2117, 2118 and 2120.

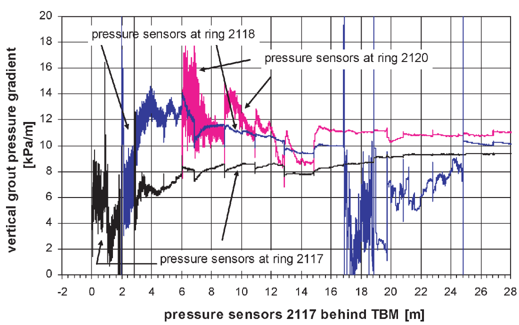

Figure 7. Vertical grout pressure gradients measured at ring 2117, 2118 and 2120. Between 16 and 25 metres electronics did not function correctly.

lower half of the tunnel, strangely, showed pressures equal to pore water pressure.

Average grout pressures measured at ring 2117, 2118 and 2120 are shown in Figure 6. It shows that grout pressures decay with distance from the TBM, in correspondence with the fundamental processes described in Talmon & Bezuijen (2005).

The pressure distribution over the circumference of the tunnel lining is determined by specific weight of the grout and flow resistance when the grout is distributed around the lining. The variation of the vertical grout pressure gradient with distance behind the TBM is shown in Figure 7. This figure shows that at upon exiting of ring 2117 this vertical pressure gradient is rather small: 6 kPa/m. This implies that there has been a down-flow of grout with a high flow resistance.

Measured vertical displacements of the tunnel lining (measured by a water levelling system measuring vertical displacements between ring 2117, 2118 and ring 2057) indicate that the tunnel lining had moved to a high position before the pressure sensors came into the grout. Before exiting of Ring 2117, the lining had already moved upward some 6 cm, see Figure 8. Ring 2057, located 120 m behind the TBM served as a reference.

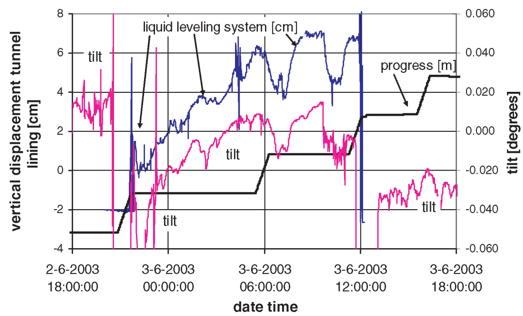

Figure 8. Vertical position of ring 2117 measured by a liquid leveling system and tilt of ring 2117 measured when the grout sensors first came into the grout (3-6-2003 06:00).

Ring 2117 and 2118 were also equiped with tilt sensors that were attached to tunnel ring segments situated at tunnel axis height. The time trace of the tilt meters varied in concert with vertical positions measured with the liquid levelling system. Near the end of drilling for ring 2120 (3-6-2003 12:00), the liquid levelling system had to be dismantled, the tilt measuring system continued measuring though. The tilt-measuring system showed that after the drilling of ring 2120, the tilt had dropped 0.04 degree, which according to Figure 8 corresponds to a vertical downward movement of about 6 centimetres since ring 2117 and 2118 came into the grout.

Overviewing the data, it is concluded that when ring 2117 and 2118 came into the grout the tunnel lining was at a temporally high position. The course of tilt meter data, vertical pressure gradient and vertical position correspond qualitatively. A representative vertical grout pressure gradient at the back of the TBM is 12 kPa/m.

4.3 *Measured grout pressures at Noordplaspolder*

The data from Noordplaspolder produced a more familiar picture: vertical grout pressure gradients of about 15 kPa/m immediately behind the TBM and a vertical gradient of about 8 kPa/m at 17 meters behind the TBM, see Figure 9.

5 ANALYSIS OF GROUT PRESSURES BEHIND THE TBM

A finite difference model for the calculation of grout pressures immediately behind a TBM was first presented by Talmon et al. (2001). The model calculates the distribution of grout issued from the injection ports, and consequently predicts the pressure distribution directly behind the TBM. A tail void of constant thickness is assumed and the computational domain stretches for about 5 meters behind the TBM.

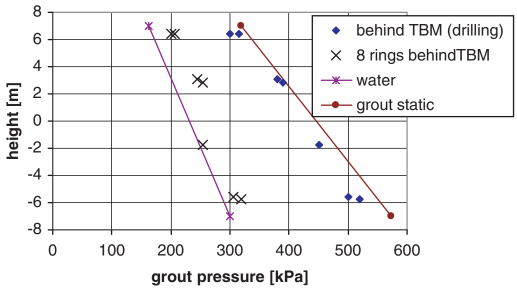

Figure 9. Vertical grout pressure profiles measured at ring 3241 of Noordplaspolder.

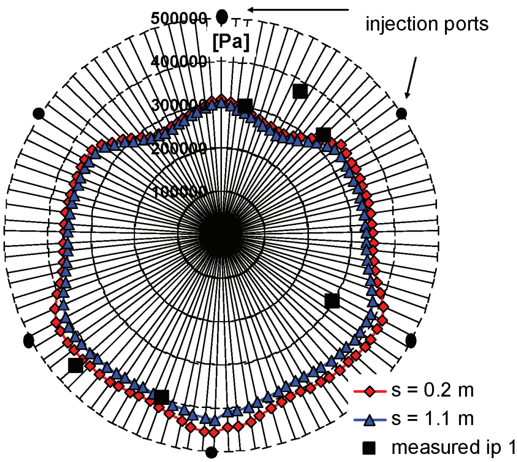

Figure 10. Grout pressures measured up to 1 m behind the TBM at "Instrumented plot no 1" and calculated pressures (model DCgrout). s = axial coordinate. Two-sided friction grout layer: $\tau_y = 1.5$ kPa, 60% of grout is injected through injection ports in the upper half of the TBM.

When the results of the "Instrumented plot no 1" came available a calculation of the grout pressure distribution immediately behind the TBM was made, see Figure 10. Inputs were the vane test results of March 2002 and the measured distribution of flow rates over the six grout-injection pipes.

Only if two-sided friction of the grout layer is assumed, like in the original model, a fair correspondence with the measurements is achieved. A pressure difference of nearly 150 kPa is calculated between the bottom and the crest of the tunnel. This pressure difference is equivalent to a vertical grout pressure gradient of 10 kPa/m. For reference, self-weight of the grout would produce a vertical pressure gradient of 18.1 kPa/m. For 300 kPa at the crest, this would lead to a pressure of 562 kPa at the bottom of the tunnel.

When first data of the COB-passage came available it was found that the distribution of flow rates over the injection ports was quite comparable: 55% of the grout was injected through the three upper ports. The grout pressure data (ring 2117) showed a small vertical grout pressure gradient of about 4 a 6 kPa/m, see Figure 7. This could not be explained by means of the DCgrout model. The measured vertical pressure gradient more is compatible with a situation where all of the grout is forced downwards. In that case the vertical pressure gradient is given by: $18.1\text{-}2{*}\tau_y/h = 3.1$ kPa/m.

At ring 2118 and 2120 a vertical grout pressure gradient of about 12 kPa/m is measured behind the TBM. Here the results correspond more with a situation where grout distributes in both the up- as downward direction from the injection ports.

The pace of the construction cycle at COB-Passage was slowed down by installation of instrumentation.

The measured pressure distribution at Noordplaspolder is more in line with earlier experience at other tunnels: f.i. Sophia Rail Tunnel (Bezuijen et al 2004).

Given the rheological properties of grout, the measured vertical grout pressure gradients can only be achieved when there is friction on both sides of the grout layer: friction at the soil-grout interface and friction at the tunnel-grout interface. Given the high frictional resistance measured over the supply lines, it seems justified to conclude that this grout is providing significant friction with the concrete tunnel lining (contrary to cementious grout employed in previous projects).

It has been found that the pressure distribution immediately behind the TBM varies strongly between measurement locations. A likely candidate that might have affected the pressure distribution is vertical movement of the tunnel lining. The higher yield stress of the grout used, compared to the grout in the earlier tunnels, has as a consequence that the movements of the lining have a larger influence on the pressure distribution around the lining.

6 CALCULATION OF THE CONSOLIDATION OF GROUT

Pressurized grout situated against a permeable surface is subject to fluid loss. Two different situations can be discerned: conditions during stand still of the TBM and conditions during drilling of the TBM.

Under stand-still conditions no fresh grout is supplied and grout pressures drop slowly. The pressurised soil will unload. Filtration properties of grout were measured in the laboratory, and a model was made for grout consolidation during stand still (see Bezuijen and Talmon 2003). This situation is verifiable on measured pressure decay during standstill. Involved properties are permeability of grout, porosity changes of the grout, hydraulic resistance of surrounding soil and elasticity of surrounding soil.

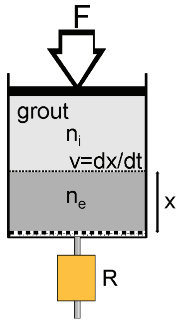

Figure 11. Skematisation of consolidation of grout. Elevated grout pressures are represented by the force F, the hydraulic resistance of pore water flow is represented by R.

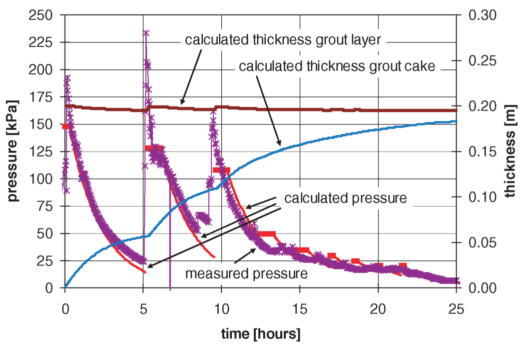

Figure 12. Numerical simulation of grout pressure decay behind the TBM at COB-passage compared with measurement: calculated pressure decay and calculated thickness of the grout cake, at $R = 2\ 10^7$ [s].

Under drilling conditions fresh grout is supplied and the grout pressure is constant. The surrounding soil will be under a constant pressure, and only permeability and hydraulic resistance are relevant.

The consolidation process is schematised in Figure 11. The hydraulic resistance R was back-calculated from observed grout pressure decay. Measured pressure decay and calculated grout pressures match well, see Figure 12. This back-calculated hydraulic resistance is however substantially higher than that of surrounding soil. A typical value for the hydraulic resistance of radial pore water flow into the surrounding soil is $R = 65000$ [s], Bezuijen (2005). It is hypothesised that bentonite slurry from the face of the TBM has invaded the grout-soil interface.

The consolidation theory allows the calculation of the development of the grout cake along the grout-soil interface, see Figure 12. The thickness of this filter cake is important to the modelling of the grout layer as an interface between tunnel lining and soil: it increases the integral stiffness of the grout layer. The foundation of the tunnel occurs by consolidation of the grout. This conclusion was also reached for the Sophia Rail Tunnel (Bezuijen et al. 2004), where cementious grout was used.

7 CONCLUSIONS

The fine grained grout at GHT has a yield stress of about 1 a 1.5 kPa and, due to absence of cement, the yield stress of this grout increases only slowly with time. It was shown that consolidation of grout determines the decay of grout pressures over at least five tunnel rings behind the TBM. The developing grout cake increases the integral stiffness of the grout layer and hence the tunnel is founded.

The associated net hydraulic resistance of surrounding soil is substantially higher than according to radial flow theory. This higher resistance is attributed to bentonite originating from the face of the TBM, having invaded the grout-soil interface.

Given the high frictional resistance measured over the supply lines and the small vertical grout pressure gradients behind the TBM, it is concluded that this grout is providing significant friction with the concrete tunnel lining (contrary to cementious grout employed in previous projects).

For a number of reasons the pressure readings at GHT were less reliable than at other tunnels. The GHT data is nonetheless valuable because it shows the general applicability of the results from earlier research at Sophia Rail Tunnel, it is providing data on the performance of cement-less grout with high yield stress and it shows a marked influence of vertical movements of the tunnel lining on the grout pressure distribution.

ACKNOWLEDGEMENT

The work was conducted as a co-operation between Delft Cluster, HSL-South Organisation and Centre for Underground Construction. The authors gratefully acknowledge HSL South Organisation, and their treasurer the Dutch Public Works Department, and Centre for Underground Construction for permission to publish.

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.

Bezuijen, A. 2004. Calibration grout pressure sensor GHT/Results of measurements (in Dutch: Ijking groutdrukopnemers GHT/Resultaten metingen), *COB report F512-O-04-127*.

Bezuijen, A. 2004. Vin tests grout GHT/Results of measurements (in Dutch: Vinproeven op grout GHT/Resultaten metingen), GeoDelft 403050/0008.
Bezuijen, A. 2005. Analysis of pore pressures close to the tunnel; influence of grout flow, (in Dutch: Analyse waterspanningen naast tunnel; invloed groutstroming op waterspanning), COB report F512-05-04.
Bezuijen, A. & Talmon, A.M. 2003. Grout the foundation of a bored tunnel, 2003, *Proc. int. conf. on Foundations "Innovations, Observations, Design & Practice" (ICOF 2003 Dundee)*, Thomas Telford, London.
Bezuijen, A., Talmon, A.M., Kaalberg, F.J. & Plugge, R. 2004. Field measurements on grout pressures during tunnelling of the Sophia Rail Tunnel, *J. of Soils and Foundations of the Japanese Geotechnical Society,* Vol.44, no.1, pp39–48.
Shirlaw, J.N., Richards, D.P., Ramond, P. & Longchamp, P. 2004. Recent experience in automatic tail void grouting with soft ground tunnel boring machines, in *proc. Underground Space for Sustainable Urban Development (ITA Singapore)*, Elsevier.
Talmon, A.M. & Bezuijen, A. 2005. Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts, *5th int. Symp. Geotechnical Aspects of Underground Construction in Soft Ground, 15–17 June, ISSMGE-TC28*, Amsterdam, The Netherlands.
Talmon, A.M., Bezuijen, A., Aanen, L. & van der Zon, W.H. 2001. Grout pressures around a tunnel lining, *proc. IS-Kyoto conference on Modern tunneling Science and Technology,* pp817–822.