Theme 2: Construction method, ground treatment, and conditioning for tunnelling

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

Ten years of bored tunnels in The Netherlands: Part I, geotechnical issues

K.J. Bakker
COB, Delft University of Technology, Delft, The Netherlands

A. Bezuijen
Deltares, Delft University of Technology, Delft, The Netherlands

ABSTRACT: Ten years have passed since in 1997 for the first time construction of bored tunnels in the Netherlands soft soil was undertaken. Before that date essentially only immersed tunnels and cut-and-cover tunnels were constructed in the Netherlands. The first two bored tunnels were Pilot Projects, the 2nd Heinenoord tunnel and the Botlek Rail tunnel. Since then a series of other bored tunnels has been constructed and some are still under construction today. At the beginning of this period, amongst others Bakker et al (1997), gave an overview of the risks related to bored tunnels in soft ground and a plan for research related to the pilot projects was developed. After that in 1999 the 2nd Heinenoord tunnel opened for the public, the "Jointed platform for Bored tunnelling", in short GPB, was organized, to coordinate further research and monitoring of bored tunnels. This platform is supervised by the Center for Underground Construction. In this paper a summary is given of some of the most characteristic observations on these 10 years of underground construction in the Netherlands. In the first part of this paper the focus is on geotechnical interactions, and stability, whereas part two will focus more on structural related issues.

1 INTRODUCTION

In 1992 a fact-finding mission was sent to Japan by the Dutch government, which reported that it should be possible to construct bored tunnels in the Dutch soft soil conditions. Up to that time essentially only immersed and cut-and-cover tunnels were constructed in the Netherlands, as boring of tunnels in soft soil conditions, at that time, was considered to be too risk full.

After this conclusion things went quite fast; in 1993 the Dutch minister of Transport and Public works ordered the undertaking of two pilot projects, the 2nd Heinenoord Tunnel and the Botlek Rail Tunnel. The projects were primarily aimed at constructing new infrastructure and besides that for monitoring and research in order to advance the development of this new construction method for the Netherlands. The projects started in 1997 and 10 years have passed since then.

At the start of the pilot projects, the difficulties with respect to constructing bored tunnels in soft soil conditions were evaluated and a plan for monitoring and research was put forward, see Bakker et al (1997). Since then, the 2nd Heinenoord tunnel, see Fig. 1, and a series of other bored tunnels were constructed.

After the completion of the pilot projects a Joint Platform for Bored tunnels was established (GPB) that coordinates the monitoring and research at the various other Dutch tunnelling projects. The GPB, an initiative of the larger clients for underground infrastructure on the government side, was organised under supervision of the Netherlands Centre for Underground Construction; COB. The research was organised in such a way that results of a project would be beneficial for a next project starting a little later.

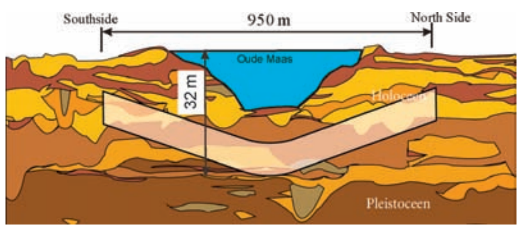

Figure 1. Geological profile at the 2nd Heinenoord tunnel.

Unquestionably a lot has been learned from the performed monitoring and research. The results of this process have been noticed abroad. In 2005 the Netherlands hosted the fifth International symposium of TC28 on "Underground Construction in Soft Ground". Researchers and experts from all over the world came to Amsterdam, to learn about the Dutch observations on tunnelling and to visit the construction works for the new North-South city metro system in Amsterdam.

Table 1. Bored tunnels completed after 1997 in the Netherlands.

		Completion (Year)	Bored length (m)	Depth (m)	Outward Diameter (m)
2nd Heinenoord tunnel	Road	1999	945 (dual)	30	8.3
Western Scheldt tunnel	Road	2003	6700 (dual)	60	11.30
Botlek Rail tunnel	Rail	2004	1835 (dual)	26	9.60
Sophia Rail tunnel	Rail	2005	4000 (dual)	27	9.60
Pannerdensch Canal Rail tunnel	Rail	2005	1615 (dual)	25	9.60
Green Hart tunnel	Rail	2006	8.620 (single)	30	14.90

The above event was also the occasion for the presentation of a book; "A decade of progress in tunnelling in the Netherlands" by Bezuijen and van Lottum (2006), where this research is described in more detail.

This paper(s) gives some highlights of the main research result of the past decade.

2 REVIEW OF THE 1997 SITUATION AND WHAT CAME AFTER

In the design phase for the 2nd Heinenoord tunnel a main concern were the soft soil conditions in combination with high water pressures. In general in the Netherlands the water table is just below the soil surface. Furthermore the 8.3 m outward diameter for this first large diameter tunnel was a major step forward, compared to past experience in the Netherlands; experience that was mainly based on constructing bored tunnels, pipes or conduits up to about 4.0 m diameter. This gave some concern with respect to the amount of extrapolation of empiric knowledge.

With respect to the soft-soil conditions, the low stiffness of the Holocene clay and peat layers and the high groundwater table; nearly up to the soil surface, were considered a potential hazard and a challenge for bored tunnels. The soil profile at the 2nd Heinenoord tunnel, see Fig. 1, is indicative for the heterogeneous character and on occasion sudden changes in underground soil layering that one might encounter. In addition to the heterogeneity and the ground water, deformations due to tunnelling might influence the bearing capacity of any existing piled foundations in the vicinity. And as the common saying is that the Amsterdam Forest is underground, one might realize the potential risks involved for the North/South Metro works in Amsterdam.

Characteristic for a high water table is buoyancy; the effect that the tunnel might be floating up into the soft upper layers above the tunnel due to the gradient in the groundwater pressure. Besides the risk of breaking up of these soil layers, the rather flexible bedding of the tunnel and the deformations that this may cause need to be analysed.

Therefore research was aimed at clarifying the effects of the soft underground, groundwater effects, and the effect of tunnelling on piled foundations.

After the successful construction of the two Pilot projects, a number of other bored tunnelling projects followed, see Table 1. Mention worth is that the Green Hart Tunnel holds until recently the record as the largest diameter bored tunnel in the world.

Still under construction are the tunnels for RandstadRail in Rotterdam, the Hubertus Tunnel for a road in The Hague and the North/South metro works in Amsterdam.

With respect to the construction of the North/South metro works in Amsterdam, the station works have made quite some progress and the bored tunnel is in a preparation phase. The elements of the immersed tunnel; the extension to Amsterdam North under the river IJ, are waiting for the completion of the immersion trench under the Amsterdam Central Station. For the bored tunnelling part, the TBM is expected to start excavation at the end of 2008.

Ten years after the pilot projects, the question arises whether the observations and related research have confirmed the above issues to be the critical ones or that advancing insight may have removed these issues from the "stage" and swapped these for other topics giving more concern.

In this paper some of the characteristic events and results of this past decade will be described. The choice for the topics being discussed is influenced by the projects that both authors were involved with, without intent to minimize the importance of other research that is not discussed in this paper. Further issues related to groundwater effects and grouting are described in more detail in a separate paper in this symposium by Bezuijen & Talmon (2008).

3 EXPERIENCES WITH BORED TUNNELS IN THE NETHERLANDS IN THE PAST DECADE

3.1 *An instability of the bore front*

During the construction of the 2nd Heinenoord Tunnel, approximately in the middle underneath the river Oude Maas an instability at the excavation front developed,

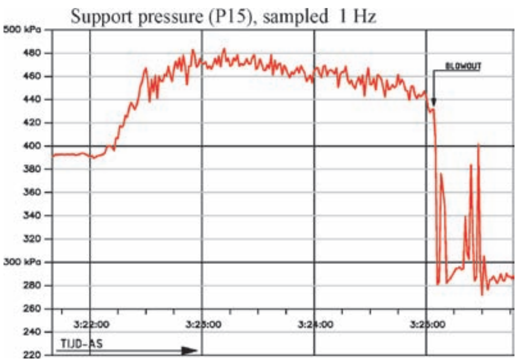

Figure 2. Support pressures before, during and after the "Blow out" at the 2nd Heinenoord tunnel.

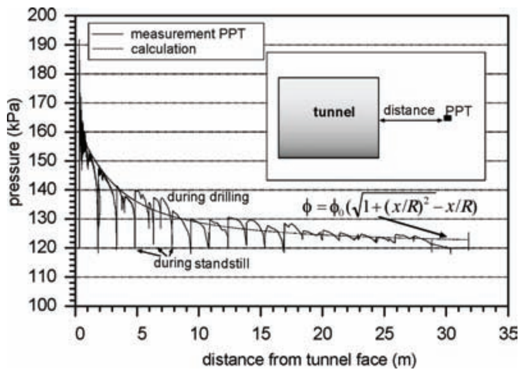

Figure 3. Pore water pressure distribution in front of the TBM.

see Fig. 2; afterward commonly referred to as "The Blow-out" (see also Bezuijen & Brassinga, 2001).

Backtracking the situation learned that after that a pressure drop was observed, in his efforts to restore frontal support, the machine driver first pumped bentonite to the excavation chamber; considering a deficiency in the bentonite system. When this did not help, air was pumped to the bore front; not realizing that the front itself already had collapsed. This collapse created a shortcut between the excavation chamber and the river. The action of pumping air was noticed by shipmasters on the river, which reported air bubbles rising to the water surface, which caused the failure to be known as the "blow-out". In this case the pumping of air had not been beneficial to the restoration of stability because pressure loss was not the cause but one of the results of the event.

This frontal stability at the 2nd Heinenoord tunnel has attracted some public attention. Presumably it is less known that loss of frontal stability has also occurred since then with some regularity at the other tunnels under construction in the years after, e.g. during construction of the Sophia Rail Tunnel and the Green Hart Tunnel, however without much delaying the construction process. At the 2nd Heinenoord Tunnel, construction work was delayed for several weeks before the crew succeeded in restoring frontal stability, filling up the crater in the river bottom with clay and bringing in swelling clay particles in the excavation room.

From the evaluation of the monitored pressures in the excavation room, it appeared that before the development of the instability, the frontal pressure was raised above the advised pressure for frontal support; i.e. at about 470 kPa instead of about 310 kPa. see Fig. 2 (pressure gauge P15 is in the excavation chamber at tunnel axis level).

In retrospect it was understood that during standstill, the pressures were raised to get a larger gradient in the pipes in order to improve the transport of excavated material; i.e. Kedichem clay that was found in the lower part of the excavation front and appeared to be difficult to pump through the hydraulic muck transport system.

The measurements indicate that excavation had started without releasing pressure to the standard support level during excavation. In that condition instability developed within 15 seconds after that the wheel started cutting. At stand still, when sufficient time has passed for a proper vertical cake sealing of bentonite to build up at the front, a high support pressure is not much of a problem, as the pressures used are way below those that might override the passive resistance at the front. However, as the pressure itself is fluid pressure, when the cake-sealing is taken away during excavation, and water can penetrate the front, according to Pascal's law for a fluid without shear stresses, the pressure also works in the vertical direction, and if this pressure exceeds the vertical soil pressure this will trigger an uplift and possibly a breaking out of soil layers, and apparently that is what has happened here. In their paper on face support Jancsecz and Steiner (1994), for this reason gave a warning about the limits to the face support pressure, for situations with little overburden.

Research learns that for the fine sand that we have in the Pleistocene sands layers in the Netherlands, penetration of bentonite in the pores is negligible. Excavation therefore means removal of the cake sealing; Research by Bezuijen and Brassinga (2001), indicates that it normally takes about 4 to 5 minutes to build up a new cake sealing after the excavation wheel has removed the sealing during excavation. The time between passings of chisels, in the order of 20 seconds is too short for that. It must be emphasized that this effect is not only important for the upper limit to face support pressures, but may also give a limitation to the lower limit of the support pressure. A method to discount for this effect was given by Broere (2001), see also Fig. 4.

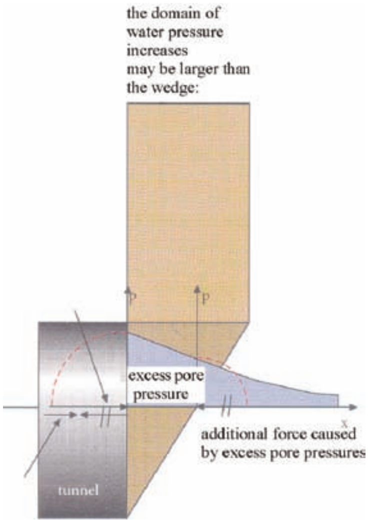

Figure 4. The effect of removal of the cake sealing during excavation on pore-pressures in the front. The influence zone for excess pore-pressures may be larger that the zone normally considered in stability analysis.

The situation of a low soil cover underneath the river bottom is not the only situation that might be critical to the above effect, also if the soil cover itself is relatively light, such as in the case of the thicker layers of peat overlaying the sand where the Green Hart Tunnel was excavated, this might lead to a critical situation. A local failure might be triggered where the generated excess pore pressure in front of the tunnel face can lift the soft soil layers.

The knowledge gained with the monitoring of the 2nd Heinenoord tunnel was applied for the Green Hart tunnel, and may have prevented instabilities at the bore front at larger scales; see Bezuijen et al. 2001 & Autuori & Minec (2005).

3.2 *Tail void grouting and grouting pressures*

To measure the soil pressures on a tunnel lining is difficult. In the start-up phase for the monitoring of the 2nd Heinenoord Tunnel, a number of international experts on tunnel engineering advised not to put too much effort on this topic, as "the results would probably be disappointing". Due to the hardening of the grout, the period for meaningful pressure measurements would be short and to prevent bridging effects the size of the pressure cells would have to be large and therefore costly.

Still, against this advice, the measurement of grouting pressures was undertaken, and repeated for a number of tunnel projects. It appeared that the interpretation was difficult when the grout has hardened, but for the fresh grout until 17 hour after injection it was possible to give an accepted interpretation of the measurement results (Bezuijen & Brassinga, 2004), and a lot of experience has been gained that has contributed to a better understanding of the grouting process and the pressures acting on the tunnel lining. With these results it was possible to predict grouting pressures and tunnel loading, see Talmon & Bezuijen (2005).

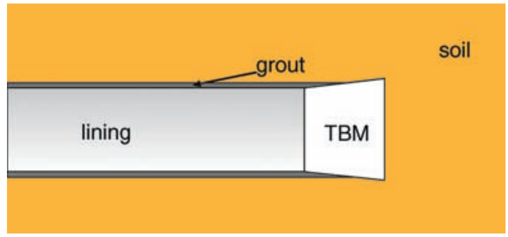

Figure 5. Under circumstances the Grout material from the tail void might flow into the gap behind the tail of the TBM, giving cause to increased loads.

Based on various evaluations of the force distribution in the tunnel lining, see amongst others, Bakker (2000), it came forward that the initial in-situ soil stresses around the tunnel do not have a dominant influence on the compressive loading of the tunnel. Due to the tapering of the TBM and in spite of the tail void grouting there is a significant release of the radial stresses around the tunnel, see Fig. 5.

The final loading on the lining relates more to the effectiveness of the grouting process, the distribution of the grout openings and the consolidation of the grout than to the initial in-site soil stresses, see Bezuijen et al. (2004). Whether this reduction of the in-situ radial stresses is a lasting effect that will remain for the full lifespan of the tunnel may depend on the creep sensitivity of the soil, see also Brinkgreve and Bakker (2001), and Hashimoto et al. (2008).

3.3 *Surface settlements*

Hoefsloot et al. (2005), have shown that the application of a stress boundary condition between tunnel and soil in 3D tunnel analysis has a better corroboration between measurement and calculation of soil deformations around the tunnel and subsequently of the loading on the tunnel, than the application of the so called "contraction method".

Although this effect was known in the literature, see for example Mair and Taylor (1997), for the research team that worked at the 2nd Heinenoord tunnel the observation that the numerical predictions of surface settlements lacked accuracy was disappointing. At the start the expectations on numerical analysis had been quite high. Shortly after the first observations were evaluated it was realized within the team, that it were only the empirical predictions by Peck (1969) for a

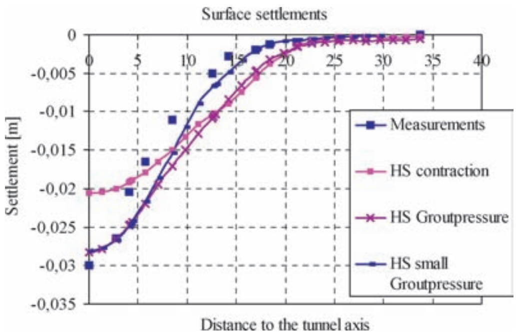

Figure 6. Surface settlements; measured and back-calculated with different material models.

volume loss of about 1% that gave a reasonable corroboration with the measurements. The finite element calculations, at that time mainly based on an application of the elastic-plastic Mohr-Coulomb model in combination with a contraction type of modelling for the tail void loss, predicted a too wide and too shallow surface settlement.

This disappointing result created a problem for the intentions to apply 3D numerical analysis in deformation predictions for tunnel projects in urban areas, such as for the Amsterdam North-South line metro works.

Since then, however, a lot of effort has been put in the improvement of the numerical prediction of soil deformations. To begin with it was the project team for the Amsterdam Metro works, see Van Dijk & Kaalberg (1998), that gave a first indication for an improvement, with the proposal to model the stresses at the tunnel soil interface instead of the deformations. With the introduction of this grout pressure model the results improved. Later on, when the physics in the process became better understood, i.e. the importance to account for the high stiffness of the soil in unloading, double hardening was introduced with the development of Hardening Soil, as a material model; with this development, the calculation results largely improved compared to the measurements, see Fig. 6. The latest development is the introduction of small strain stiffness in the Hardening Soil Model, see Benz (2006), which up to now gives the best results, see Möller (2005).

Theoretically the result might further be improved introducing anisotropy in the model; such models are being developed nowadays, e.g. in the framework of European Research; AMGISS, e.g. see www.ce.strath.ac.uk/amgiss.

4 EVALUATION OF THE LEARNING ISSUES

Nowadays it's not the soil deformation during "normal" excavation process that makes us worry about surface settlements. With an average tail void loss of about 0.5% of the diameter or less, the deformation might only be a problem for situations of under-excavation of buildings or if the structures are located very close to the excavation track. For tunnels in urban area, there is more concern with respect to bore-front stability; especially when the upper stratum of the soil above the Pleistocene layers, where the tunnels are usually positioned in, consists of soil with a relative low density, as in the Netherlands. For these situations with relatively light upper layers of peat or clays with organic parts, one has to be very careful controlling the support pressures during excavation, as on the one hand there is a lower bound value of the support to prevent cave in, but on the other hand, the upper limit triggered by an uplift of light upper layers may also be not far. This will limit the pressure window to work in.

Front instability has occurred at various tunnelling projects in the Netherlands. If the tunnel is outside any urban area this might not give too much problems; however if the tunnel is underneath a city road system, or close to pile foundations this may cause severe problems, as instability might cause a sinkhole in the pavement and foundation settlements.

With respect to the accuracy in the prediction of soil deformations: Apart from the well known empiric model of Peck (1969) that predicts the shape of the settlement trough but not the volume loss, the numerical models have become quite reliable in predicting surface and subsurface deformations, both vertical and horizontal. The improvement, mainly achieved in 2D analysis has opened up the possibility for a reliable deformation analysis in 3D of tunnelling in urban areas. For an adequate prediction of deformations it is important to model the grouting pressures as a boundary condition to the excavation, in combination with the application of a higher order material model, that takes into account the small strain deformation behaviour of sand, see Benz (2006).

Further it is recommended, and planned for, to integrate the Delft Cluster Grout pressure model in the Plaxis 3D Tunnel software. The latter would contribute to the applicability of the numerical models as a more general tool for underground construction. This would enable a better analysis for the loading on the tail of the TBM and of the tunnel lining.

Within certain limits some cost saving structural improvements are expected to be possible and, even more important, insight is obtained in the mechanism involved.

5 CONCLUDING REMARKS

Ten Years have passed since the first large diameter bored tunnelling project in the Netherlands in Soft soil was undertaken. Since then some world records with respect to tunnelling have been broken in the Netherlands; i.e. the largest diameter (for the Green Hart

Tunnel), the highest outside pressure on a segmental tunnel (for the Westernscheldt Tunnel), the application of an Earth Pressure Balance shield in coarse sand, and the largest length of constructed tube in one day, (Pannerdensch Canal Tunnel).

Before the underground construction works were started, and the tunnelling projects were in a pre-design stage, the softness of the Netherlands underground attracted a large part of the attention, see Bakker (1997). In retrospect the influence of a low stiffness as a source of risk and influence on underground construction was confirmed, but sometimes in a different perspective, or related to other physical processes than foreseen.

With respect these new insights the following conclusions were drawn:

1 The low stiffness of the soil may also lead to increased flexibility of the tunnel tube. The deformation of the tube during hardening of the grout, and the additional Eigen stresses that this may cause is still a research topic.
2 For a proper prediction of surface settlements and soil deformations, it is important to model the grouting pressures at the interface between soil and tunnel (or grouting zone). Further to improve the prediction of the width of the settlement trough, the use of small strain analysis is advised.
3 During excavation in fine sand, such as the Pleistocene sand layers in the Netherlands, the supporting cake fluid will be removed by the chisels on the excavation wheel. Therefore, in cases of limited overburden the upperbound to the support pressure must be carefully determined to prevent instability of the overlaying soil.
4 In addition; for the determination of the lower limit to the support pressure, the increased pore pressures in the front also needs to be taken into account.

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