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Ten years of bored tunnels in 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 (1997), gave an overview of the possible 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.

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 con-structed 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 pro-jects, 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. After completion, the pilot projects have triggered the start of a series of other bored tunnelling projects.

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 (1997). Since then, the 2nd Heinenoord tunnel, see figure 1, and a series of other bored tunnels were constructed. After the pilot projects a Joint Platform for Bored tunnels was established (GPB) that coordi-nated 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 Con-struction; 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.

Unquestionably a lot has been learned from the performed monitoring and research. The results of this process have been noticed abroad. In 2005 the Neth-erlands hosted the fifth International symposium of TC28 on *Under-ground 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.
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 gives some highlights of the main research result of the past decade.

Revieuw 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 combi-nation 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 expe-rience 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 figure 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 foun-dations 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 are buoyancy effects; the effect that the tunnel might be

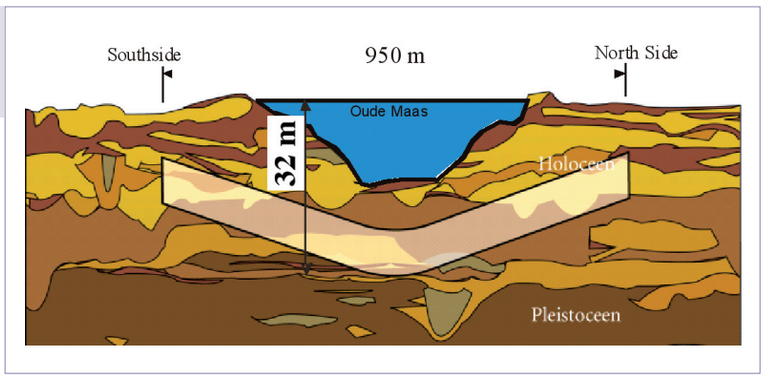

Figure 1 *Geological profile at the 2nd Heinenoord tunnel*

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 tunnel 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).

Experiences with bored tunnels in The Netherlands in the past decade

1 Structural damage

An early experience with the difficulties for bored tunnels in soft ground was the damage to the lining that occurred during the first 150 metres of construction of the 2nd Heinenoord Tunnel. On average the damage was too high compared to experiences from abroad and was considered to be unacceptable. Although, the integrity of the tunnel was not at stake, there was worry about the durability of the tunnel and the level of future maintenance.
Characteristic to the damage was cracking and spalling of concrete near the dowel and notches, *see figure 2*. Quite often the damage was combined with differential displacements between subsequent rings and with leakage. The evaluation report, see Bakker (2000), attributed the damage to irregularities in the construction of the lining at the rear of the TBM and subsequent loading during TBM progress. Further a correlation of the damage with high jack forces was observed; these appeared to be necessary to overcome the friction in this part of the track, which prevented smooth progress.
With respect to the tunnel ring construction, it is difficult to erect a stress free perfect circular ring. The ring needs to be built onto the end of a former ring that already has undergone some loading and deformation from the tail void grouting while it partially has left the tail of the TBM, see figure 3.The further deformation is characterised by the trumpet shape of the tubing that it

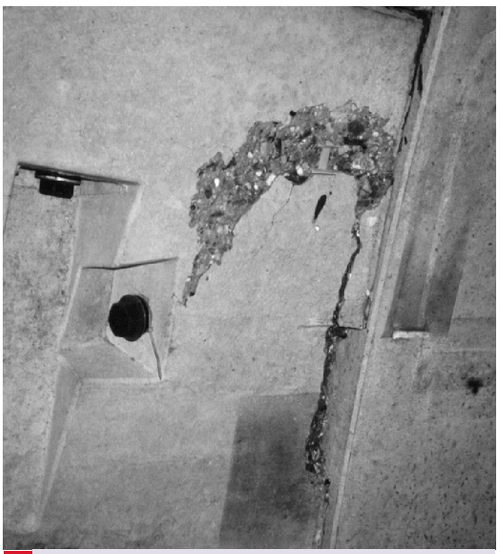

Figure 2 *Damage to the Dowel and notch sockets*

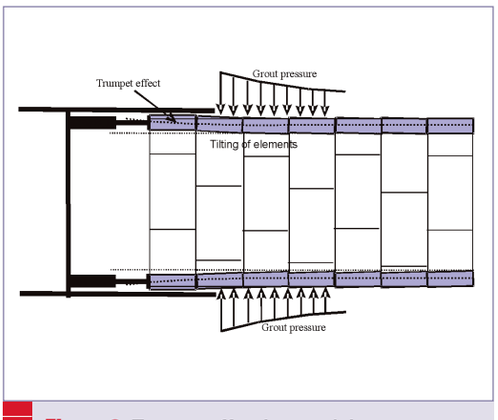

Figure 3 *Trumpet effect in tunnel ring construction of the (gray) inner ring is 8.3 m.*

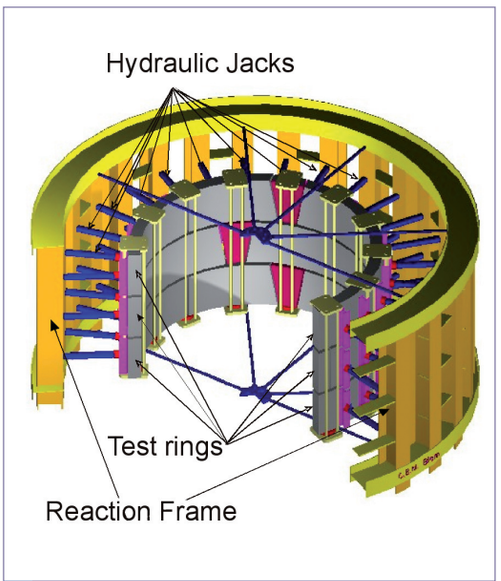

Figure 4 *Large-scale tunnel ring testing in the Stevin Laboratories at Delft University (the diameter of the (gray) inner ring is 8.3 m.*

		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	8620	*single*	30	14.90

Table 1 *Bored tunnels completed after 1997 in the Netherland*

causes, with the inevitable related stress development in the lining. The trumpet shape and the high jacking forces lead to local stress concentrations and irregular deformations in the lining and occasional to slipping between the different tunnel elements. The slipping of elements was blamed to the use of kaubit in the ring joint.

Originally kaubit strips had been used in the ring joint. These kaubit strips, of flexible bituminous like material, were used to prevent the occurrence of stress concentrations; so some slipping was meant to occur, but the "dynamic" character of the slipping that actually occurred that influenced the final geometry of the lining and had triggered cracking was unexpected. Especially the cracking and overloading of the dowel and notch system was unforeseen.
Failure of the dowel and notch system, *see figure 2*. led to spalling and in some cases to leakage. In the cases that leakage was observed this must have been correlated to damage to the notch at the outer side of the lining, creating a shortcut to water penetrating behind the rubber sealing there.
After the main conclusions were drawn, it was decided to exchange the kaubit strips for thin plywood plates. Due to the larger stiffness and shearing resistance, shearing of the concrete elements at large was further prevented and the damage limited.
Besides this technical measure, the evaluation was the trigger for the undertaking of fundamental research into lining design, that included large scale physical testing of tunnel tubing at Delft University, see figure 4. In this project that was a combined effort of physical and numerical testing, the details of assembling tunnel segments into subsequent tunnel rings and these into a tube were investigated. Amongst others the main results of the project were reported by Blom (2002), and Uijl et al (2003).
This research was fundamental for the choice to omit the dowel and notches for the Green Hart tun-nel; which led to a nearly damage free tunnel lining.

A different issue, not settled yet, is the durability of plywood and the consequences of wood rot on the long-term tunnel behaviour. An unwanted loss of the longitudinal pre-stress of a tunnel might influence the tunnel flexibility and deformations, possibly leading to leakages. On the other hand, experience learns that compression largely increases the durability of wood. The ply wood material is compressed to a strain of more than 50% during tunnel construction. At such a high level of straining the wood cells might have collapsed.

2 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, see figure 5; afterward commonly re-ferred 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, who 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 figure 5 (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

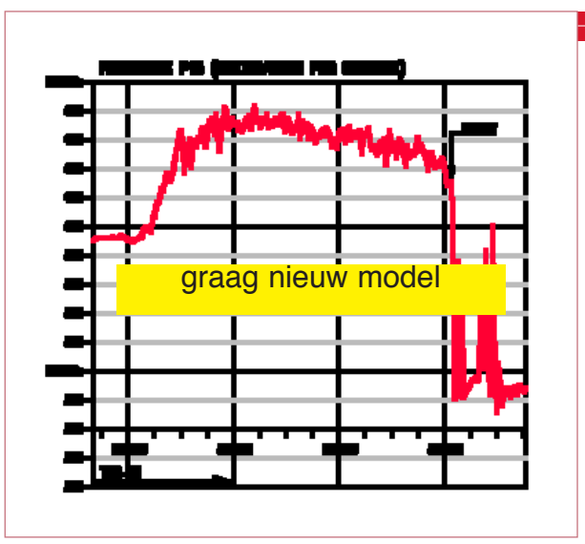

Figure 5 *Support pressures before, during and after the "Blow out" at the 2nd Heinenoord tunnel*

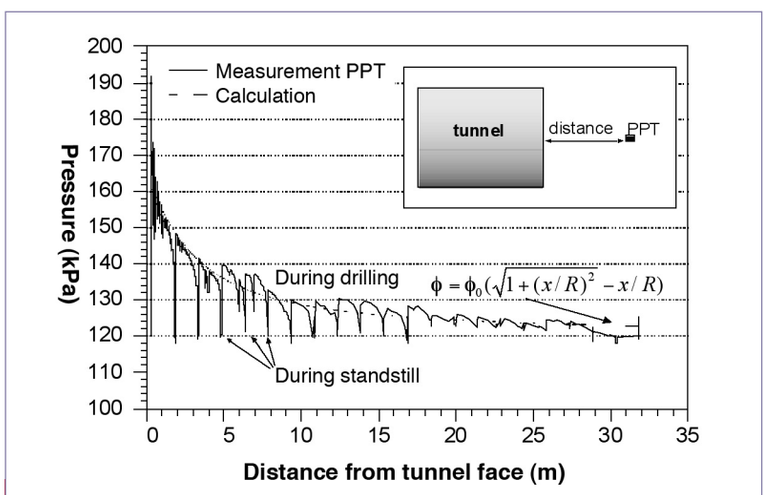

Figure 6 *Pore water pressure distribution in front of the TBM*

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 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 et al (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 figure 7.*
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).

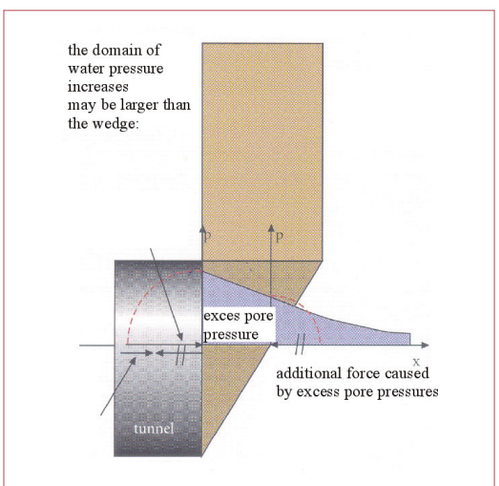

Figure 7 *The effect of removal of the cake sealing during excavation on pore-pressures in the front. The influence zone for excess pore-pressures may be larger that the zone normally considered in stability analysis.*

3 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 interna-tional experts on tunnel engineering advised not to put too much effort on this topic, as *"the results would probably be disappointing"*. Due to the hard-ening of the grout, the period for meaningful pres-sure measurements would be short and to prevent bridging effects the size of the pressure cells would have to be large and therefore costly.

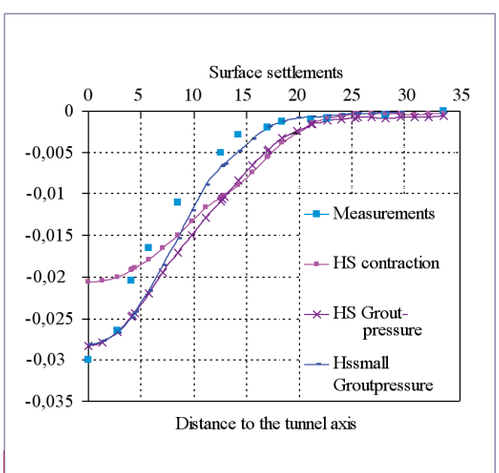

Figure 8 *Surface settlements; measured and back-calculated with different material models.*

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 et al, 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).

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 figure 8.*
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 Brinkgreve and Bakker (2001).

4 Surface settlements

Hoefsloot et al. (2005), have shown that the applica-tion of a stress boundary condition between tunnel and soil in 3D tunnel analysis has a better corrobora-tion 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 (1997), for the research team for 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

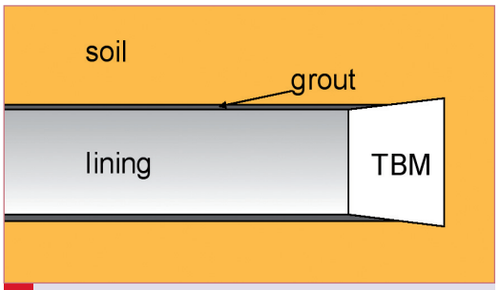

Figure 9 *Under circumstances the Grout material from the tail void might flow into the gap behind the tail of the TBM, giving cause to increased loads.*

Figure 10 *Test site for the Pile-tunnel interaction test .*

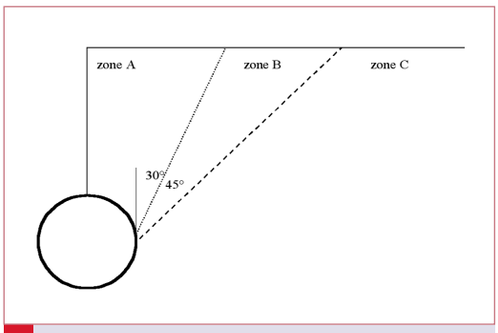

Figure 11 *Zones that indicate different effects on piles foundations.*

observations were evaluated it was realized within the team, that it were only the empirical predictions by Peck (1969) for a 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. However, since then a lot of effort was 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 figure 8.* The latest development is the introduction of small strain stiffness in the Hardening Soil Model, see Benz (2006), that 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.

5 Deformations of the TBM machine during con-struction of the Westernscheldt tunnel

During construction of the first tube for the West-ernscheldt tunnel, unexpected deformations of the tail of the TBM were observed; i.e. the air space be-tween tubing and tail of the TBM narrowed at a cer-tain stage in an unexpected way. The shape of the observed deformations did not coincide with the as-sumed soil loading and gave the impression that it was a large deformations effect; i.e. buckling. However, at first buckling was not accepted as a cause because the tapering of the TBM was assumed to give sufficient stress release to guarantee a suffi-cient decrease in isotropic stress. Further a certain bedding effect was assumed to be always present and the combination would make buckling unlikely. Buckling would only be plausible for a much higher loading of the tail of the TBM in combination with the absence of any bedding reaction. However, the insights have changed since then. In general there may be no overall contact between the soil and the tail of the TBM; when grout is injected in the tail void, the increased pressure on the soil, compared to the original stress will push the soil from the TBM and grout will flow between the TBM tail and the soil, see figure 9. This means that the pressures on the TBM tail are higher than anticipated in the past and there might be no bedding reaction. This could well explain the occurrence of buckling and the deformations of the TBM tail. A 1-D calculation model has been developed and is verified with FEM simulations (Bezuijen & Bakker, 2008). This model shows that also the high stiffness of soil during unloading, which led to the HS and the HSsmall material models, made it likely that the common tapering, approximately equal to an equivalent volume loss of 0.4 %, is sufficient to lose the larger part of the effective radial stresses, which helps to develop a gap between the tail of TBM and the soil.
The grout pressures exerted on the tail of TBM might be much higher that the soil stresses, and in absence of bedding, buckling could well explain for the deformations.

6 The influence of tunnel boring to piled foundations

Large scale testing of pile foundations was per-formed during construction of the 2nd Heinenoord Tunnel. This was done in order of the Project Bu-reau of the Amsterdam North/South metro works to get a better understanding of the processes,
A trial field with loaded piles and pile configurations was installed in the area near and above the track of the TBM, see figure 10. One of the main concerns was that due to an increase in pore pressure the effective stresses around the pile tip might be affected and that a release in isotropic stresses might trigger a drop in pile bearing capacity.
However, against this reasoning there is also numerical and analytic evidence, (assuming cylinder symmetric analysis), that indicates that the release in stresses due to tunnelling is limited to a rather small plastic zone in the close vicinity of the tunnel lining, see also Verruijt (1993).
The analytical model reveals that strain as a function of the distance drops as a function of, which would indicate that the influence zone would be limited in size. This reasoning in combination with the fact that the strains due to tunnelling in general are quite small; the largest strains often being less than 0.5 or 1.0 %, makes plastic zones further away than D/2, measured from the tubing, unlikely. Only above the tunnel this zone can be larger.
However, reasoning and analysis is one thing; measuring and validation is another; based on the field measurements and physical model

research in Delft and Cambridge Kaalberg et al. (2005), proposed a zoning as shown in figure 11, with the following indicators; a zone 'A' above the tunnel where the settlement of a pile is expected to be larger than the soil deformations. A zone 'B' adjacent to the tunnel, with an inclined influence line, where the pile will follow the soil deformation at the tip of the pile, and further a zone 'C', outside Zone B, where at soil surface level the settlement of the pile will be less than that of the soil surface. This zoning proposal more or less coincides with the main results as published by Selemetas (2005) that were mainly based of physical testing in a geotechnical centrifuge.
The results published by Kaalberg et al. and others are valid for the average volume loss that can be expected during tunnelling (0.5 to 1%) Earlier centrifuge testing by GeoDelft indicated that larger deformation effects are possible for higher volume losses (up to 7% was tested). Such volume losses are well above nowadays practice, but it means that during a calamity, piles over a larger area may be affected

7 Longitudinal deformations of the tunnel tube

In a paper by Bakker (1997), the development of longitudinal stresses in a tunnel lining due to irregu-lar bedding in soft soil was mentioned as an item for research. Irregular bedding that could be the result of zones with different elasticity or else due to the stiff foundation of a shaft or bedding in the deeper Pleis-tocene layers; especially near the transition between Holocene and Pleistocene layers.
The measurement of longitudinal stresses in itself has turned out to be cumbersome. Within the monitoring scheme for the 2nd Heinenoord a trial measurement was undertaken. In addition to that measurements from the Sophia Rail Tunnel were back-analysed with 4D finite element analysis, and after that the longitudinal stresses were also measured during the construction of the Green Hart Tunnel.
To begin with the latter situation: measurements were taken with a tubular liquid level devise of the longitudinal deformations of the tunnel during the grouting process. From these measurements the observation came forward that the tubing exhibited large vertical movements, up and down, between 20 to 30 mm during excavation and tail void grouting was measured, and a total vertical shift of the tubing vertical of about 60 mm at one location (See also Talmon & Bezuijen, 2008).
This amplitude was surely unexpected and is not fully accepted yet. Nevertheless it is clear that vertical deformations do occur in the zone where the grout material is still fluid, and during excavation and may lead to an alternating deformation; upwards when the TBM is excavating and during grouting and downwards if the TBM is at stand still.
With respect to the 3D staged construction analysis of tunnel construction for the Sophia Rail Tunnel, that was undertaken for the COB F220 committee, a combined DIANA and PLAXIS 3D analysis was performed, see Hoefsloot et al, (2005). The outcome of these various analyses more or less coincided; which might have been expected as the mathematical base of both models is quite similar, and in general deformations remain small, so the soil reactions will most probably have mainly been elastic.
The main conclusion with respect to this effect was that this issue can be properly analysed with a relatively simple model based on the concept of a beam with an elastic bedding and a series summation, such as developed by Boogaards (1999), and later on applied by Hoefsloot (2002). For the model concept see figure 12 and figure 13 for a comparison between model outcome and measurements. However, using generally accepted parameters, the measured deformations are much higher than according to these models. Recently, Talmon et al. (2008) have presented results that may explain the lower stiffnesss that are found in the measurements (the lining stiffness can be lower due to only local contact between the elements and the soil stiffness reduces due to unloading of the soil around the tunnel), but these are not yet generally accepted.

Cross passages

The design for the Westernscheldt tunnel in the Netherlands did trigger a debate on tunnel safety. Some major accidents with tunnel fires, such as oc-curred in the Channel tunnel and at the Mont Blanc tunnel in the Alps did reveal the vulnerability and relative unsafe situations in tunnels with oncoming traffic or in a single tube in general.
For the Westernscheldt tunnel, a twin tunnel with one way traffic per tube, the discussion focussed on what distance between cross passages would be acceptable to guarantee that escaping people would be able to find a safe

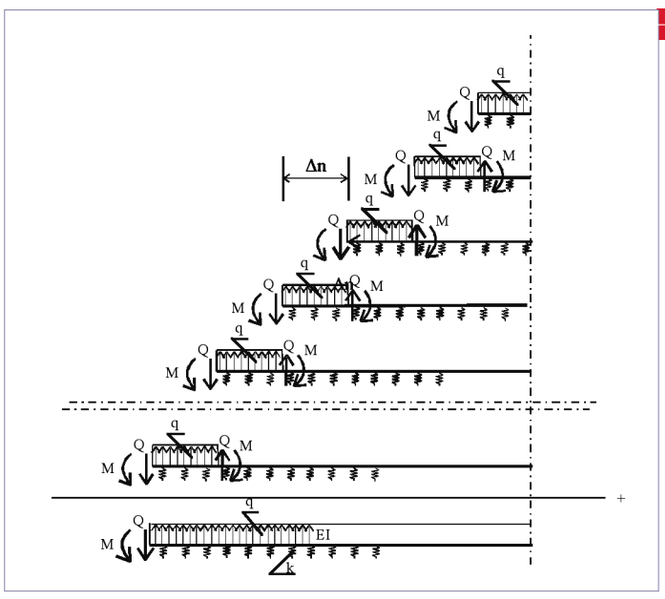

Figure 12 *Conceptual model for the analysis of beam effects in the tube of a bored tunnel by Boogaards (1999) Thorbecker-veld Gouda.*

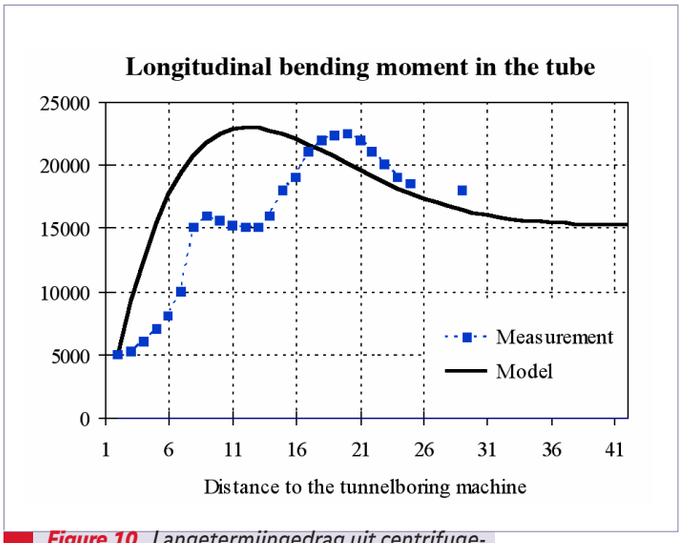

Figure 10 *Langetermijngedrag uit centrifuge-test, Stewart, Jewel & Randolph, 1996* [11, 12].

haven by entering the other tube; assuming that the traffic is stopped, by an automatic control system. The outcome of these safety studies was a cross connection at least every 250 m, which is nowadays more or less the reference situation in the Netherlands.
The task to construct these cross passages is a further technical effort. During the construction of the Botlek Rail Tunnel a vertical shaft and freezing were the main construction techniques as the cross passages could be positioned outside the area under the Oude Maas River. The positive experience with freezing for the Botlek Rail Tunnel was helpful in the decision making for the Westernscheldt Tunnel, but there the freezing was done from the tunnel tube as the track underneath the estuary is too long and too deep with respect of the water table to enable the shaft type method.
Although the method in itself is costly, its reliability is an important advantage and therefore it is also used for the cross passages of the Hubertus Tunnel and is expected to be used in future projects. For the single tube Green Hart Tunnel tunnel safety is achieved by construction of a separation wall with doors.

Evaluation of ther learning isues

The research on grout pressures, in combination with the structural research on lining design has gained us the insight that the lining thickness and the necessary reinforcement are mainly determined by the loading in the construction phase and to a lesser degree to the soil pressures. In engineering practice the thickness and reinforcement of the tubing is mainly determined by the most unfavourable jack-forces during TBM excavation in combination with an unfavourable tail void grouting scenario. Diffi-culty with these is, that it's the contractor's preroga-tive to decide on the necessary jack-forces that will enable him to construct the tunnel and also what scenario he will use for the tail void grouting. This might lead to conservative assumptions in the design office in order to avoid liabilities if a problem would occur during construction.
With respect to the generality of this conclusion it has to be considered that the main observations that were discussed relate to tunnels that are safely located in stiff Pleistocene sand layers. We must however consider the possibility of tunnels in softer soil layers that are more susceptible to consolidation and creep. The consolidation and creep can counteract the general tendency of stress release and arching in the soil and lead to a much higher radial loading. One may think of a loading that may be on the level of the initial soil stresses before tunnel construction; the Ko stress situation or even higher.
Such a situation was accounted for in the design for RandstadRail in Rotterdam, where a full steel lining was chosen for a part of the track where the tubing mainly rests in the upper much softer Holocene clay and peat layers, that foreseeable would have an extra loading on the lining due to consolidation and creep (Pachen et al. 2005).
Nowadays it's not the soil deformation during "normal" excavation process that makes us worry. 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 the situation of a relatively light upper stratum with peat or clays with organic parts, one has to be very careful in control of 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.

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 seg-mental 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 ground support may give rise to increased vulnerability of the lining for jacking forces by the TBM during excavation. Care must be taken to precise shape of the elements and joints to prevent too high stresses during assembly.

2. 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.
3. 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.

4. During excavation in fine sand, such as the Pleistocene sand layers in the Netherlands, during excavation the supporting cake fluid will be removed. In case of limited overburden the upperbound to the support pressure must be carefully determined to prevent instability of the overlaying soil.

5. In addition; for the determination of the lower limit to the support pressure, the increa-sed pore pressures in the front also needs to be taken into account.

6. The stiffer Pleistocene sand layers might not always be able to follow the tapering of the TBM. It is expected that this may give rise to gapping behind the tail of the TBM. If grout penetrates this gap, this may cause higher loads on the TBM than is normally assumed.

7. No proof was found that tunnel driving in normal operation might give cause to loss of bearing capac-ity of piles. Settlements in general are related to the settlement of the ground and the position of the pile toe with respect to the zones indicated in figure 11. ■

With acknowledgement to the Netherlands Centre for Underground Construction for their consent to publish about the research they commissioned and coordinated, and with thanks to Cees Blom for the use of some of the figures.

References

– Autuori, P. & S. Minec, 2005, *Large diameter tunnelling under polders*, Proceedings 5th Int. Symposium on Underground Construction in soft Ground, IS-Amsterdam 2005.

– Bakker, K.J., P. v.d. Berg & J. Rots, 1997, *Monitoring soft soil tunnelling in the Netherlands; an inventory of design as-pects*, Proc. ISSMFE, Hamburg.

– Bakker, K.J. 2000, *Soil Retaining structures, development of models for structural analysis*, Balkema, 2000, Rotterdam.

– Benz, T. 2006, *Small strain stiffness of soils and its conse-quences*, Doctor Thesis, IGS Universität Stuttgart.

– Bezuijen, A. & Brassinga H.E.,2001. *Blow-out prsessure measured in a centrifuge model and in the field*. Proc. Int. Symp. on Modern Tunnelling Science and Techn. Kyoto.

– Bezuijen, A., A.M. Talmon, F.J. Kaalberg and R. Plugge, 2004, *Field measurements of grout pressu-res during tunneling of the Sophia Rail Tunnel. Soils and Foundations* vol, 44, No1, 41-50, Feb.

– Bezuijen, A. & A.M. Talmon, 2006, *Grout pro-perties and their influence on back fill grouting. Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005

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

– Bezuijen, A. & H. van Lottum (eds), 2005, *Tunnelling A Decade of Progress.* GeoDelft 1995-2005.

– Bezuijen & Bakker, 2008, *The influence of flow around a TBM machine, Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

– Bezuijen A. & A.M. Talmon, 2008, *Processes around a TBM, Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

– Blom C.B.M. 2002, *Design philosophy of concrete linings for tunnels in soft Soil*, Delft Univ. Press, The Netherlands

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

Brinkgreve, R.B.J. & K.J. Bakker, 2001, *Time-dependent behaviour of bore tunnels in soft soil conditions; a numerical study*, Proceedings, ICSMGE Istanbul, Turkey.

– Broere, W, 2001, *Tunnel face stability & new CPT applications*, Delft University Press.

– Hoefsloot, F.J.M. & K.J. Bakker, 2002, *Longitudinal effects bored Hubertus tunnel in The Hague, Proceeding 4th Int. Symposium on Underground Construction in soft Ground*, IS-Toulouse.

– Hoefsloot F.J.M. & A. Verweij, 2005, *4D grouting pressure model PLAXIS, Proceeding 5th Int. Symposium on Under-ground Construction in soft Ground*, IS-Amsterdam 2005

– Kaalberg F.J., Teunissen E.A.H., van Tol A.F. and J.W. Bosch, 2005, *Dutch research on the impact of shield tunnelling on pile foundations. Proceedings of 16th ICSMGE*, Osaka.

– Jancsecz, S. and W. Steiner, 1994, *Face Support for a large Mix-Shield in heterogeneous ground conditions, Tunnelling 94, British Tunnelling Association*, 5-7 July 1994

– Möller S.C. & P.A. Vermeer *Prediction of settlements and structural forces in linings due to tunneling, Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005

– Pachen, H.M.A., H. Brassinga & A. Bezuijen, 2005, *Geotechnical centrifuge testst to verify the long-term behavior of a bored tunnel, Proc. 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005.

– Peck, R.B., 1969, *Deep excavations and Tunnelling in soft Ground, Proceedings 7th ICSMFE* Mexico.

– Selemetas D., J.R. Standing and R.J. Mair, 2005. *The response of full-scale piles to tunnelling. Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam 2005.

– Talmon, A.M. & A. Bezuijen, 2005, *Grouting the tail void of bored tunnels: the role of hardening and consolidation of grouts. Proceeding 5th Int. Symposium on Underground Construction in soft Ground*, IS-Amsterdam.

– Talmon, A.M. & A. Bezuijen, 2008, *Backfill grouting research at Groene Hart Tunnel. Proceeding 6th Int. Symposium on Underground Construction in soft Ground*, Shanghai.

– Uijl, J.A. den, A.H.J.M. Vervuurt, F.B.J. Gijsbers and C. van der Veen, 2003, *Full scale tests on a segmented tunnel lining. In Proc. ITA World Tunnelling Congress 2003*, Amsterdam, The Netherlands, 12-17 April 2003.

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