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Ten years of bored tunnels in The Netherlands: Part II structural issues

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ABSTRACT: 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 risks related to bored tunnels in soft ground and explained about a plan for research related to the Pilot projects. Ten years have passed, a lot of monitoring and research has been done. In this paper that is split in two parts a summary is given of some of the most characteristic observations of these past 10 years of underground construction in the Netherlands. In this second part, the emphasis will be on structural related issues discussed whereas in part one, frontal stability, grouting and soil deformations are discussed.

1   INTRODUCTION

In 1992 the Dutch government sent a fact-finding mission to Japan, to report on the possibility 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 the report, that advised positive, 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 the construction of 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, and a series of other bored tunnels were constructed. Unquestionably a lot has been learned from all the monitoring and research that was performed.
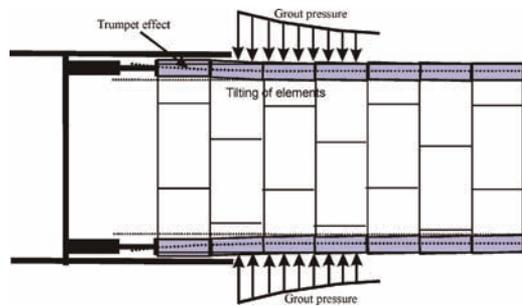

Figure 1.   Trumpet effect in tunnel ring construction.

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". 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.

In the present paper some highlights of the main research result of the past decade will be given. The paper is split in two parts, where part one includes some general observations and discusses face support,

grouting and surface settlements, whereas part two is more about structural issues.

2 REVIEW OF THE 1997 SITUATION AND WHAT CAME AFTER

A main concern with respect to boring tunnels in the Netherlands were 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.

Furthermore the 8.3 m outward diameter for the first large diameter tunnel was a major step forward, compared to past experience; experience that up to that time was mainly based on constructing bored tunnels, pipes or conduits up to about 4.0 m diameter.

In addition to that, in general 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 are buoyancy effects. Besides the risk of breaking up of the soft upper 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.

Ten years later, the question arises whether the observations have confirmed the above issues to be the critical ones. 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.

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

3.1 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 Fig. 2. Quite often the damage was combined with

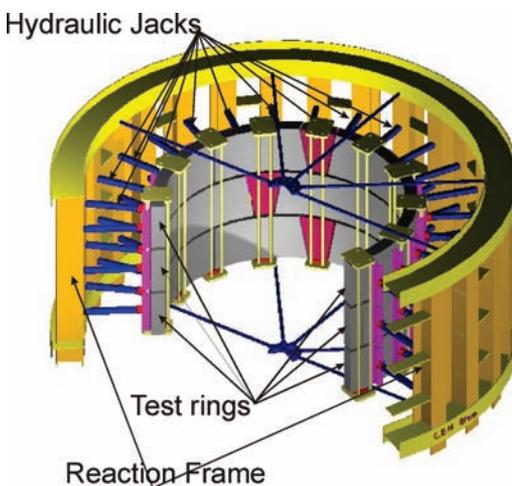Figure 2.   Damage to the Dowel and notch sockets.

Figure 3.   Large-scale tunnel ring testing in the Stevin Laboratories at Delft University (the diameter of the (gray) inner ring is 8.3 m).

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

Figure 4.   Test site for the Pile-tunnel interaction test.

Figure 5.   Zones that indicate different effects on piles foundations.

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 Fig. 3. The further deformation is characterised by the trumpet shape of the tubing that develops, see Fig. 1, 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 a bituminous material called 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 Fig. 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 Fig. 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),
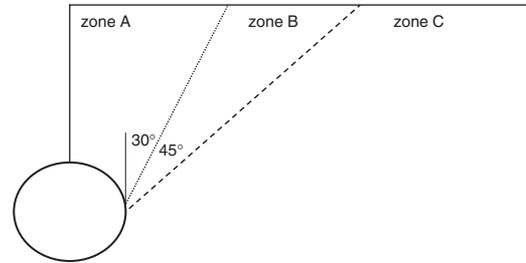
and Uijl et al (2003). Based on this research it was decided to omit the dowel and notches for the Green Hart tunnel; which led to a nearly damage free tunnel lining.

A different issue, not settled yet, is the durability of plywood and the consequences of wood rot on the long-term tunnel behavior. 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.

3.2   Deformations of the TBM machine during construction of the Westernscheldt tunnel

During construction of the first tube for the Westernscheldt tunnel, unexpected deformations of the tail of the TBM were observed; i.e. the air space between tubing and tail of the TBM narrowed at a certain stage in an unexpected way. The shape of the observed deformations did not coincide with the assumed soil loading and gave the impression that it was a large deformations effect; i.e. buckling.

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 sufficient 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 Fig. 5 in part I of this paper. 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 HS_{small} 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 than the soil stresses, and in absence of bedding, buckling could well explain for the deformations.

3.3 *The influence of tunnel boring to piled foundations*

Large scale testing of pile foundations was performed during construction of the 2nd Heinenoord Tunnel. This was done in order of the Project Bureau 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 Fig. 4. 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 $1/r^2$, 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 Fig. 5, with the following indicators; a zone "A" above the tunnel where the settlement of a pile is expected to be larger than the soil displacements. 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.

3.4 *Longitudinal deformations of the tunnel tube*

In the paper by Bakker et al. (1997), the development of longitudinal stresses in a tunnel lining due to irregular 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 Pleistocene 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, i.e. (time dependant 3D 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 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 mainly have been elastic.
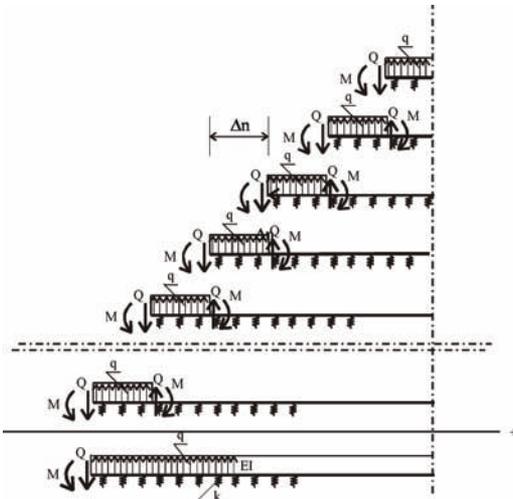
Figure 6. Conceptual model for the analysis of beam effects in the tube of a bored tunnel by Boogaards (1999).

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 & Bakker (1999), see Fig. 6 and later on applied by Hoefsloot (2002). See Fig. 7 for a comparison between model outcome and measurements from the 2nd Heinenoord tunnel.

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 stiffness 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.

4 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 occurred 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 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
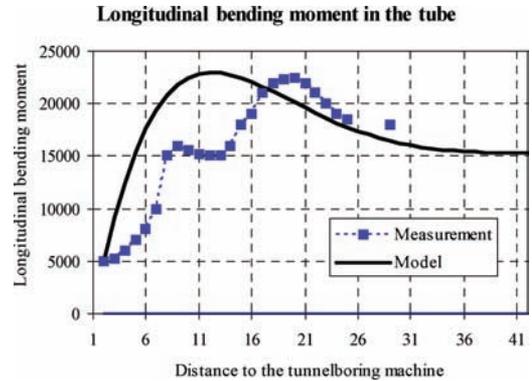

Figure 7. Longitudinal bending moment due to the construction forces measured and calculated acc. to Bogaards model.

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.

5 EVALUATION OF THE LEARNING ISSUES

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. Difficulty with these is that it's the contractor's prerogative 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. Consolidation and creep might counteract the general tendency of stress release and arching in the soil and lead to a much higher radial loading. One might think of a soil pressure on the lining that may be on the level of the initial soil stresses before tunnel construction; the K_o stress situation or even higher than these initial stresses. 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).

However, with respect to lining design, within certain limits some cost saving structural improvements are expected to be possible and, even more important, insight is obtained in the mechanisms involved.

6 CONCLUDING REMARKS

Ten Years have passed since the first large diameter bored tunnelling project in the Netherlands in Soft soil was undertaken. 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 to the new insights gained 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 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.

4 No proof was found that tunnel driving, in normal operation, might give cause to loss of bearing capacity 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 fig. 5.

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