The Second Heinenoord tunnel; the main monitoring results

Le tunnel 2ieme Heinenoord; Les resultats d'une project de monitoring

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ABSTRACT: During the construction of the Second Heinenoord tunnel, the first large diameter bored tunnel in the Netherlands, an extensive monitoring program was executed to gain knowledge and experience about bored tunnels in soft soil. In this paper the main results will be described. Conclusions are drawn with respect to bore technology; borefront stability and influence of boring and grouting on surface settlements. Furthermore the design of the tunnel lining is discussed, given the staged construction of the lining and thrust forces during boring.

RÉSUMÉ: Pendant la construction du tunnel 2ieme Heinenoord, le premier tunnel en Hollande executé dans le sol alluvionnaire doux, on a executé un program independant de monitoring pour ce gagner connaissance et experience en batir des tunnels dans sol doux. Dans cette contribution, les resultats plus importants sont expliqué. Les conclusions sont fait: aux cas des excavations a front de la machine, l'importance de faire une pressure de support pour assurez une situation stabilisé, modeler les pressures d'injection du grout pour calculer les deformation du surface. Et enphin les conclusions a ce cas du construction le conduit ou bout du machine, et l'influnce de Jack force a le tension dans le conduit du tunnel.

1 INTRODUCTION

In 1999 the Second Heinenoord tunnel was opened, extending motorway A 29 South of Rotterdam from 2 x 2 lanes to 2 x 3 lanes. In 1993 when the decision for this new tunnel was made it was decided to use this project as an opportunity for the development of underground construction in soft soil. Up to that time large diameter bored tunnels had not been constructed in the Netherlands, (Bakker et al 1997). In February 1997 TBM tunnelling for this project was started from the North Bank of the river "Oude Maas". The machine was turned on the South Bank to start the construction of the second tube in the reverse direction. The tunnel was completed in June 1998. The total length of the tunnel is 1350 m. (one way), including a bored section of 950 m. The cross-section consists of two tubes with an external diameter of 8.30 m. No cross connections between the tubes were constructed. The TBM was of a slurry support type.

The monitoring project was completed in 1999 and conclusions about the project were reported, (COB-K100 1999). The research was focussed on three main fields of interest:

- soil behaviour around the tunnel;
- boring technology;
- tunnel construction

In the next section a brief description will be given on the set up of the monitoring project. After that the main conclusion drawn from the project will be highlighted for each field of interest and finally the overall conclusions and recommendations will be given.

2 ORGANISATION AND EXECUTION OF TEST PROJECT

Monitoring of the Second Heinenoord tunnel project was undertaken under the auspices of the Dutch Centre for Underground Construction, CUR/COB. The project was a co-operation between public and private parties working together in commission K100, supported by a grant of the Dutch government. The total monitoring budget was about 6 million Euro.

Figure 1 The Second Heinenoord tunnel after completion.

Prior to tunnel boring predictions were carried out, (COB K100-04 1997), which simplified the evaluation of the measurements. The monitoring itself was divided in three parts: one for the Tunnel-Boring Machine, a second related to two subsoil monitoring fields; one on each bank of the river Oude Maas, and finally two instrumented rings where installed into the final tunnel lining (Bakker et al 1999b).

During construction of the tunnel, three incidents have occurred which influenced the planning of the project:

1. At the start of the project, unexpected damage occurred on tunnel segments due to a combination of soil loading and thrust forces.
2. Due to mall functioning of the computer system controlling the thrust forces, the TBM moved backward. When the jack pressures where released the TBM moved backwards for more than 15 cm damaging the rubber tail sealing system and causing a large inflow of water and soil into the tunnel. Repair and finally installation of an additional steel brush tail sealing system delayed the project more than 6 weeks.
3. A blow-out of bentonite support slurry into the river Oude Maas did develop that had to be stopped and sealed before the tunnel boring process could proceed.

3 SOIL BEHAVIOUR AROUND THE TUNNEL

3.1 *Monitoring fields*

On both sides of the river monitoring fields were instrumented with geotechnical measurement devices. Both fields referred to as the "north" and the "south" monitoring field, are rectangular with dimensions of 75 meter (length) and 50 meter (width). The average soil cover in the north field is 15 meter, in the south field about 13 meter. In order to measure the influences of the tunnelling process on the surrounding soil (deformations, soil and water pressures) an integrated measuring system was installed, see Figure2. The system consisted of settlement markers, inclinometers and extensometers to a maximum depth of about GL –26 meter, piezometers, soil pressure cells and water-pressure gauges.

3.2 *Soil conditions*

An extensive soil investigation program has been carried out in order to derive the geotechnical profile at the monitoring fields. A large number of in-situ and laboratory tests have been performed, including cone penetration tests, boreholes, vane-tests, dilatometer tests, pressiometer tests and triaxial testing (extension and compression).

Because of the presence of old river gullies, large variations in soil conditions occur, see Figure 4. In general the Holocene deposits extend to a depth of about GL –17 meter and consist of peat, clay and loose to medium dense sand layers. At the north monitoring field the Holocene layer consists of relatively more sandy material, while at the south field the soil is more peaty and clayey. Underneath these Holocene deposits 8 meters of dense to very dense sand occurs, followed by a 2-meter thick stiff silty clay layer. Beneath this clay layer mainly dense sand layers are found.

The tunnel, which cuts both through the Holocene and Pleistocene layers, was entirely driven under the ground water level, which (depending on some tidal mfluence) is at about 2 metres depth.

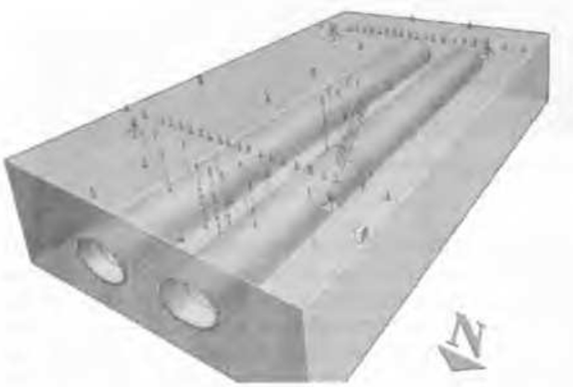


Figure 2. Birds-eye view of the geotechnical instrumentation

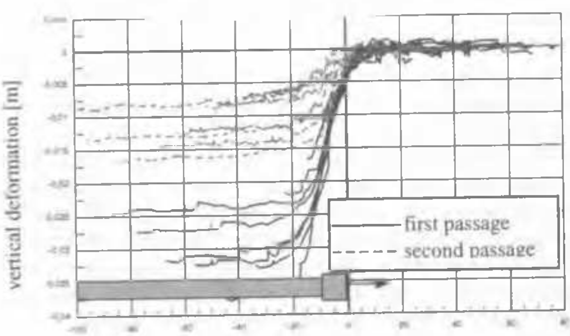


Figure 3 Green tield settlements above tunnel axis at Northern Monitoring Area

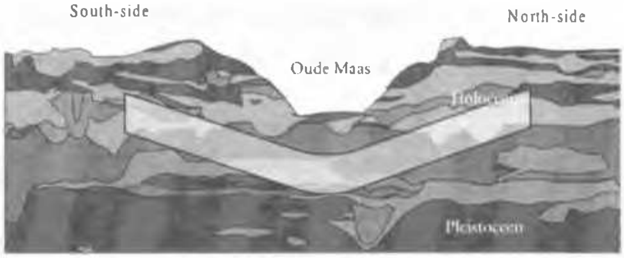


Figure 4. Geotechnical Profile at the 2nd Heinenoord Tunnel

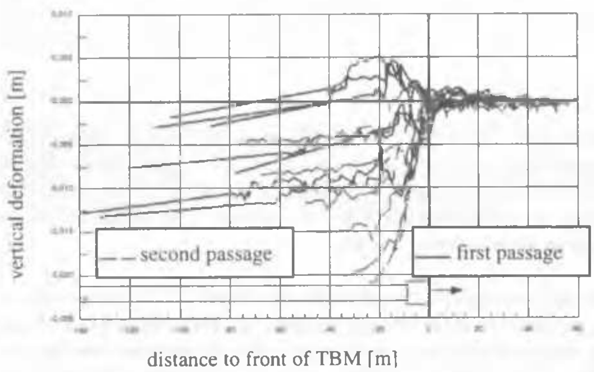


Figure 5 Green tield settlements above tunnel axis at Southern Monitoring Area

3.3 *Predictions*

Before construction, soil deformations and stress variations due to the tunnelling process were predicted using a number of different calculation models. Surface settlements were calculated using analytical and empirical formulas, such as those formulated by Peck (1969[a])and Sagaseta (1987). Both two- and three-dimensional finite element models were used to predict horizontal and vertical soil movements and stress variations. The results were compared with the measurements (see below).

3.4 *Measurements*

During the passage of the north ("sandy") monitoring field, up to a distance of about 7 meter ahead of the tunnelling machine, almost no settlement was observed. While the machine passed, the settlements rapidly increased and at a distance of about 25-meter behind the face the settlements remained constant see Figure 3. During the first passage of the field the maximum settlement varied between 22 and 37 mm, whereas during the second passage a maximum settlement occurred between 7 and 17 mm.

The contribution of the front effects to the total settlement was also during passage of the south monitoring field ("more clayey and peaty" than the north field) very limited. During passage soil deformations occurred. However, the maximum settlement in the southern field was considerably smaller than in the north monitoring field and varied between –3 mm (heave) to about 10 mm during the first passage and between 4 mm and 20 mm during the second, see Figure 5. Another difference with the north-monitoring field was that the settlement did not stabilise at a distance of 30 meter or more behind the face, indicating that under this soil condition time-dependent behaviour plays a role.

3.5 *Evaluation and conclusions*

The maximum surface settlement was highly dominated by tail void effects; front effects were negligible.

A relation was found between the maximum settlement and the injected volume of grouting material into the tail void. At the north-monitoring field the relation between both quantities was almost linear. Although at the south monitoring field the same

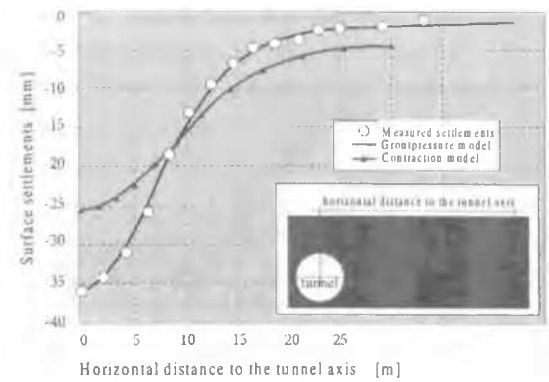


Figure 6. Comparison between calculated and measured settlement troughs: modelling a contraction and modelling the grouting pressures.

(yellow-green-blue = settlement, < 55 mm)
(red = heave, < 10 mm)

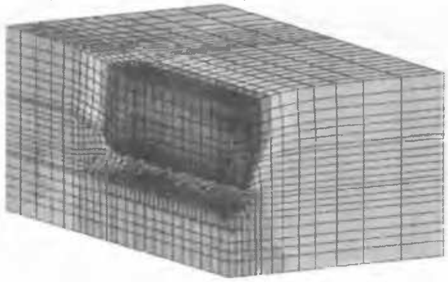


Figure 7 3-dimensional finite element analysis of surface settlements with the groutpressure model

tendency was observed, the relation was less distinct due to several reasons, such as time-dependent effects and the use of over-cutters. Therefore it is concluded that accurate settlement control requires adequate control of the grouting process.

Another important conclusion is related to the way of modelling. Predictions using finite element techniques modelled the tail void effects as a uniformly distributed contraction of the tunnel area. This assumption resulted into a relatively wide settlement trough, see Figure 6. The real trough was steeper and narrower than given by the numerical predictions. Afterwards the contraction model was modified into a grouting pressure model, see Figure 7, in which the pressure distribution within the tail void could be applied as a load on the surrounding soil. From these back analysed results it was concluded that the measured results could be reproduced very well using a grouting pressure model.

4 BORE TECHNOLOGY

4.1 *Introduction*

With respect to the third theme, tunnel boring technology, the following three main issues were investigated:

- Determining the stability of the bore front;
- Determining the forces during excavating;
- Determining the effectiveness of the bore process.

In this paper only the first item is described. In order to obtain more insight into the mechanisms, which determine the stability of the bore front, both during the excavation stage and during the standby stage of the TBM, water-pressures were monitored, directly in front of the TBM, and at the bore face. Normally the slurry pressure at the bore-front is kept slightly above the local pore-water pressure. Furthermore over-cutting was experimentally monitored.

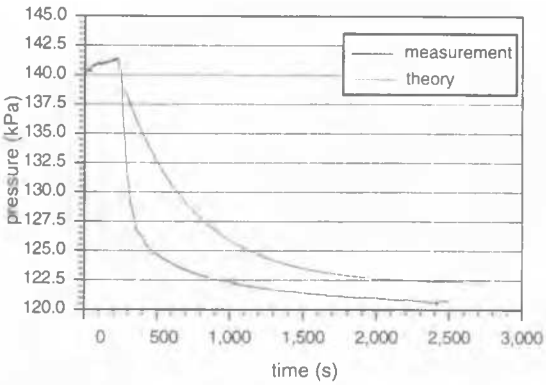


Figure 8. Pore pressure measurements as a function of time after stopping of the excavation of the TBM, 1.8 m from the measuring device

4.2 *Instrumentation*

Both the TBM and the subsoil were instrumented. At the TBM 16 piezometers were installed at the arms of the cutting wheel. These were measured with a frequency of 1 Hz during the construction phase of both tunnels. Other measurements were:

- density measurements of the bentonite input and the slurry output
- pressure cells at the 2x14 main jacks
- position of the cutting wheel
- bentonite level in the working chamber
- torque of the TBM
- temperature of the bentonite slurry
- pressure in the slurry pipe
  -wear of the cutters
- six piezometers in front of the TBM, some of which were "eaten up" by the TBM during crossing.

During excavation some experiments were carried out such as measuring the torque and the cutting forces, while variations where made in cutting wheel penetration.
The data from the instruments in the TBM were transmitted by two independent data acquisition systems, a trail ring and radio transmission. Both systems performed satisfactory.

4.3 *The effect of water pressures in front of the TBM*

The water pressure in front of the TBM was monitored both in sand layers and in clay layers, see the Figures 9 and 10, (Lengkeek & van Schelt 1999), which present the effect of the excavating stage on the water pressures. The average groundwater pressure at that depth is about 120 kPa. At the bore front an over pressure is applied, and a sealing of the front develops during standstills because the bentonite slurry forms a filter cake.

During rotation of the cutting wheel the filter-cake is scraped off continuously and bentonite flows into the soil. This leads to an increase of the pore-water pressure. This increase was measured as far away as 30 m from the TBM. When the TBM approaches the piezometer the pore-water pressure increases. In Figure 9, the last day before hitting the piezometer is shown. In this figure the differences between standstill periods and rotation of the cutting wheel are clearly visible.

The last minutes before hitting the piezometer are presented in Figure 8. In this figure, the influence of both the cutting as the drag teeth can be distinguished and also the 5 arms of the cutting wheel can be recognised. The measurements show that cutting wheel rotates at about 0.75 rpm. Negative pore pressures due to the cutting process have not been observed.
At another measurement point the TBM stopped just 1,8 meter in front of the piezometer. In Figure 8, (Bezuijen & Litjens 1999), the sealing effect of the bentonite cake is shown. During boring the water pressure is about 140 kPa. After standstill it takes

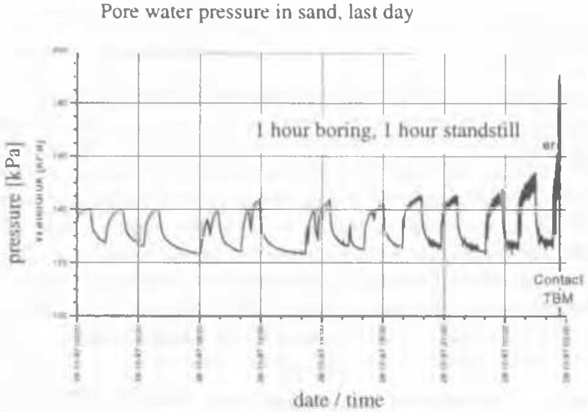


Figure 9. pore-water pressure in sand layer last day

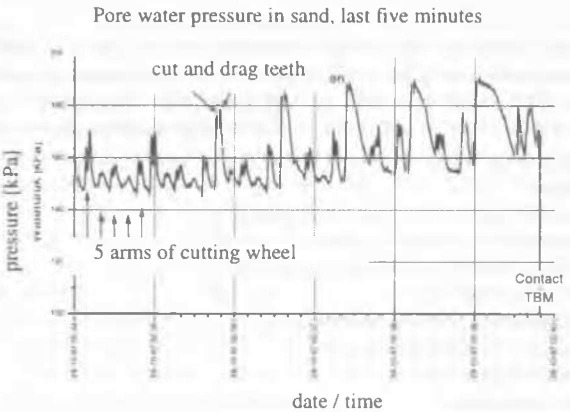


Figure 10. pore-water pressure in sand last minutes

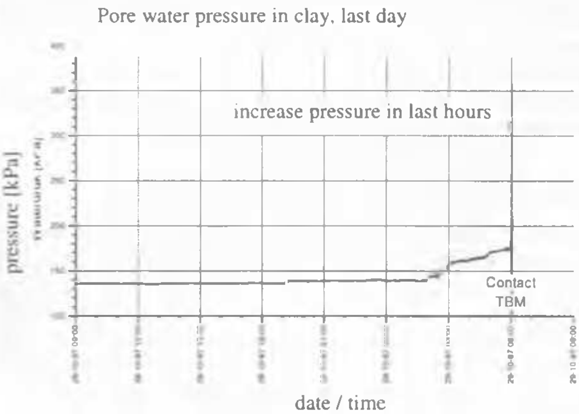


Figure 11. pore-water pressure in clay layer last day

about 200 seconds to build up the filter-cake. After 300 seconds the piezometer measures the average water pressures. This implies that during cutting, there is insufficient time for cake sealing to build up.

The increase of the pore pressure during excavation has an adverse influence on the bore front stability.

In figure 11 and 12, (Lengkeek & van Schelt 1999), similar measurements are presented, for a clay layer instead of a sand layer. As one can observe, during the advance of the TBM, the pore-pressure gradually rises up to the level of the support pressure in the TBM. This pressure rise even can be observed during standstill of the excavation process, which is being understood that no additional plastering of the excavation face develops in clay. It is obvious that due to the lower permeability of the clay the increase of water pressure during excavation does not takes place.

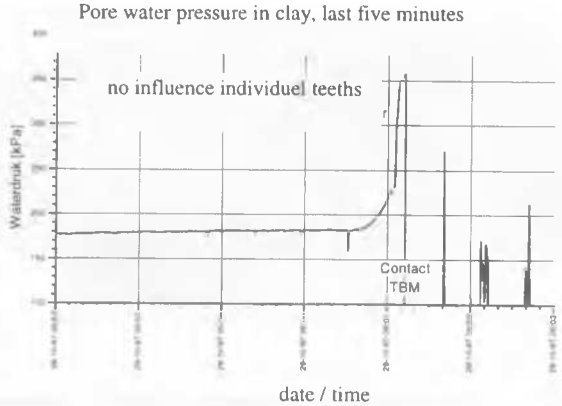


Figure. 12 pore-water pressure in clay last minutes

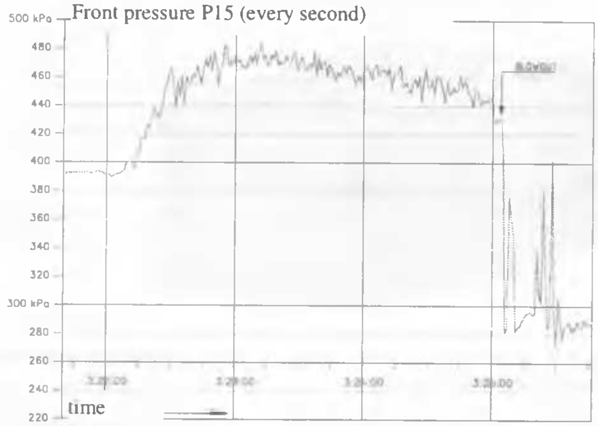


Figure 13 bore front pressure at time of the blowout

4.4 *The bore front stability during blow-out*

At the location of the lowest covering (about 8 m above the tunnel) under the riverbed, a blow-out occurred. During this blow-out all the instruments remained intact. In figure 13 the support pressures at the bore front are plotted against time.

Just minutes before the starting of excavation of a new tunnel ring, the pressure in the air chamber was increased to 450 kPa. This pressure is equal to the pore pressure plus 2.4 times the effective soil stress. During this period the cutting wheel was at a stand still. Within seconds after the cutting wheel was started, the support pressure dropped and the blow-out was a fact.

It might be concluded from these observations, that when a filter cake seals the bore front the maximum slurry pressure can exceed the maximum passive soil stresses without a blow-out.

The TBM operator prior to excavation of a new tunnel ring increases the bore front pressure. This is done to improve the discharge of slurry out of the bore chamber. When subsequently the cutting wheel is put in motion, the bentonite cake sealing is destroyed and the pore pressures in front of the TBM will equalise with the installed support pressure (450 kPa). At the location of the blow-out this installed support pressure was evidently too high and in a few seconds the pressure dropped to the ambient water pressure.

This blow-out showed that in sandy soils a sealing bentonite filter cake couldn't develop because the progress of the cutting wheel is much higher than the speed at which a filter cake is formed. Increasing the pressure in the bore chamber prior to starting the cutting wheel therefore leads to blow-out danger at critical locations.

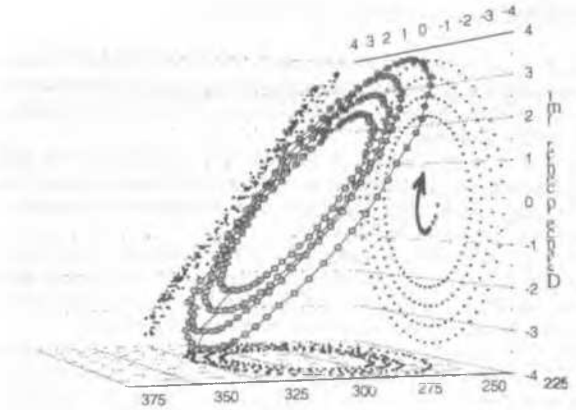


Figure 14 measurement bore front pressure of the cutting wheel during one turn

Figure 15. Damage to the tunnel lining; i.e. damage near the dowel and notch system

4.5 *Distribution of stresses at the bore front*

The distribution of slurry pressure at the bore front was another objective of investigation. In order to make a safe assessment of bore front stability, this distribution should be known accurately. During the second passage of the monitoring field this pressure distribution was measured repeatedly, for a number of tunnel rings, by pressure cells installed on the rotating arm of the cutting wheel. In Figure 14 pressures in the bore chamber are given for one complete rotation of the cutting wheel. From this figure, which is representative for a number of measurements, it is concluded that the pressure increases linearly with depth. This means that the density of the slurry in the chamber is constant. The measured density varied between 1260 kg/m^3 and 1450 kg/m^3, which is much higher then expected.

4.6 *Influence of applying overcutters*

As an experiment during the passage of one of the monitoring fields the contractor bored 37 rings using overcutters and directly afterwards 22 rings without overcutters. During that test the overall thrust forces were monitored. It could be concluded that the overall thrust force decreased with about 40 %, due to overcutting.

5 TUNNEL CONSTRUCTION

During the start of the construction of the tunnel tubes, i.e. along approximately the first hundred meters of construction of the tunnel tube, the lining showed more damage than expected. The damage appeared mostly near the edges of the tunnel segments,

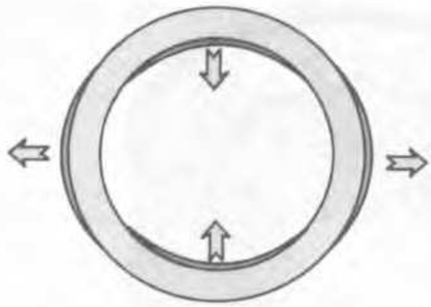
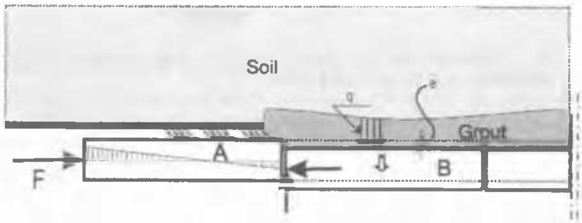


Figure 16. Differential deformations due to ovalisation



Figuur 17. Eccentricities due to partial loading during tunnel construction of the tunnel lining, where tunnelring a is unloaded in radial direction and tunnelring b is subjected to radial loads by grouting-pressures

i.e. often near the dowel and notch system at the ring-joint, see Figure 15. The damage was analysed by Leendertse (1997). Due to these observations there was there was an increased interest in the measurements from the instrumented tunnel rings, measuring soil pressures and tunnel lining stresses, which were installed in the monitoring field, just some 50 meters further.

The measurements from the first instrumented ring indicated that, apart from a more or less regular stress distribution which could be related to the soil loading, peak stresses could be observed close to the key-element, i.e. the closing element of the ring. A back analysis of the lining stresses indicated that it is possible to relate the tangential bending moments to the soil loading. The evaluation of stresses in the lining showed that apart from the soil loading and the deformations related to that, there is an additional loading due to the staged installation of tunnel elements. To begin with an "unloaded" ring is mounted onto the end of an (due to grouting pressures) already partly loaded ring, in which process the ring is closed pushing the key segment into place. After that, the combined system is loaded by grouting when the Tunnel Boring Machine moves forward.

In this process eccentricities are introduced due to:

1. pushing in of the key-element
2. ovalisation of the tunnel ring, see Figure16

To begin with it appears to be difficult to build up a circular ring and in addition to that an eccentricity is introduced with respect to the supporting force into the already constructed tunnelring, see Figure 17. Both due to the fact that there is a load transfer between the adjacent rings and due to this induced eccentricity additional stresses are introduced into the tunnel ring.

In order to analyse this problem, learning from the observations on the first measuring ring, special care was taken that the stresses due to assembly of segments into a ring, before soil loading had taken place, were measured.

Analysis of these second measurements confirmed that the stresses in the tunnel lining can be subdivided into a part that is related to the soil loading, which is relatively easy to calculate, and a part that is related to the assembly of the tunnelring. The latter leads to 'eigen' stresses, (Bakker et al 1999b). The latter has a more or less stochastic character and is much more difficult to evaluate.

In order to analyse the measured stresses in the tunnel lining in more detail, Van der Horst, Blom and Jovanovic (1998) have performed 3D Finite Element analyses.

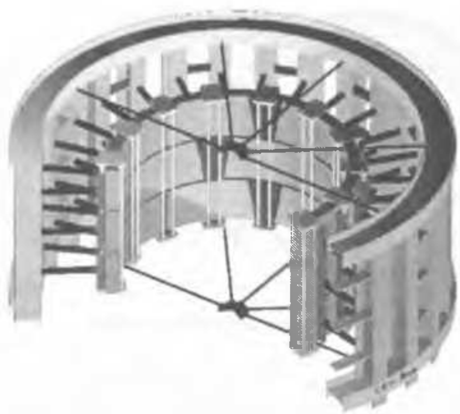


Figure 18. Experimental test setup for large scale testing of tunnel loading situations, (v.d Horst et al 1999)

When the importance of assembly stresses was recognised it was concluded that further experimental research was necessary and therefore a large test set-up was organised in the Stevin Laboratory of Delft University where these stresses could be monitored in a controlled situation and measured in more detail. For results of this see (Blom et al 2000).

6 OVERALL CONCLUSIONS AND RECOMMANDATIONS

The monitoring of the Second Heinenoord tunnel strongly enhanced research and development of tunnel engineering in the Netherlands.

An improved finite-element model for the evaluation of settlement troughs has been developed based on correct modelling of the grout pressures and the application of hardening soil material models for sand.

During excavation there is insufficient time for bentonite cake forming and therefore an increased pore-pressure around the bore front has to be taken into account.

An unforeseen benefit of the observation systems during the blowout was that afterwards, based on the back-analyses and evaluations it appeared to be possible to derive a relatively simple model for the evaluation of bore-front stability.

The use of over-cutters gives a decrease in the necessary thrust forces the TBM enforces on the tunnel lining

Improved models proved to be necessary to evaluate stresses and strains in the tunnel lining during assembly of the tunnel rings and subsequent loading during advance of the tunnel-boring machine.

The experience gained by this project has strengthened the confidence in the bore techniques for tunnelling in soft soil. A number of new bored railway and road tunnels have since then been planned in the Netherlands, 5 of which are presently under construction.

We can conclude that since 1993, the year that the decision was made to bore the first tunnels in the Netherlands as pilot projects, accompanied by extensive monitoring, the tunnel-boring technology in the Netherlands has matured.

ACKNOWLEDGEMENTS

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