Monitoring pilot projects using bored tunnelling: the Second Heinenoord Tunnel and the Botlek Rail Tunnel

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ABSTRACT: *Two pilot projects for bored tunnelling in soft soil have been undertaken in the Netherlands. The monitoring was commissioned under the authority of the Centre for Underground Construction (COB). A description of the research related to the Second Heinenoord Tunnel and the Botlek Rail Tunnel will be given. The Second Heinenoord Tunnel has completed its boring works, while the boring of the Botlek Rail Tunnel has started in march/April 1999. Experience from the first project is put into use for the development of the latter. Both monitoring schemes have three logical phases: predictions, measuring and evaluation. An overview of all instrumentation and a selection of the results of the Second Heinenoord tunnel are given.*

1. INTRODUCTION

The use of the subsurface is not new for the Netherlands. In this typical delta country, dominated by natural and artificial waterways, there is a tradition to apply tunnels instead of bridges to cross the major waterways in order to facilitate the water-traffic. Up to 1995 some 55 km of road and rail tunnels has been accomplished. Most of which are immersed tubes.

In 1993 however the Dutch Government decided to finance two experimental bored tunnelling projects. One is the second tunnel under the river Oude Maas near Heinenoord (Rotterdam), see Fig. 1. The other is the new railway tunnel also under the Oude Maas near Botlek (Rotterdam). An intense programme of research and monitoring has accompanied both projects. The Second Heinenoord tunnel has completed its boring activities. A large amount of

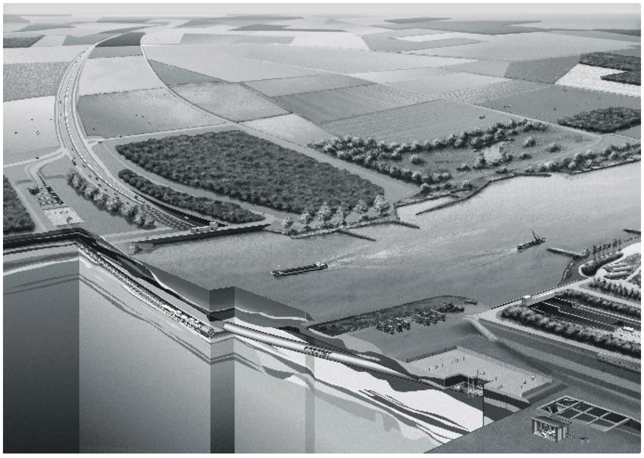

Figure 1 Aerial view of the Heinenoord tunnels

data about tunnelling and tunnel behaviour has been gathered. At the time of writing of this article a large effort is being carried out evaluating the results comparing predictions and measurements validating feasible design models. Therefor its still too early to draw final conclusions yet. The Botlek Rail tunnel has started construction in March 1999. Both projects offer a unique opportunity to enhance the experience and the specific knowledge on large-diameter shield tunnelling in very soft soil.

Since then the perspective has changed. These projects have been followed by at least five other major TBM-tunnelling projects (all of which are described elsewhere in this issue of *T&UST*)

1) The North-South metro line under the city of Amsterdam (under design)
2) The road crossing of the Western-Scheldt inlet (under construction).
3) The tunnel for the High speed rail line in the line between Amsterdam and Rotterdam (under design
4) and 5) the Sofia Tunnel under "The North" (under construction), and the tunnel under the Pannerdensch canal (under design) in the "Betuweroute" for Cargo rail

The monitoring of the pilot projects is being executed under the co-ordination of the Centre for Underground Construction, COB (see Admiraal 1999 in this issue of T&UST). The strategy of COB is to integrate research with projects under construction. If there is lack of knowledge for a future project the development is integrated with monitoring and experiment at a tunnel under construction. This strategy is beneficial for the financing of research and development. This approach leads to revolving funds for monitoring and research for underground construction.

2. MONITORING PILOT PROJECTS

A fruitful monitoring project needs a frame of reference. Without such a frame of reference the risk of divergence in the development would be too high. In order to provide such a frame of reference two leading theses have been stated:

- Measuring is knowing
- A measurement which has been preceded by a prediction has added value

It was decided therefor to make predictions related to the measurements, and to evaluate the measured data in the light of these. The predictions, based on empiricism, numerical analysis as well as applied physical models, i.e. laboratory experiments, were executed and reported before the boring phase of the tunnel, as "K100-04, Predictie rapport Tweede Heinenoord tunnel" (1997).

A concise summary of an inventory into the lack of knowledge relating to boring of tunnels in soft soil was, for the geotechnical part was published by Bakker et/al (1997). A prediction of the geotechnical deformations was published at the London meeting and Symposium of Technical Committee 28 at City University April '95, see Bakker et/al (1996)

3. THE SECOND HEINENOORD TUNNEL; MAIN MONITORING SCHEME AND CHARACTERISTIC RESULTS

3.1 The Second Heinenoord Tunnel

The existing Heinenoord Tunnel in motorway A 29 is one of the major daily congestion's for the traffic toward and from the Rotterdam "Europoort" area. To extend the motorway capacity from 2 x 2 lane to 2 x 3 lanes a new crossing had to be constructed for local traffic. This new crossing was originally designed in 1990 as an immersed tunnel. Due to budget problems, the project was postponed till 1993. The opportunity to use this tunnel as a pilot project for bored tunnelling has speeded up the construction of the tunnel again.

In 1996 the actual execution of the project began with the simultaneous construction of shafts and ramps. The bored tunnelling part was started in February 1997 from the North bank, and ended there too, in June 1998, after turning on the south bank. The cross-section consists of two tubes with an external diameter of 8.30 m. The total length of the tunnel is 1350 m with a bored part of 950 m for each tunnel.

There are no cross-passages. As the detailed study into the suitability of machine types for the soil, showed a slight preference for a Slurry-type machine over an Earth Pressure Balance (EPB) shield, the former was chosen.

3.1.1 Geological profile and soil properties at Location Heinenoord

In order to derive sufficient insight in the geological situation at the measuring area an extensive soil investigations program was carried out, including boreholes, vane-tests, CPTs, dilatometer- and pressiometertests. Parameters such as strength and weight where derived subjecting the samples to laboratory tests.

The tunnel cuts both through cohesive Holocene layers and sandy Pleistocene layers. At the measuring area on the North Bank, the tunnel mainly goes through Holocene deposits and sand layers, whereas on the South Bank, typical peat, and clay layers are to be found in the tunnel drive. On both measuring areas, just the bottom of the tunnel is bedded into the Pleistocene sand. On the North bank when arriving at the measuring area, which is at a distance of 75 m from the start shaft, the roof of the tunnel lies at a depth of 12 meter below soil surface, increasing up to 13 meter at the furthest side of the measuring area, 75 m further.

The tunnel was entirely driven under the ground water table, which lies just below the soil surface, which is approximately at 0.0 N.A.P. (N.A.P. being the Dutch reference level).

3.2 The monitoring scheme

Besides monitoring of the construction itself, the monitoring scheme consists of three technical parts.

1) Monitoring the processes related to tunnel boring machine
2) Monitoring of the geotechnical deformations
3) Monitoring of the structural behaviour of the tunnel lining.

3.2.1 The tunnel boring machine

Additional to the standard controls instrumentation of the TBM, which monitor topics, such as;

- position and speed of the cutting wheel etc.
- position and pressures of the-hydraulic jacks, grouting pressure, and volume,

Extra attention is given to the interaction between operating parameters, such as Slurry pressure, pressure distribution, stability and digging process parameters and finally deformation behaviour in front of the tunnel face. For that, extra instrumentation was added to the standard operating system of the TBM

With respect to the latter, it was not feasible to have inclinometers in the track of the TBM therefor direct measurements of the deformation in the track of the TBM could not be derived.

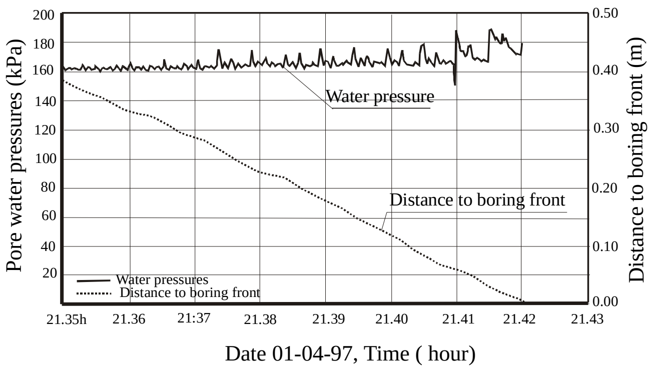

Figure 2   Pore water pressures near the boring front

In the interior of the Slurry chamber pressure gauges are placed on the bulkhead and on the rotating cutting wheel, in close proximity of the cutting edge. In addition to that, with the purpose to monitor the stability of the bore-front, it was decided to have water pressure gauges in the track, which were 'eaten' by the TBM.

As can be seen in Fig. 3, successful measurements up to the point that the connection between instrument and data logging system was broken were established. It can be recognised that the passing by of each spoke of the rotating wheel (with an excavating chisel) leads to a peak in the water pressure curve. There is a slight increase of the water-pressure, during the advancement of the boring machine. The fact that water-pressure peaks are visible is interpreted as that the thickness of the bentonite cake is relatively thin and that the cake is being removed by each chisel, and recovers in the intermediate period.

3.2.2 Bore-front instability

In the night of 28 august 1997, an instability of the Bore-front developed with loss of slurry material into the river Oude Maas. This instability halted the tunnel boring for a period of four weeks. The incident developed after the bentonite pressure in the working chamber showed a drop down. It was tried to maintain pressure with additional measures, including the extension of the support plates to prevent soil penetration in the chamber, which did not, succeeded. Sounding of the river bottom after the instability showed that a depression in the river bottom was created with a diameter of approx. 6 m. and a depth of about 2 m. To recover a stable situation for boring the depression hole in the riverbed was excavated and filled up with expanding clay pellets and sand. In addition to that, the working chamber was cleaned, and it was tried to raise the support pressure, which was not directly successful. After some manipulation, including excavation nearly without support pressure, a forward progress of several meters could be achieved without inadequate displacements, after which support pressure could be raised. After which the location of the incident could be left behind.

Although the machine had been operated with a support pressure higher than the vertical soil pressure, the main cause of the incident was blamed to heterogeneity in the subsoil. This heterogeneity might be invoked due to the fact that somewhere in the vicinity of the incident, during the construction of the first immersed Heinenoord tunnel a mooring pile had been present.

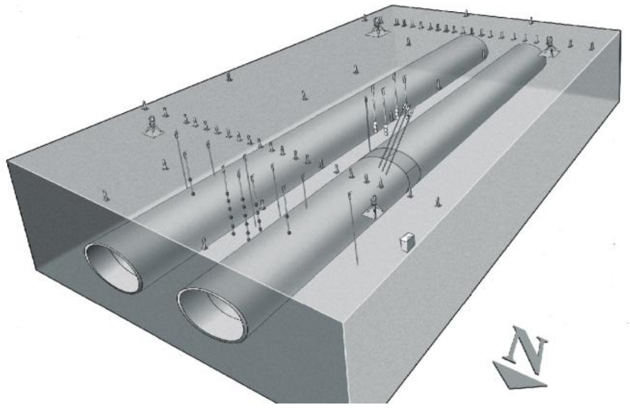

Figure 3 Measuring area at North Bank of river 'Oude Maas"

3.2.3 Monitoring the geotechnical deformations

Both on the North bank as well as on the South bank of the river Oude Maas a 50 x 75 square meters measuring area was installed for geotechnical monitoring, see Fig 1; A and D, and Fig. 2. The measuring area on the south bank is situated on soft Holocene ground layers. At the side of the measuring area at a distance of 75 m nearest to the start shaft, there is a soil cover of 12 meter on top of the roof of the tunnel. Increasing up to 13 meter at the other side of measuring area. In order to measure the influences of the tunnelling process on soil parameters and deformations, an integral measuring system was installed, including inclinometers, extenso-meters, surface level points, soil pressure cells and water-pressure gauges. For the specification of this system it had to be considered that: data collection with a high frequency should be possible; e.g. to measure the immediate effect of the bore-front of the TBM passing. On the other hand the system should be flexible enough to adjust to low frequencies too, such as needed for processes like consolidation effects of creep, long after the TBM has passed. Data has to be obtained which is fitted to the scientific goal. E.g. the accuracy for measuring

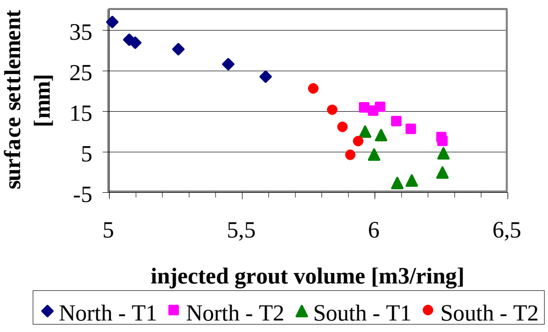

Figure 4 Maximum surface settlement as a function of injected grout volume (m^3/1.5m tunnel)

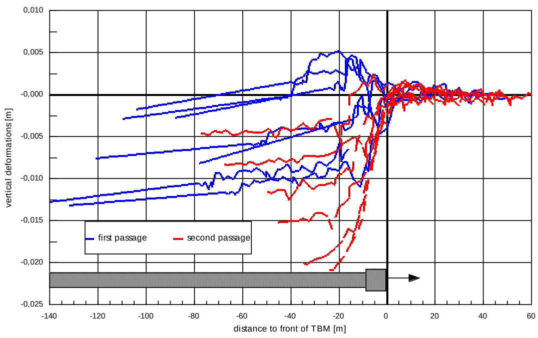

Figure 5 Green field settlements above tunnel axis at Southern Monitoring Area

the effects of changes of the rotation speed of the cutting wheel of the TBM can be totally different to the accuracy needed for the width of the settlement trough.

The Measuring activities were not allowed to disturb or decrease safety of the boring process, Therefor, arrangements had to be made with the contractor to follow a minimum distance alongside the outside of the tunnel lining and in front of the TBM face, of two meters. Only water-pressure gauges, smaller than 10 cm length and 5 cm diameter were allowed to be 'eaten' by the TBM. Larger sizes were supposed to damage pumps and other Slurry transport systems. These demands have proven to be sufficient, as no damage or disturbances whatsoever has been observed. It should be mentioned that the eaten water pressure gauges, of which results have been given in Fig. 2 have been seen in the muck separation system, but could not be saved; i.e. the system could not be halted within the available time span.

The maximum measured green field settlements above the tube axes are presented in Fig. 4. Generally, evaluating both measuring areas, up to a distance of approx. 5 to 10 m ahead of the TBM, almost no settlements occur. While the TBM passes, the settlements rapidly increase. The relatively small settlements at the front of the TBM indicate that soil displacements due to front effects remain small. Apparently, settlements that occur above the

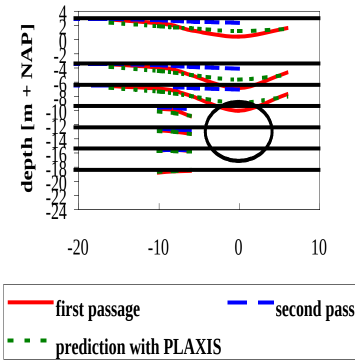

Figure 6 Vertical ground deformation at the Northern Monitoring Area

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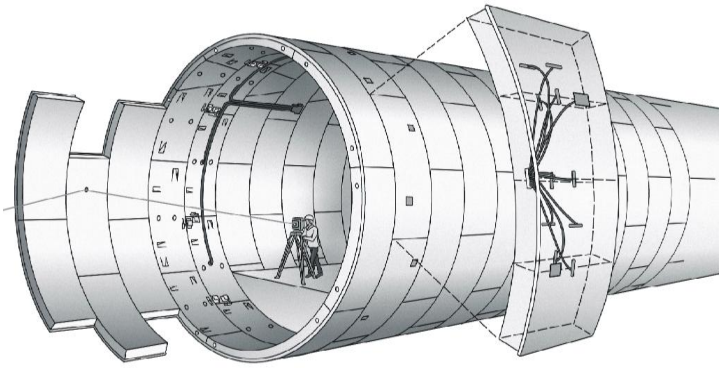

Figure 7 Measuring ring and instrumentation

face are caused by (spreading) soil deformations due to overcutting and tail effects. The applied front pressures are almost equal to the horizontal ground pressures.

At the Northern Monitoring Area it appears that, at a distance of 20 to 30 m behind the face, the settlements remain constant, which indicate that time dependent settlements are small. The subsoil at the Northern Monitoring Area mainly consists of sand. After the first passage, the maximum settlement at 32 m (4*D*) behind the face varies between 22 and 37 mm. After the second passage of the TBM, the maximum settlement varies between 7 and 17 mm.

During the first passage of the Southern Monitoring Area, see Fig 5, a heave of 5 mm has been measured behind the TBM. It appears that due to the backfill grouting process not an extension, but a compression of the soil above the tube axis occurs. At 32 m (4*D*) behind the face, the settlement varies between –3 mm (heave) and 10 mm after the first passage and 4 mm to 20 mm after the second passage. At a distance of more than 30 m, settlements do not stabilise, which indicate that time dependent settlements occur. These time dependent settlements are caused by the weight of an applied sand fill and consequently a compression of clay and peat layers.

Evaluating all the settlement measurements, it appears that a distinct relation arises between these settlements and the volume of grout injected in the tail void (see Fig. 4). At the Northern Monitoring Area the relation is almost linear. Due to consolidation effects and the use of overcutters, the relation between settlement at the Southern Monitoring Area and volume of injected grout is less distinct. This means that accurate settlement control asks for an accurate control of the grouting process.

The settlement troughs appeared to be steeper and smaller than predicted by finite element calculations. The volume of the vertical ground deformations at ground surface varies between -0.2% (heave) to +0.8% (settlement). In the calculations, a uniform contraction of the borehole due to tail void effects was assumed. This assumption results in a stress release and extension of the soil above and at the side of the tunnel and consequently a relative wide settlement trough. From the installed inclinometers and extenso-meters, it appears that an extension of the soil occurs above the tunnel. At the sides of the tunnel sometimes a small extension, but mainly a compression of the soil occurs. This compression is attributed to the

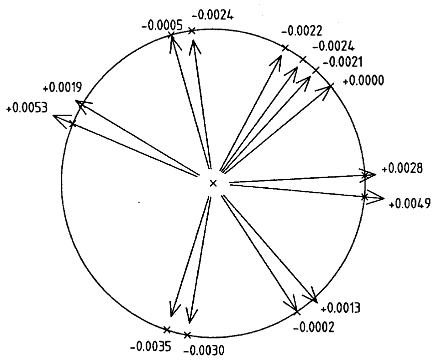

Figure 8 Deformations of the lining, Ring 568, measurement series 200, cycle, 148 and 149 just after installation and soil loading, (displacements in m)

grouting of the tail void. The vertical displacements, predicted and as measured values at the Northern Monitoring Area are presented in Fig. 6.

3.2.4 Monitoring of the structural behaviour of the tunnel (lining)

On two locations, one under the measuring area on the North Bank, and one on the south bank, a Tunnel-lining ring was equipped with strain gauge elements composed in different directions per element (ten strain gauges per element).

All seven elements in a ring were instrumented in order to derive a full insight in the stress distribution in the ring (see Fig. 7) as a function of time and distance behind the TBM. Complimentary to strain gauges, pressure cells were put on the outer surface of the elements; e.g. two pressure cells per element on 7 elements. In order to get an impression about differential deformations between elements, special devices where placed, bridging the joints between segments; details of these instruments are given by Leendertse (1997).

In addition to the common type position measurements with marked spots on the tunnel lining an additional measuring scheme was developed and implemented which enabled to monitor the deformations of the tunnel lining in the direct vicinity of the TBM. This is the area where the largest gradient in deformation with respect to time was expected.

This system uses an automated laser equipped Theodolyte system, and automatically measures inclination and distance of a number of pre-positioned points on the lining. This system which measured the deformations of the lining during the period that the unloaded ring left the tail of the TBM. An impression of radial displacements as measured is given in Fig. 8.

The biggest displacement seems to occur directly after the ring leaves the tail of the TBM and is loaded by the soil. The soil loading is being activated, in a staged process. To begin with, due to grouting a pressure driven action is developed, which reverses to soil structure interaction while the grouting material consolidates and hardens. This process develops approximately within a period of 24 hours.

Generally the roof of the tunnel tends to move downwards, between 0.003 and 0.006 m, whereas the sides of the tunnel deforms in the outward direction between 0.002 and 0.004 m. The bottom of the tunnel seems to be relatively stable, coming up only slightly, between 0.000 and 0.003 m.

3.2.5 Damage to the tunnel lining

At the start of the tunnel boring process, i.e. the first 100 m of the boring the damage to the lining was higher than expected. Characteristic to the damage was leakage of the joints in combination with differential deformations between segment rings. Especially near the K-segment, the closing segment, spalling of concrete edges has occurred. At some places differential deformations between rings seems to have exceeded the tolerances of the dowel and notch system in the ring joint. In situations that obviously the side of the notch system has broken at the outside of a segment, leakage of the joint developed. These observations have led to a more detailed numerical analysis of the construction stage of the tunnel lining, which supports the conclusion that a proper control of the installation phase is crucial for an adequate control of the integrity of the lining elements. A more detailed description of this analysis is given by Blom et/al (1998).

It is a topic of further research at the moment whether the adoption of triplex wood plates instead of Kaubit in the ring joint as material to avoid too high installation stresses did not counter-acted it's purpose.

3.2.6 Back analysis of measuring rings for the soil loading part

Based on the evaluation of the measurements on the North bank the importance of installation stresses was recognised. For the 2nd measuring ring on the South bank, it was decided to start data collection of stresses directly after assembly of the tunnel ring. With this approach it was possible to determine bending moments with and without installation stresses, (see Fig. 9).

To begin with, the reduced moments have been back analysed with the 2 Dimensional Finite Element code PLAXIS. As input for the soil data the information given in table I was

Table 1 Description of layers and soil parameters for the South Bank

symbol	soil type	top of layer [m] N.A.P.	(γ_{sat} ($_{dry}$)) [kN/m^3]	F_{undr} [kPa]	c' [kPa]	φ' [°]	ν [-]	E_{oed} [MPa]	K_0 [-]
OA/1/OB	Mixture of sand and clay	+ 3.50	17.2 (16.5)	-	3	27	0.34	5.2	0.58
3	Sand, local parts of clay	- 3.25	19.5	-	0	35	0.30	26	0.47
4	Peat	- 4.50	13	35	7	22.5	0.35	4.2	0.60
16	Clay, silty	- 7.25	16.3	35	5	26	0.34	4.1	0.60
18	sand, local parts of clay	- 10.50	20.5	-	0	36.5	0.30	40	0.45
32	sand, gravel	- 14.00	20.5	-	0	36.5	0.30	60	0.50
38A	clay, local parts of sand	- 21.50	20.0	140	7	31	0.32	16	0.55
38F	Sand	- 24.50	21.0		0	37.5	0.30	80	0.55

F_{undr} = undrained shear strength
C' = drained cohesion
φ' = drained friction angle
ν = Poisson's ratio
E_{oed} = Oedometer modulus of soil compression
K_0 = The coefficient of initial effective horizontal soil stresses

Table 2 parameters for the 2ndnd Heinenoord tunnel

R	= 3.975 m
d	= 0.35 m
E	= 30.000.000 kPa

used. For the structural data the information according to table II were used.

Where R is the tunnel radius, d is the wall thickness and E is the Young's modulus of the tunnel lining concrete.

The bending stiffness of the lining was reduced with 50 %, in order to account for joints. In Fig. 10 the reduced moments with the PLAXIS back-analysis are compared with the measurements. Fitting the Finite Element analysis to the data, both for the measuring ring on the North Bank as well as for the South Bank, it was observed that with respect to the tunnel lining forces, the best conformity was found for a contraction of approximately 0.5 %. For that volume loss, the low soil stresses measured where reproduced best, see Fig 11. For a calculation without volume loss the resemblance is less: a smaller amplitude of the bending moment is calculated, i.e. M_{max} = 79 kNm/m, instead of 93 kNm/m. Here the method of modelling the volume loss with a contraction was chosen, because of its practical ease. The research of the COB has learned however that for the evaluation of stresses and strains in the surrounding soil; i.e. the evaluation of the settlement trough, a better approach was found based on the modelling of grouting pressures, see Dijk et/al (1998)

The normal forces in the tunnel lining are reproduced much less than the bending moments. It seems as if the low soil stresses are not compatible with the normal forces being measured. It is thought that the soil stresses, as measured, have to be related to an earlier stage of stress development than the normal forces. It is as if the low stress level directly after grouting is "frozen in" by the hardening process of the cement in the Grout. It must be considered that maybe after that the Grout has cemented the effective soil stresses have increased again, which was not measured due to the fact that the grouting material has 'over-bridged' the pressure gauges. The strain gauges on the other hand have a higher degree of reliability. This assumption is supported by the fact that after removal of the axial support structures in the start shaft, and in the receiving shaft, the soil pressures measured have increased up to a level more in agreement with the measured tangential normal forces. (The latter did not change during this period).

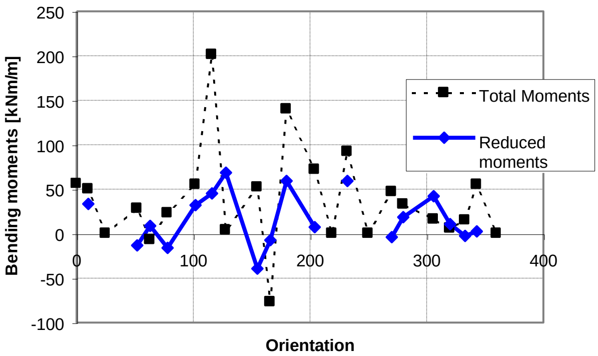

Figure 9 Bending moment measuring ring south bank; total and reduced for installation.

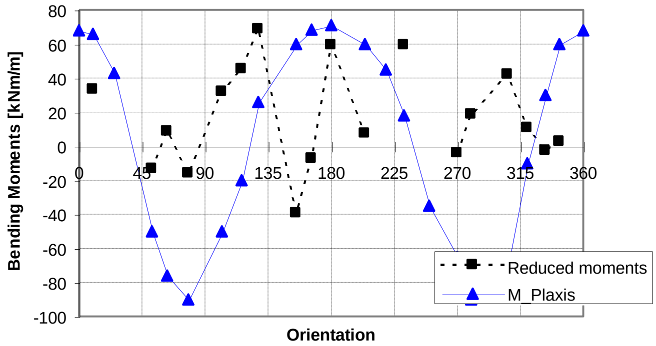

Figure 10 Comparison between back-analysis with PLAXIS and bending moments due to soil loading, for the measuring ring South

3.3 The test pile project at the location of the second Heinenoord tunnel

In addition to the monitoring of COB K100, an extensive monitoring of the boring's influence on piled foundations was performed; i.e. the test pile project of bureau "North-South"-line Amsterdam. This project was initiated with the perspective of the yet to be build metro-line through Amsterdam, where most of the foundations are piles; i.e. timber pile foundations on the first sand layer for the brickwork houses with the weaker foundations. A more detailed description of this part of the monitoring was given by Leendertse et/al (1997). More detail related to the project "North-South"-line Amsterdam can be found in the contribution of Kaalberg et/al (1999); Design of the north/south metroline under Amsterdam, in this special issue of Tunnelling and Underground Space Technology.

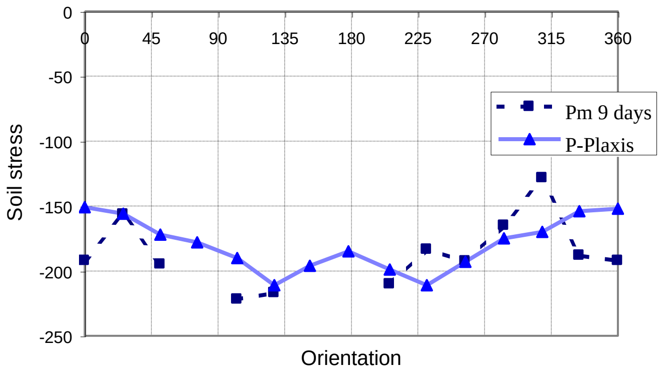

Figure 11 Radial soil stresses on the tunnel lining

4. THE BOTLEK RAILWAY TUNNEL; MAIN MONITORING SCHEME

4.1 The Botlek tunnel

The "Havenspoorlijn" is the existing railway connection between the harbour and the industrial areas of Rotterdam and forms the beginning of the “Betuweroute”, the connection with the European hinterland. Due to the expected growth in cargo transit, the capacity of this line bas to be enlarged. One of the major bottlenecks is the existing (single track) Botlek bridge over the river Oude Maas.

More detail about the project Botlek can be found in the contribution by Jonker & Maidl in their contribution to this special issue of Tunnelling and Underground Space Technology. Here we will focus our attention on the monitoring part of the project.

4.2 The monitoring scheme

The monitoring at the Botlek railway tunnel, during construction phase will focus on:

- TBM's related aspects ;
- Geotechnical aspects ;
- Structural behaviour of the lining in the construction stage ;
- Dynamic behaviour of the tunnel.

Just as in the case of the Second Heinenoord tunnel, it was decided to perform predictions with known models before measurements were carried out, and to compare the measured data with these predictions.

4.2.1 TBMs related aspects

During the boring process, much data will be collected to control the stability of the boring front, the mixing of the different soil types and foam, the transport of the soil/foam mix, the forces and position of (parts of) the TBM and the grouting process. Examples are given in table 3.

In addition to the already mentioned "permanent" measured data during the boring of both tubes, a number of experiments will take place at three representative locations such as:

- taking soil samples from the mixing space by means of a specially developed instrument ;
- followed by compression test, size distribution and dry content ;
- Performing vane tests within the mixing space.

Table 3. Examples of data that will be collected to control the stability of the boring front, the mixing of different soil types and foam, the transport of the soil foam mix, the forces, and the positions of (parts of), the TBM and grouting process

• Pressure in the mixing space	• Pressures and displacements of the jacks
• Rotation velocity, and forces on the screw conveyor	• Pressures and capacity of the grouti conduits
• Capacity and pressure of the foam generator and lances	• Water pressures in the mixing space
• Forces, velocity and position of the cutting wheel	• Total pressure measurements inside the scre conveyor
• The position and inclinations of the TBM	

Attention will be paid to the wastage of the cutting teeth and the gathering of the process data to assess the needed boring time, build-in time for a ring, waiting time and so on.

4.2.2 Geotechnical aspects

In this part, the influence of the boring process on the changes in stresses and deformations in the soil will be monitored. At six representative cross sections stresses and deformations will be measured at the soils surface and deeper in different layers, before, during and after the TBM's passing. An integral boring control system using online measured data due to boring such as soil's and surrounding constructions' settlements, will be fitted into the TBM's steering controls. As a part of such a system, several types of surface displacement measurement systems will be used and evaluated.

Attention will also be given to the parameters of the outcoming soil, within the scope of soil's reuse. For an EPB type TBM the out-coming soil is often mixed with foam, as its cohesion is poor. The transport of the soil-water-foam mix is done hydraulically.

The volumes of out-coming soil are measured in the depot and laboratory tests will be performed; i.e. void ratio, pH, size distribution, content of fine particles, organic content, consistency, triaxial tests and oedometers. Afterwards the possibilities for reuse will be exposed. From the soil in the depot a consistency-time relation will be developed.

4.2.3 Structural behaviour of the tunnel Inning in the construction stage

The development of peak stress in the lining during the construction's stage was an important result of the monitoring of the Second Heinenoord tunnel, see § 3.2.5. This fact was confirmed by numerical analyses using recently developed models, see Blom (1998) and v.d

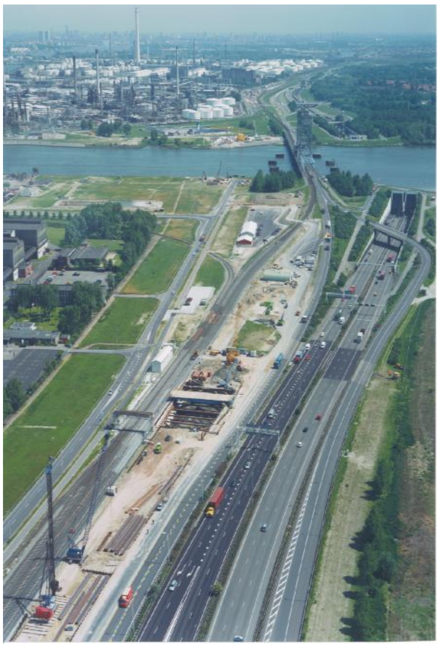

Figure 12 Botlek rail tunnel, under construction

Horst (1999). For the Botlek Rail tunnel it is expected that the axial forces from the TBM for an EPB shield are higher than for a slurry shield. Therefore it was decided to monitor the local stresses in the lining immediately after that the segments form a new ring. After that the TBM's jacks are acting their loads on the lining, local stresses are monitored for a period of a week. Other than the closing segment, the stress and strain measurements take place in only three segments, and subsequently for **five** rings after each other.

4.2.4 Dynamical behaviour of the tunnel

In the past few years in the Netherlands models were developed to predict the level of vibrations, as a function of vibration source (be rail traffic), a transmitter (be soil) and a receiver (be a person or foundation). To validate these models, experiments will be performed with a cargo train and a controlled harmonic dynamic loading in the tunnel as vibration source. Dynamic soil parameters will be retrieved from the results of a so-called dynamic cone penetration test. Three instrumented driven piles seeded at different distances from the tunnel's lining simulate the receiving foundation. In the lining vibration gauges will be installed in radial, tangential and axial directions; besides that some vibration gauges are to be placed in the soil.

In this manner, the models for predicting the vibrations' level will be validated for a situation featuring a bored tunnel.

5. CONCLUDING REMARKS

The monitoring of tunnelling projects in the Netherlands offers a unique chance to enhance the knowledge of tunnelling under soft soil conditions. As the Dutch geology with its high level of ground water, and its thick layers of Holocene deposits are challenging for underground works.

The monitoring at Heinenoord has shown that it is possible to bore in soft soil. The bore-front instability and the damage patterns at the lining have demonstrated that low stiffness and strength of the soil leads to a smaller bandwidth to endure pressures, and a lower capacity to support the lining. The measurements of stresses, strains and local displacement of the tunnel lining has revealed that stresses related to the assembly of the tunnel lining might be dominant in comparison to the stresses related to the soil's loading. These early stresses have to be added onto the latter ones due to soil loading before the results are compared to the criteria, which have to be taken into account, and then evaluated.

The promising results of the Second Heinenoord tunnel measuring area enhance the expectation of the monitoring at the Botlek tunnel.

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