Recent Dutch experiences in developing structural monitoring systems for shield driven tunnels

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This paper discusses two major structural monitoring programs that have been recently conducted in the Netherlands in the Second Heinenoord tunnel and Botlek Railway tunnel respectively. The Second Heinenoord tunnel was the first large diameter shield driven tunnel in The Netherlands. It was constructed in the period March 1997 to July 1998. Because of to its novel construction technique for the Dutch building community, a large monitoring program accompanied the construction of the lining. This paper focusses on the structural monitoring program at the Second Heinenoord tunnel. The monitoring concept, the execution of the monitoring and the lessons learned are discussed. Subsequently, the enhanced monitoring program for the Botlek Railway tunnel, that was conducted in 2000, is presented. Finally, some future developments in monitoring the structural behaviour of shield driven tunnels are discussed.

Key words: shield driven tunnels, structural behaviour, monitoring

1 Concept for monitoring structural behaviour

With respect to the general monitoring concept of the Second Heinenoord tunnel, three coherent aspects were identified:

- tunnelling (tunnel boring machine control parameters, grout behaviour, properties of excavated soil),
- geotechnics (soil behaviour and surface settlements),
- structural behaviour of the lining.

Two monitoring sites were chosen, one on the North bank and one on the South bank of the River Oude Maas, as shown in Figure 1. Figure 2 shows an overview of the North bank monitoring site. The position of the equipment at both sites was chosen such that the aspects mentioned previously and their could be studied. At each site, a ring in the western tube was designated for structural monitoring, approximately in the middle of the monitoring site as indicated in Figure 2. A thorough discussion on the full monitoring program and the overall research program related to the Second Heinenoord tunnel is given in [1].

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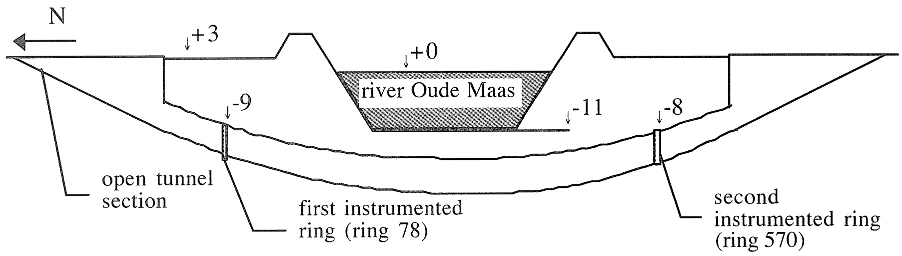


Figure 1. Position of the monitoring rings. The numbers refer to the height above sea level, in meters.

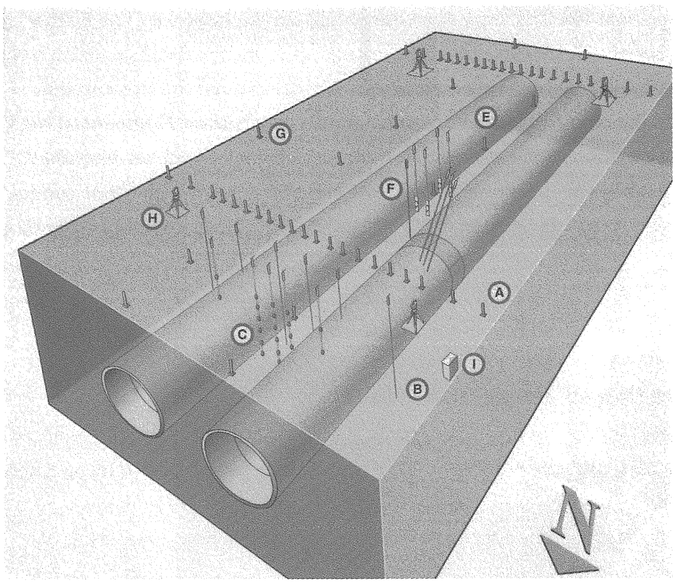


Figure 2. North bank monitoring site. The letters refer to specific measurement devices: A, G, H, I belong to the surface settlement monitoring system, B are soil stress monitoring stations, C are extensometers and inclinometers. The structural monitoring ring is shown near F.

The research program that accompanied the construction of the Second Heinenoord tunnel was based on a threefold approach: prediction, monitoring and evaluation. For the structural behaviour, a large set of predictions was made by several parties [2]. The predictions were made with a variety of models ranging from simple 1 D analytical models to full 3D FEM models. Although the models differed in complexity and assumptions on the soil-structure interaction (spring-like behaviour versus radial loading patterns), they agreed on one basic assumption. All models assumed a perfect assembly of the lining, i.e., no significant stresses were expected due to misalignments and other imperfections that may occur during the assembly of a segmented ring in a TBM.

This assumption was commonly accepted at that time in the engineering practice. Therefore, this seemed to be a reasonable starting point for the monitoring concept for the structural behaviour of the Second Heinenoord tunnel. The predictions had given maximum ring forces and moments in the lining due to the soil conditions at the construction site. Under the assumption that the assembly of a ring is perfect, a homogeneous stress distribution may be expected in the cross-section of a lining segment, expect for the variations due to the load introduction path of the TBM jack forces. In that case, strain measurements are a proven method to derive the stresses in the lining. It was decided to use embedded strain gauges in the monitoring rings to protect the measurement devices from the rough conditions in the TBM and tunnel under construction.

A basic requirement for the monitoring system was, since the monitoring period was set at 5 years from the moment of assembly of a monitoring ring. Therefore, both the measurement devices and the data acquisition system needed to be robust and stable. The vibrating wire principle was adopted to measure the strain, since a vibration wire shows no decay with time, contrary to the resistance wire strain gauge. Robustness was ensured by embedding the devices in the concrete. The vibrating wire devices are easily attached to the reinforcement before casting as shown in Figure 5. The data acquisition system consists of three parts to ensure durability. The first part is the connection of the vibration wires to the data acquisition system. The first part is illustrated in the right part of Figure 3. All vibration wires are connected to a central junction box at the interior surface of a segment. The second part is also shown in Figure 3: all boxes connect to a central data acquisition system mounted on the tunnel lining. The third part was a back-up system outside the tunnel, connected to the central data acquisition system in the tunnel by a data transmission cable.

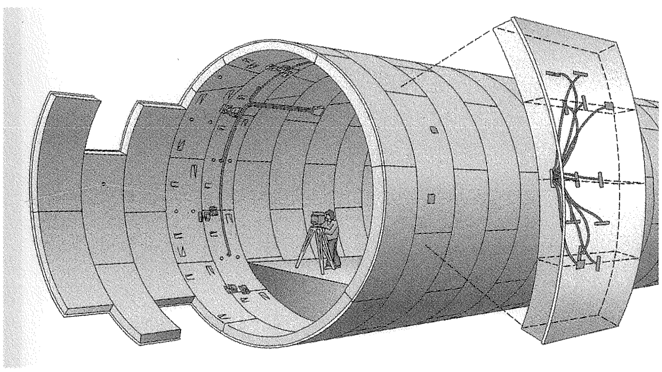


Figure 3. Concept of the monitoring ring.

A further basic condition for the monitoring concept was the requirement to have a coherent general monitoring strategy. For that purpose, the geotechnical monitoring part included soil stress monitoring stations near the monitoring rings. Pressure cells were mounted at the exterior of the monitoring rings to relate structural and geotechnical monitoring. Moreover, the pressure cells may help to verify the assumptions in the predictions with respect to radial loading on the lining.

The final part of the monitoring concept was the measurement of ring displacements to verify the assumptions of the ring behaviour. Two types of measurements were conducted. The first type was a convergence measurement to identify changes in the ring diameter. The second type concerned the joint displacement measurements on six positions.

Technical details of the devices in the monitoring rings are given in the following section.

2 Monitoring set-up

The position of the strain gauges in a segment had to be carefully chosen because of the load introduction effects of the TBM jack forces. One of the 2D finite element models was used to predict the stress distribution due the TBM jack forces. The position of the strain gauges was chosen in a zone where the variation in strain was small over the length (153 mm) of the vibration wire. The positions as shown in Figure 4 were found suitable and feasible with respect to the reinforcement layout. Each position contained a pair of vibrating wire devices, to be able to determine the normal forces (average strain) and the bending moments (curvature) in a cross-section, as shown in cross-section A-A of Figure 4.
A segment contained 10 vibrating wire devices in 5 measurement positions. Vibration wires from Geokon, type VCE 4200 were used. They have a range of -1500 to +1500 μm/m with a resolution of 1 μm/m. Two positions were used to measure axial strains, three positions were used to measure tangential strains as can be appreciated from Figure 4.

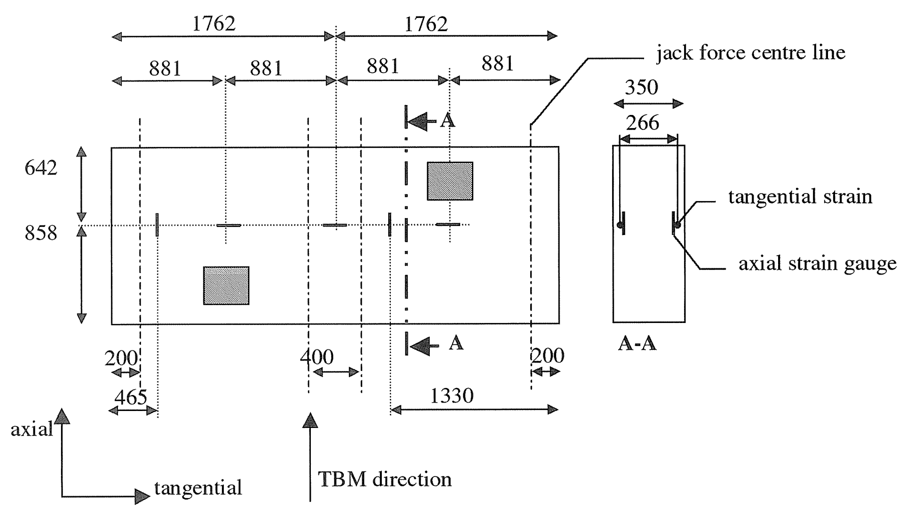


Figure 4. Left: plan view of a Second Heinenoord tunnel lining segment with positions of strain gauge pairs. One segment contains 10 strain gauges. Right: cross-section A-A, including the embedded strain gauges. Dimensions are in mm.

Figure 4 also shows the positions of the pressure cells, indicated by the squares. The pressure cells are from Geokon, type 4850-2 with an exterior surface of 150x250 mm^2. The range of the cells is 0 – 3500 kPa, with a resolution of 1 kPa. The pressure cell is a mercury filled reservoir. The pressure is measured by a membrane. The membrane strain is detected by a vibrating wire, so that the pressure cell also has a stable reading.
The segments of the two monitoring rings were instrumented according to the scheme shown in Figure 4 before casting. During October 1996, the segments of the two monitoring rings were instrumented at Schokbeton BV, Zwijndrecht. After completing the instrumentation of the reinforcement webs, the segments were cast. Only 5 of 200 devices did not survive the casting and subsequent actions, like compacting and release from the cast. The northern ring contained 70 vibrating wires, 14 pressure cells and 18 displacement devices, whereas the southern ring contained 76 vibrating wires, 14 pressure cells and 18 displacement devices. The data acquisition system, therefore, consisted of 198 channels.

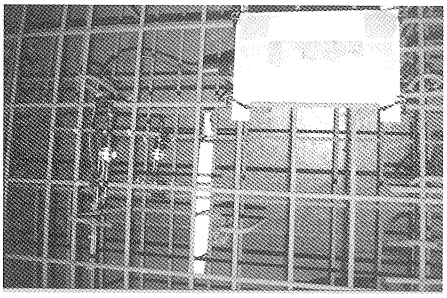


Figure 5. Top view of strain gages and pressure cell mounted on reinforcement, just before casting the concrete of a segment.

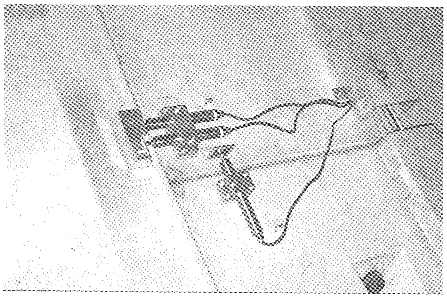
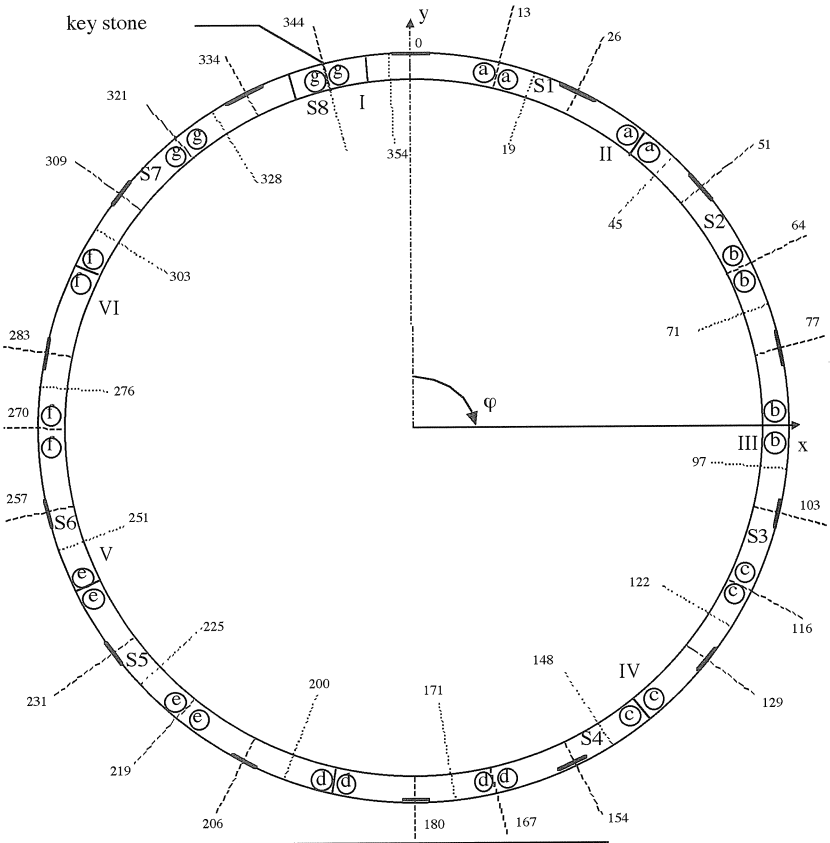


Figure 6. Joint displacement transducers.

In December 1996, the segments were taken to the TNO laboratories for calibration as discussed in the following section. After calibration, the segments were ready for assembly. The first ring to be assembled was the northern monitoring ring. It was placed as ring nr. 78 in the western tube. During a five hour operation on April 3 1997, the ring was assembled, the data acquisition system mounted on the lining and the data transmission tested. Mounting the joint displacement devices on the monitoring ring was also included in the operation. The displacement devices were LVDT's from Monitron, type MTN/IEDSGAS/227, with a resolution of 25 μm and a range of ± 5 mm. Figure 6 shows one measurement position for the joint displacements. One device measured the axial displacement between the monitoring ring and the previous ring (North: between rings 77 and 78, South: between rings 569 and 570). A second device measured the tangential displacements between two segments. Finally, the third device measured the radial displacements between two segments.

The northern ring was located at 117 m from the start shaft. The southern monitoring ring was assembled as ring 570 of the western tube on November 3, 1997, again during a five hour operation. The southern ring was located at 95 m from the end shaft.

A cross-section of a monitoring ring is given in Figure 7. The cross section indicates the positions of the TBM jacks and the positions of the measuring devices. Each device had a unique identification number using the polar coordinate of the cross section where the device was located. The keystone was only instrumented in the southern ring following a recommendation on the basis of the preliminary analysis of the northern monitoring ring results.



Legend:
S1..8 = segments 1 to 8 (8 = key stone)
a..g = jack force groups a to g
I..VI = joint displacement transducers I to VI
= measurement cross-sections for tangential strains
------- = measurement cross-sections for axial strains
.... = segment perimeter
____ = pressure cells

Figure 7. Cross-section of a monitoring ring. Devices are identified by the polar coordinate φ. Note that there are seven jack force groups. The jacks in one group have equal jack pressures.

3 Calibration

Prior to the assembly of the monitoring rings in the Second Heinenoord tunnel, the segments were calibrated by means of laboratory tests. The tests were conducted at TNO in December 1996. The first test series aimed at simulating the load introduction path in a segment for the TBM jack forces. Hydraulic jack forces were applied on the segment faces according to the as-designed centre lines of TBM jacks, i.e., on $^1/_4$, $^1/_2$ and $^3/_4$ of the segment length. Note that the as-built TBM jack force centre lines, as indicated in Figure 4, are different: the jacks do not act on $^1/_4$ and $^3/_4$ of the segment length, but on 200 mm from the segment edge. In the laboratory, the hydraulic jack force was gradually increased and simultaneously the strains were measured. Subsequently, a force-strain relationship was found that linked the axial loads to the readings in the vibrating wire transducers.
In the second series of tests, a ring force and ring moment was applied, as shown in Figure 8. The basic assumption at the start of the monitoring project was that the stress distribution in a ring cross-section is homogeneous. Therefore, the segment's structural behaviour was described by the following calibration formulae:

$$M_{tang} = C_M \cdot \tfrac{1}{2}\,(\varepsilon_i - \varepsilon_{i+1}) \qquad (1)$$

$$N_{tang} = C_F \cdot \tfrac{1}{2}\,(\varepsilon_i - \varepsilon_{i+1}) \qquad (2)$$

where
M is the bending moment in the cross-section [kNm];
N is the normal force in the cross-section [kN];
ε_i is the strain in the top device in cross-section A-A of Figure 4 [μm/m];
ε_{i+1} is the strain in the bottom device in cross-section A-A of Figure 4 [μm/m];
C_F is the calibration factor for a normal force [kN/μm/m];
C_M is the calibration factor for a bending moment [kNm/μm/m]

The calibration factors that were derived from the test results are summarised in Table 1. Note the difference in calibration factors for tangential moments between northern and southern monitoring rings. In the northern ring, the vibration wires were mounted under the reinforcement, in the southern ring the wires were mounted on top of the reinforcement. Consequently, the measured strain in the southern ring was larger than in the northern ring for an equal bending moment.

Table 1 Calibration factors for segment cross-sections that include strain gauges. A tangential cross-section is shown in Figure 4 (A-A), an axial cross-section is defined as 1 m wide by the height of the segment (0.35 m).

Ring	tangential		Axial	
	C_F [kN/μm/m]	C_M [kNm/μm/m]	C_F [kN/μm/m]	C_M [kNm/μm/m]
northern	22.4	1.72	11.4	1.38
southern	22.4	1.95	11.4	1.38

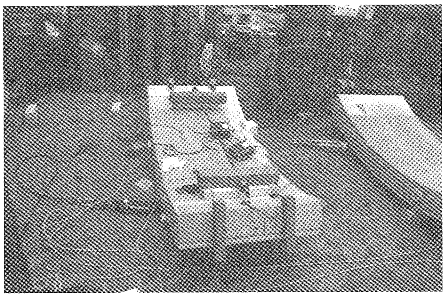


Figure 8. Calibration test of a segment. The hydraulic jack introduces a tangential force and a moment.

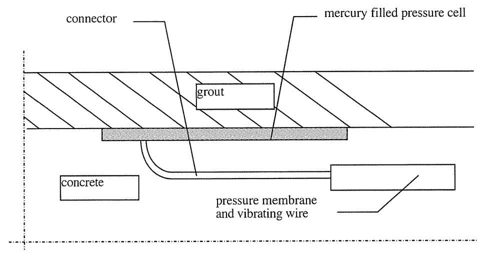


Figure 9. Detail of pressure cell.

The pressure cells were already calibrated by the manufacturer. The calibration sheet data were not verified, but used directly.

Monitoring program

The monitoring program consisted of several periods that can be summarised as follows:

- During the first two weeks from assembly of a monitoring ring, any minute a sample of all devices was recorded.
- After two weeks and running until a half year after assembly, the sample period was 7 minutes.
- After the first half year of recordings, the sample period was set to 30 minutes.
- The sample period was set to 1 minute again when the TBM passed a monitoring ring while constructing the eastern tube.

The continuous monitoring program was finished in April 1999, as tunnel completion pro-cedures required the data transmission cable to be removed. From that moment on, the data acquisition system has been logged on a periodic basis, until 2001. During the continuous monitoring period, there have been a few data transmission failures, mainly due to damage to cables. Although the cables were protected, heavy handling in the tunnel, like the construction of the inlay, damaged the cablework several times. Most accidents with cables happened more than a month after assembly of the monitoring rings, probably due to decreasing attention of the contractor.

Monitoring procedure

Each sample of the monitoring program was stored by the data acquisition system in the raw data file. The raw data has been converted to engineering units (like pressures, normal forces and bending moments) by means of Visual Basic routines in Microsoft Excel, using the calibration results. During the monitoring period, these results were stored and presented on the back-up PC outside the tunnel.

Evaluation the monitoring approach

The monitoring results have extensively been studied and reported, for example in [3]. The analyses of the data led to some surprising results that are a good starting point for the evaluation of the

monitoring approach. The most significant conclusion was that the assembly of the lining might lead to local stresses and strains of the same magnitude as the stresses and strains due to external loading [4]. Therefore, all predictions were proven to underestimate the maximum stress levels that were observed during the monitoring period.
An improved FEM model was developed to match the observed behaviour. The calculation results from this model were compared to the measured strains to calibrate the model. Subsequently, a stress distribution in the lining could be visualised. This showed that the stress distribution in a segmented lining was far more irregular than previously believed. As a consequence, it was concluded that the calibration procedure as summarised in equations (1) and (2) is obsolete for a shield-driven tunnel lining, since these equations assume a homogeneous stress distribution. The interpretation of the measurements should be performed on the strain level.

In retrospective, the positions of the strain gauges were not ideally chosen. Some of the gauges happened to be positioned in a zone with low stresses. As it is hard to predict the exact position of strain extremes, for future monitoring projects it is recommended to use a more densely instrumented ring. A minimum seems to be two pairs of strain gauges in any segmental cross-section, as illustrated in cross-section A-A of Figure 13.
Although the joint displacement transducers worked properly, it was found that the use of six positions was insufficient to fully capture the joint behaviour. More specific, the typical large joint displacements that were observed, see for example Figure 10, could not be explained from the measurements. It seems to be essential for a full determination of joint behaviour that any segment corner is instrumented with displacement transducers.
An important issue that hampered the evaluation of the monitoring results was the strong temperature dependency of the pressure cells. The temperature dependency can be explained with the following hypothesis. The mercury in the pressure cell is enclosed by concrete and grout, as illustrated in Figure 9. Due to the large difference in temperature expansion coefficient of mercury and concrete/grout (respectively $80° \cdot 10^{-5}$ K^{-1} and $10° \cdot 10\text{-}5$ K^{-1}), the enclosure will lead to increased pressure in the mercury when temperature rises. Consequently, the pressure cell will show a fake load effect. The enclosure hypothesis was verified in a laboratory test in March 1999. First, a pressure cell was cast in concrete, with a free exterior surface. The specimen was exposed to typical temperatures that were measured in the Second Heinenoord tunnel, being 5 to 25°C. No temperature dependency was found. Subsequently, a grout layer was poured over the top of the specimen, covering the pressure cell outer surface. The temperature while pouring was 20°C. After hardening of the grout layer, the specimen was again put in the climate chamber. Now, a temperature dependency was found from 20 to 25°C. This test result confirms the hypothesis that the temperature dependency in the pressure cell readings is due to the enclosure of mercury in a concrete and grout sandwich.
It was possible to correct for the temperature dependency by regression techniques using the complete monitoring set (more than 2 years of data, which includes all seasonal effects). It may be concluded that results from long term pressure measurements in shield-driven tunnels should be considered with care.
The vibrating wire strain gauges showed a mild temperature dependency of about 1-2 μm/m/K. There may be two reasons for this effect. First, all measured strains are corrected for the difference in

temperature expansion coefficient of concrete (assumed $10 \cdot 10^{-5}\ K^{-1}$) and the vibrating wire steel ($12 \cdot 10^{-5}\ K^{-1}$, according to specifications). It may be possible that the expansion coefficients are slightly different. This has not been investigated. Secondly, some of the segments may exhibit a temperature related variation in strains due to restrained displacements. In a segmented lining, some segments may not be able to deform freely. This restrained displacement, for example due to misalignments, will result in changes in strain as the temperature changes. This hypothesis was not verified. As the temperature effects are small and the major loading effects occurred within 48 hours after assembly, the monitoring data from the vibrating wire devices continue to be very valuable.
During, and the days after assembly of the northern ring, accumulation of water just behind the tail of the TBM caused problems for the data acquisition system. Although the connectors were all sealed for watertightness, they were not prepared for water heights up to 1 m, as shown in Figure 11. The water accumulation was significant as the drive direction was downwards. In this southern monitoring ring, the problem did not occur, since the driving direction was upwards. Because of the water accumulation, some data from segments S4 and S5 (numbers refer to Figure 3) was lost during the first days after assembly. After one week, the progress of the TBM was sufficient and all connectors were checked and cleaned. From then, the data was retrieved correctly.

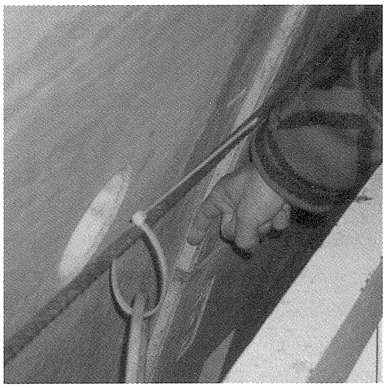


Figure 10. Large ring-ring joint displacements that were observed in the Second Heinenoord tunnel.

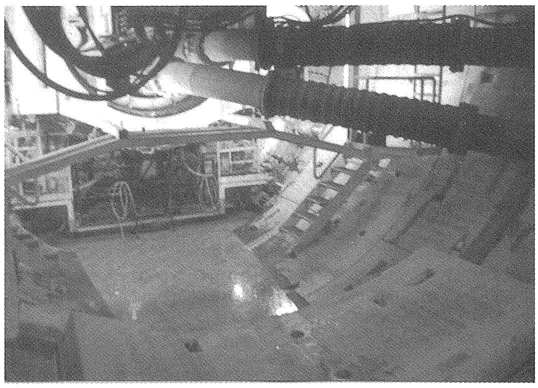


Figure 11. Excessive water in TBM near the northern monitoring ring.

4 Evolution of structural tunnel monitoring since the Second Heinenoord tunnel

Important conclusions with respect to structural monitoring of shield driven tunnels that could be drawn from the Second Heinenoord tunnel monitoring rings are :

- As the stress distribution in a segmented lining is by definition 3 D, structural monitoring programs should include at least 2 pairs of strain gauges in a segment cross-section.
- Stress paths may cross ring joints, especially near the key stone . To prove this, two or, ideally, three adjoining rings should be instrumented.

- For a full understanding of the effect of joint behaviour on the stresses and strains in a segmented lining, all corners of a monitoring ring should be instrumented with joint displacement transducers.
- As the assembly is so important for the final strain distribution in the lining, it is recommended for future projects to start monitoring of a segment before assembly.

All research issues that were risen from the above conclusions were investigated in the Large Tunnel Test Facility, a joint facility of TNO and Delft University. This unique facility can accommodate up to 3 rings with a maximum diameter of 11 m. On behalf of the Management group Betuweroute and the Projectorganisation HSL South, 3 rings of the Botlek railway tunnel were tested. The facility and the test results have been discussed in [5]. More important, an extensively instrumented monitoring ring was included in the Botlek Railway tunnel. This monitoring ring was built according to the first and last conclusion from the Second Heinenoord tunnel monitoring project. Special attention was paid to the data acquisition system, to enable data collection before, during and after assembly of the segments in the Botlek Railway tunnel. The next section discusses the improvements and modifications of the structural monitoring in the Botlek Railway tunnel.

5 Structural Monitoring in the Botlek Railway tunnel

The Botlek Railway tunnel is part of the cargo line "Betuweroute" and enables the crossing of the river Oude Maas. This line will run from Rotterdam to Germany. The Botlek railway tunnel is a shield driven tunnel and consists of 2 tubes of 1800 m length. The external tunnel diameter is 9.45 m and the lining thickness is 0.40 m. Figure 1 shows a longitudinal cross-section of the Botlek railway tunnel.

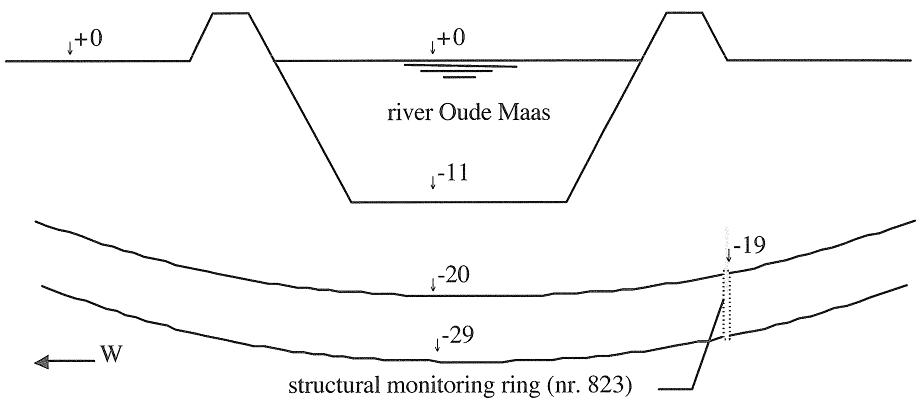


Figure 12. Longitudinal cross-section of the Botlek railway tunnel.

The structural behaviour of the lining was monitored by an instrumented ring in the northern tube, as a part of a large research program [1]. The monitoring program focused on the early life of a ring, i.e., the assembly of the ring in the TBM and the subsequent days when a ring was loaded by grout,

soil and water pressures. Ring 823 of the northern tube was chosen to be instrumented for the determination of the static behaviour of the lining. For that purpose, 232 vibrating wire strain gauges were embedded in the segments, including the keystone. A special feature of the monitoring program was that monitoring started before a segment was placed in the shield. This approach was chosen to quantify the effects of ring assembly on the final strain distribution in a lining, as the monitoring project in the Second Heinenoord tunnel had shown that considerable assembly stresses occur [1].
The ring concept in the Botlek Railway tunnel is similar to the concept in the Second Heinenoord tunnel (see Figure 7): 7 segments and a keystone. Again, vibrating wires from Geokon, type VC 4200, were used to measure the strains. Reflecting the first conclusion from the Second Heinenoord tunnel monitoring project, the wires were distributed as indicated in the left side of Figure 13. The gauges that measure axial strains were aligned with the jack force centre lines, as the largest strains may be expected there. The gauges that measure tangential strains (in ring direction) were located at spots where minimum or maximum values are expected. All gauges were located at approximately one fourth of the segment width, avoiding the segment boundaries. Again, the vibrating wires were organised in pairs as indicated in the right side of Figure 13 to determine average strains and curvature. No calibration was conducted as the evaluation of the results and comparison with the predictions was done on the strain level.
The instrumentation of the segments was similar to the procedure of the monitoring rings in the Second Heinenoord tunnel. After casting, a functional test revealed that only 4 of the 232 gauges did not function, despite the rough treatment in the casting plant.

The monitoring ring was assembled as ring 823 of the northern tube on 17 October 2000. Prior to the assembly, a data acquisition system was mounted on ring 822. In addition, cables were already

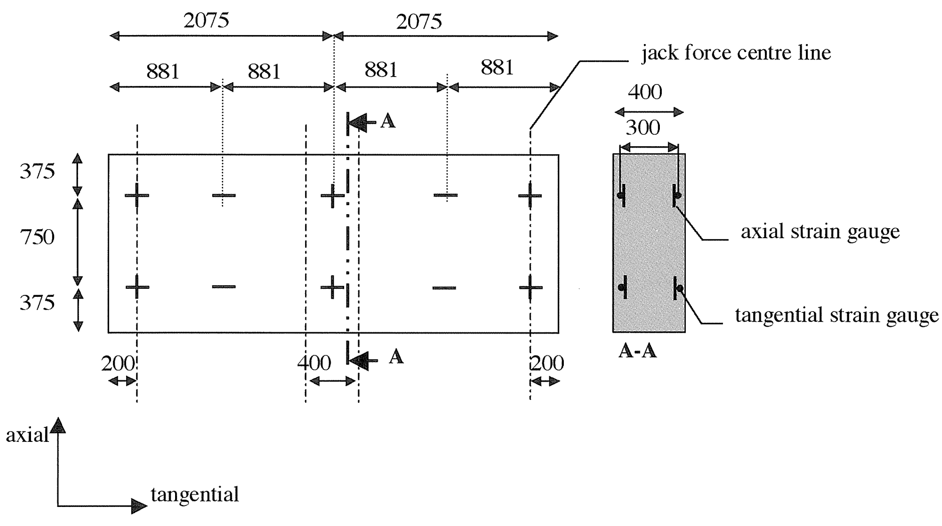


Figure 13. Left: plan view of a lining segment with positions of pairs of strain gauges. One segment contains 32 strain gauges. Right: cross-section A-A, including the embedded strain gauges. Dimensions are in mm.

connected to the segments prior to assembly. This enabled a 'plug-and-play' connection to the data acquisition system while the segments arrived in the TBM. The latter was important, as it was essential for the monitoring goals that the disturbance to the assembly process was minimal.
A segment was connected just before assembly, when the so-called erector picked up the segment. The erector is a hydraulic manipulator to pick up segments and place them anywhere on the ring diameter. The situation when a segment is picked up by the erector is a good initial situation, as the strains in the segments will be near zero (no external loads). This monitoring strategy enabled the study of all external load effects on a segment. For example, Figure 14 shows a segment just after assembly. This is a very special load case, as only two of the three jacks are active at that moment, see Figure 14.
The monitoring of the static behaviour of ring 823 lasted one week. Each 30 seconds, a sample was made of all strain gauge readings. The monitoring results confirm the results from the Second Heinenoord tunnel monitoring: the assembly and the subsequent 24 hours are the most significant in the development of the strains in a segmented lining.
Parallel to the strain monitoring, 14 pressure devices were included in the segments. These devices measured the grout pressures on the lining's exterior while the grout was in a fluid state. The pressure devices were from Wika, type 891.13.520, with a range of 3- 600 kPa and a resolution of 3 kPa. The grout pressure devices were monitored for 24 hours, in accordance with the conclusion from the Second Heinenoord tunnel that the interpretation of pressure readings is difficult as soon as the grout has fully hardened.

5 Future developments

Both Second Heinenoord and Botlek railway tunnel monitoring projects heavily relied on the information obtained from strain measurements. It appears that the interpretation of the strain data in a complex structure, like a segmented lining with gaskets, centre cones and other voids, is far from simple. This is especially true because a reading from strain gauges provides local information, while the global behaviour of the lining (like maximum bending moments) is required to improve the structural design of future tunnels. Alternative monitoring techniques need to be developed to improve the value of structural instrumentation.
In the Large Tunnel Test Facility a monitoring technique was introduced, that directly derives the global behaviour. Six very accurate rotating lasers to measure the radial distance between lining and imaginary centre of the tunnel were used to determine the radial deformations of the three rings in the facility. The lasers were from MEL, type M5L100 with a resolution of 0.01 mm at a maximum distance of 500 mm. As the lasers rotated, a complete image of the radial deformations on six positions and as a function of the polar coordinate could be established. By double derivation of the signal, it was possible to derive the curvature and associated bending moments. This technique seems to be promising for use in practice, although some practical limitations, like limited space in the TBM, may have to be overcome.
An attractive feature of the laser measurements is that it is a no-contact technique. Moreover, no special preparations of the segments are required. In general, the developments in monitoring development will favour non-contact or wireless technologies.

Figure 14. Assembly of one of the segments of the structural monitoring ring in the Botlek railway tunnel. The cable of the data acquisition system is connected to the junction box in the segment to provide monitoring data during ring assembly.

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