The 2nd Coentunnel - Design requirements from construction and service life risks.

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ABSTRACT: The 2nd Coentunnel has to solve the traffic congestion around the Dutch capital. The project consists of the design and construction of an immersed tunnel under the North Sea Canal, the upgrading of the connecting highway structure and maintenance of the assets for 30 years. The design of an immersed tunnel is strongly characterised by the construction method, the transport by sea and the shallow depth. The new tunnel has been immersed at a close distance to the existing Coentunnel. In order to manage the risks involved, specific precaution measures have been taken to protect the existing tunnel. The tunnel structure has been prepared to take the loading of a sunken ship and the impact of a dropped anchor. The paper covers the general principle of an immersed tunnel, the design requirements from construction method and maritime risks and the protection of the existing tunnel.

1 PROJECT DESCRIPTION

The Second Coen Tunnel in the A10 orbital motorway around Amsterdam is part of the larger project ‘Capacity Expansion Coen Tunnel’, including the construction of a new tunnel, the upgrading of the existing Coentunnel and several modifications to the connecting infrastructure.

Commissioned by Rijkswaterstaat (Public Works), the project ‘Capacity Expansion Coen Tunnel’ is being designed, built, financed and maintained in the context of a public-private partnership (DBFM contract) by the Coentunnel Company, a consortium of ARCADIS, Besix, CFE, Dredging International, Dura Vermeer, TBI Bouw and Vinci Grands Projects. The design and realisation of the works is being carried out by Coentunnel Construction, a joint venture of the construction companies Besix, CFE, Dredging International, Dura Vermeer, TBI Bouw, Vinci Construction and Croon Electrotechniek.

The closed part of the tunnel consists of four immersed tunnel elements (714 m in total) and two service buildings. Together with the open approaches, the tunnel is 1270 m long.

2 THE IMMERSED TUBE TUNNEL

The tunnel elements were built in the Barendrecht casting yard (Figure 1), near Rotterdam, which is owned by the the Ministry of Infrastructure and is permanently available for the construction of immersed tube tunnels in the Netherlands. Traditional formwork was used. First the floor slab was cast, then the central gallery. Finally the outer walls and roof slab were cast. Cooling was used to prevent early age cracking due to hydration heat gradients and imposed deformations.

The Barendrecht casting yard is some 150 km sailing from the project site in Amsterdam, crossing the North Sea and the locks of the port of Amsterdam.

The tunnel elements were towed using two tug boats. The tunnel elements were designed to withstand sea conditions with 2,0 meter significant wave height for normal transport conditions. A survival state was defined by 3.5 meter significant wave height and 9 second peak period. Also 0.4 m swell waves were taken into account. With these wave conditions the section forces in the tunnel element were calculated and the prestressing of the tunnel elements designed.

Figure 1. Tunnel elements in the Barendrecht casting yard.

During the normal transport conditions no joint opening was allowed. During survival conditions the prestress tendons should not break.

In a week's cycle the tunnel elements were towed from Barendrecht to Amsterdam and immersed in the trench using pontoons and a sheerleg.

The tunnel elements were placed on a temporary foundation using hydraulic rams, after which the gap below the tunnel element was filled with a water-sand slurry. After complete filling of the gap the hydraulic rams were retracted and the tunnel element could settle in the foundation layer.

The prestress tendons were cut to allow the tunnel sections to settle on the river bed and behave like the shackles of a chain. This concept of a flexible tunnel reduces the forces in the tunnel which could arise when uneven settlements occur or backfill loads are imposed on the tunnel (see section 5.3).

The section joints were provided with shear keys (in walls, floor and roof) to transfer shear loads from one tunnel section to another. Neoprene water stops were cast in, in each segment joint. The primary waterproofing in the immersions joints was provided by Gina gaskets. Secondary waterproofing was provided by Omega-seals which were bolted to cast-in steel I-beams.

3 RISK ANALYSIS EXISTING TUNNEL

The tunnel elements of the 2nd Coentunnel had to be immersed at a distance of only 13 m from the existing 50-year-old Coentunnel (Figure 2). On of the main concerns of the client was related to the structural integrity of the existing Coentunnel, which could be damaged due to construction works for the 2nd Coentunnel. Vibrations during the application of sheet-pile walls or concrete driven piles would be a real threat for the stability of the relatively weak sand-flow foundation of the immersed tunnel elements. Besides that, due to the short distance between the immersion trench and the tunnel elements, the lateral equilibrium of the construction might get lost, resulting in horizontal displacements, possible joint failure and leakage.

In order to manage the risk, the requirement specification contained the requirement that the displacement differences between two adjacent sections at an intermediate distance of 30 m should stay within the 'risk diamond' shown in Figure 3. On top of that, the client demanded a structural mitigation measure, e.g. a separation wall.

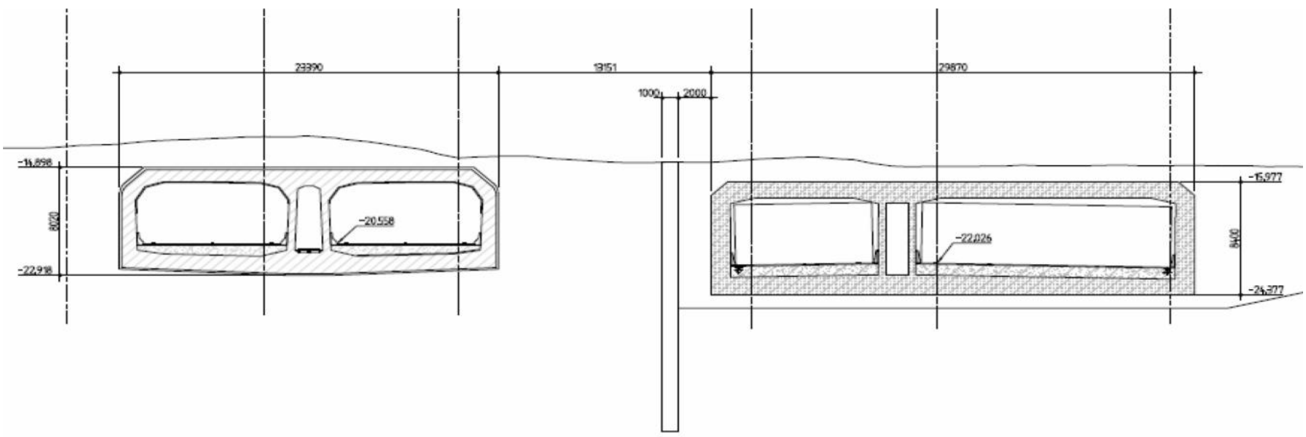

Figure 2. Typical cross section, showing the small distance between the new and the existing tunnel.

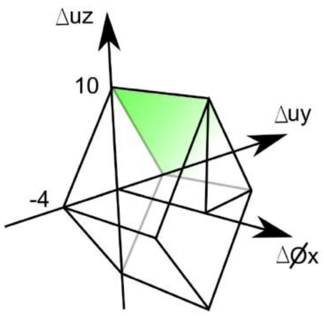

Figure 3. Risk diamond, with maximum displacement differences in mm allowed within 30 m. Note that no criterium was defined with respect to deformations.

The risks of vibrations during driving piles or sheet pile walls could easily be decreased by a vibrationless application method of the piles and walls. The lateral stability issue seemed to be a more serious issue.

The general approach for the risk analysis consisted of three parts:

- Part 1: identification of possible failure modes and the associated probabilities and consequences
- Part 2: structural analysis of worst-case scenarios in order to assess the ultimate consequences
- Part 3: assessment of the shear key strength in order to define intervention levels for monitoring and controlling the deformations.

3.1 *Failure modes*

In expert meetings the possible failure modes were identified and arranged in fault trees. Based on expert judgement, the probabilities and consequences were estimated. Based on the results of part 2 and 3, these estimates could be updated in order to get a sufficiently accurate risk estimate. For the assessment of the risks, the RISMAN method was applied.

3.2 *Structural analysis of worst case scenarios*

In order to explore the ultimate consequences, several worst case scenario's were defined, based on the assumption that the existing tunnel could be regarded as a beam with hinges (immersion joints) on an elastic horizontal and vertical foundation. In case the dredging acitivities for the immersion trench caused a geotechnical failure near the existing tunnel, both the horizontal and vertical support of the existing structure would be reduced, or even be lost. The worst case scenario's defined several sets of support reduction, with variations in the length of the affected zone, the width of the affected zone (half the tunnel width or the complete width), and the extent of the support reduction (partly or complete).

As the tunnel was regarded as a simple beam, the tunnel geometry and the support conditions had to be translated to the beam level. The geometry was translated to the beam cross section (Figure 4), the support conditions were translated in translational and rotational nonlinear spring characteristics and in forces and torques for the initial soil loads.

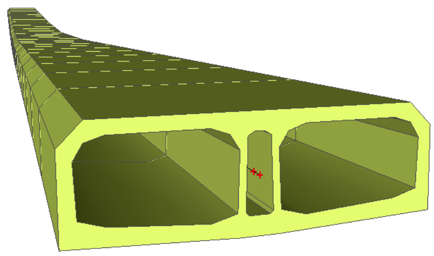

Figure 4. Model geometry: tunnel cross section applied to beam elements.

The initial and reduced support conditions were defined based on nonlinear geotechnical FEM-analyses with the Plaxis code (Figure 5), providing nonlinear spring diagrams.

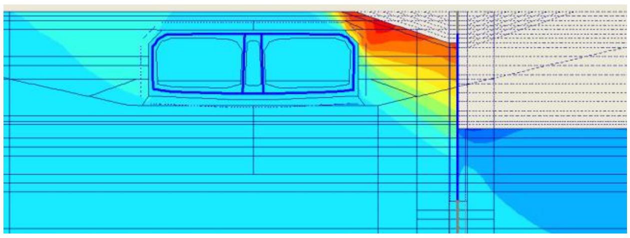

Figure 5. Typical results of Plaxis calculation: influence of separation wall failure (vertical displacements).

For the connections at the immersion joint locations two alternatives were used: real hinges (only shear force transfer) and nonlinear springs, based on the GINA-gasket properties.

This beam model gave insight in the structural behaviour under worst case conditions, expressed in terms of section forces and displacements.

The implications of the risk diamond requirement were explored by means of a shell model of 30 m part of the tunnel. This model was subjected to the required limit deformations in bending and shear, in order to find out which section forces would be needed to cause the maximum allowable deformations, and which damage (cracking, failure) would occur at given defor-mations.

Finally, the actual worst case results could be compared to the section forces associated with the risk diamond deformations. Even under these very serious conditions, the section forces (and the associated deformations) stayed well below the allowable values: see Figure 6, where the worst-case results (bullets bottom left) stay well below the failure envelopes (triangular surfaces).

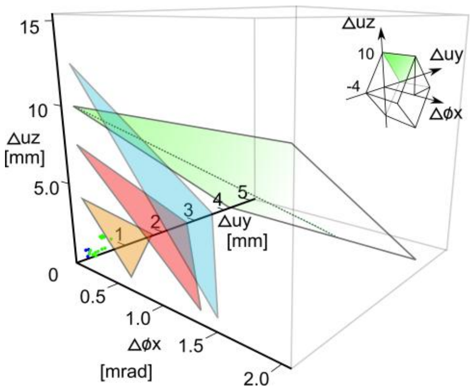

Figure 6. Comparison of worst-case results with failure envelopes.

3.3 *Definition of intervention levels*

Regardless of the low risk related to the structural integrity of the existing tunnel, still a monitoring and control strategy had to be designed. The immersion joints proved to be the most critical locations of the existing structure, with the lowest shear strength and a serious vulnerability with respect to cracking and leakage.

With help of an advanced nonlineair FEM analysis of the shear keys, the real strength with respect to shear deformations was calculated. The model used for this analysis contained a detail of the immersion joint, with the shear force transferring ring dowel (Figure 7). The model contained all the present reinforcement. Both cracking of the concrete and yielding of the reinforcement were incorporated in the material models in a realistic way. By applying an increasing lateral shear displacement of the dowel and calculating the nonlinear response of the structure, the force-displacement characteristic of the shear key could be obtained. From this analysis followed an intervention level of shear deformations of only 2,5 mm (horizontal differential displacements, measured between two points at both sides of the joint with an intermediate distance of 3 m), to be used for the design and operation of the monitoring system.

The risk analysis supported the decision to take over the risk related to the structural integrity of the existing Coentunnel, which resulted in a bonus in the bid procedure.

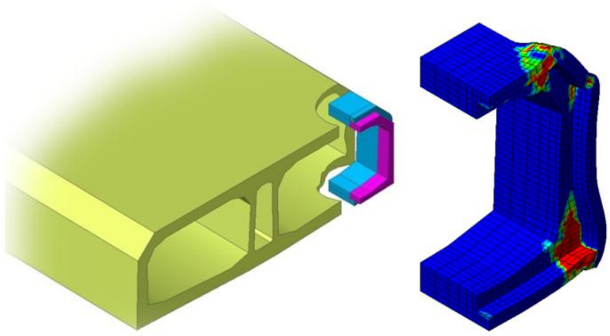

Figure 7. Finite element model of shear key detail (left). Crack pattern (right).

4 PROTECTION MEASURES

As stated before, the excavation of the immersion trench next to the existing Coentunnel was cause for concern. The distance between the second and first Coentunnel is only 12 m. The excavation of the trench would jeopardize the integrity of the existing tunnel. The contract specified very strict criteria for the maximum allowed movement and deformation of the tunnel.

To ensure stability of the existing tunnel while dredging a trench next to it, several options were considered; excavation of the trench on both sides, use of anchor piles to fix the tunnel and the use of several wall systems like sheet pile walls, combi-walls and tubular soil mixing piles. After evaluation of the constructability, costs and schedule a combi-wall system was chosen. Combi-wall systems are comprised of two main sections: steel sheet pile and a king pile. The king pile may be either a steel beam or pipe pile. In this specific case steel pipe piles 1500 mm in diameter were chosen.

4.1 *Separation wall design*

Structurally the separation wall is a cantilever retaining wall. These structures are highly sensitive to the soil characteristics. To assess the soil conditions CPT's were made every 25 meter in the alignment of the separation wall which showed the presence of the former trench and the original soil, consisting mainly of sand deposits.

The cantilever wall is loaded by earth pressure and water pressure differences, the latter being introduced by passing ships. Ships sailing at considerable speed in a shipping channel produce sizable water level variations along the length of the ship. Furthermore the ships propeller produces steep pressure gradients which act on the wall. The water level variations and pressure gradients can be translated into a pressure profile which act on the channel bed.

A FEM-model using Plaxis code was used to calculate the deformation of the tunnel during excavation of the trench and due to passing ships (Figure 5). The water pressure variations were introduced in the model by using a phreatic level according to the channel bed pressure variation and undrained behavior during the passage of the ship.

The maximum calculated horizontal displacement of the tunnel is 6 mm (including wave load 7 mm) at the most unfavorable section. In more favorable sections the displacement reduces to 3 mm (including wave load 4 mm). Vertical displacements of the tunnel elements were found to be 4 mm at the side of the separation wall and virtually zero at the other side of the tunnel. This deformation of the tunnel was considered acceptable. The combi-wall would move some 90 mm, but this would not have a significant effect on the stability of the tunnel.

4.2 *Separation wall construction*

During construction of the separation wall an extensive monitoring system was employed, measuring the movement of the tunnel elements, joint openings and deformation of the combi-wall continuously during excavation of the trench. It was shown that the measured deformations were within the range of the calculated deformations.

The stability of the foundation of the old tunnel was of particular interest. The foundation had been applied with the sand flow method, and it was unclear what the compaction of the foundation layer would be. The risk of liquefaction had to be avoided at all cost. For installation of the combi-wall hammering the pipe piles would cause vibrations which could lead to excess pore pressure in the sand flow foundation and could trigger liquefaction and compaction of the foundation layer and settlements of the tunnel. To prevent vibrations a drilling rig was developed for the project which could install the pipe piles to the required depth from a pontoon (Figure 8). A drilling motor on top of the pile drove the piles up to the required depth. The pipes were provided with Larssen interlock connections and grout tubes to fill the void on the in- and outside of the pipe with a heavy grout mix. After installation of the pipe piles, sheet piles were pushed in between connecting with the Larssen interlocks.

With this procedure the separation wall was installed free of vibration. No deformations of the tunnel were measured during the installation procedure, which has proven to be successful.

Figure 8. Drilling rig for the pipe piles of the separation wall.

5 DESIGN CHALLENGES

The design of the immersed part of the 2nd Coentunnel contained several challengeing aspects; only a few of them can be covered here.

5.1 *Asymmetrical cross section*

The tunnel had to give way for three lanes heading south and two reversible lanes, resulting in an asymmetrical cross section (Figure 2). In a symmetrical cross section, the loads on both tubes counterbalance each other more or less, and both mid walls are loaded in compression. In an asymmetrical cross section however, the load on the roof of the wide tube is only partially counterbalanced by the other tube load, and part of the load is taken by bending in the mid walls – so one of the walls is perma-nently loaded in tension, which is quite unfavourable for durability reasons and with respect to the vulnerability for crash loads.

5.2 *Maritime risks*

With regard of the intensive shipping on the North Sea Canal, the tunnel had to be designed to take the loading of a sunken ship. The client (Rijkswaterstaat, RWS) prescribes in its Guidelines for the Design of Concrete Structures that a loading of 150 kN/m^2 must be taken into account for sea-going vessels and 50 kN/m^2 for inland waterway vessels (including dynamic behaviour), and also that these loads should be verified by means of a risk analysis.

The loading of a sunken ship affected the design of the concrete structure in various ways:

- the loading must be borne by the roof (roof reinforcement);
- the loading is transferred to adjacent segments by the shear keys (shear key reinforcement) to an certain extent – depending on the stiffness of the support;
- the shear key forces lead to a greater support reaction under the adjacent segments (floor reinforcement);
- the transverse force is taken up in the walls in both the loaded and the adjacent segments (wall reinforcement).

Design for loads like these is a combination of risk analysis and force distribution. The forementioned effects are greatest if a ship sinks directly next to an expansion joint. The longer the segments, the smaller the probability that a ship sinks directly next to a joint, but the greater the forces if it happens.

As prescribed by RWS, the loading was verified by means of a risk analysis. To determine the governing ship size, an inventory of all sea-going vessels that passed the Coen Tunnel in one year was ussed. Based on these ships' characteristics (dimensions, tonnage, type of vessel), we determined for each ship the probability that the ship would sink on to the Coen Tunnel, the loading depending on the position in length and width direction and the probability distribution of this loading. In this way, a cumulative probability distribution of the loading on the tunnel was determined, and a representative ship and its associated loading could be determined. This loading turned out to be only a little lower than the value prescribed by RWS, so it was decided to use the RWS value.

The structural behaviour of the tunnel loaded by a sunken ship was calculated by means of a DIANA calculation (Figure 9), which provided insight into the distribution of the shear key forces over the four walls, the increased support reaction and the associated transverse forces.

For dropped anchors, a similar approach was followed in order to define the governing anchor weight: based on general relationships between ship tonnage and anchor weight (Luger, 2006), an anchor weight was estimated for each of the ships in the same ship passage inventory. With this data a cumulative anchor weight distribution was obtained, resulting in a governing anchor weight at an exceedance probability of once in a million years.

For further reference the reader is referred to Saveur (1997).

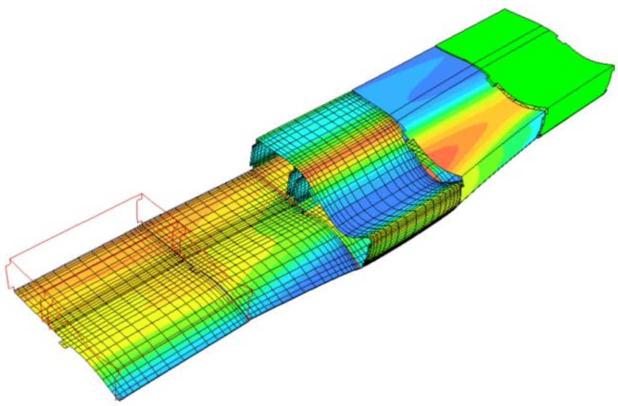

Figure 9. Displacements, force distribution and support reactions due to a sunken ship.

5.3 *Temperature influence on shear forces*

After immersion the transport prestress tendons were cut, in order to allow the tunnel to settle on the foundation. The force distribution in this elastically-supported, unevenly-loaded and subdevided beam is more complex than it first appears. The force distribution proves for example to be affected by temperature variations in the tunnel, even though these variations in the closed tunnel section are only small compared with for example the open approaches. Besides the vertical bedding, account must also be taken of a longitudinal interaction (friction), and proper account has to be taken for the non-linear stiffness of the rubber seals in the immersion joints (GINA gasket). In order to calculate and understand the longitudinal behaviour of the tunnel, a special beam action model has been used.

In this model, the segments are modelled independently as elastically-supported beams. Traditionally, a tunnel would have been considered as a chain of hinge-jointed shackles (chain model). In reality, the segments touch each other either at the upper or the lower edge (Figure 10). In the beam action model, the segment interaction in the longitudinal direction was therefore modelled eccentrically, with a very stiff no-tension-connection (block model). Furthermore, in the segment joints only transverse forces can be transferred. Also in the immersion joints eccentric contact was taken into account; the force transfer in the longitudinal direction was based on the non-linear compression stiffness of the GINA gasket.

This modelling with eccentricities is particularly necessary to determine the effect of temperature variation correctly. When for example a chain of hinged rods would have been used, with a local subsidence due to e.g. a weaker support or a concentrated soil load, then at that point a small negative eccentricity would be present. An increase in the normal force due to a rising temperature would then have led to an increase in vertical displacements, larger support reactions and thus to lower transverse forces and moments. In reality, the longitudinal contact in the segment joint at the position of the greater subsidence takes place at the upper edge: a large positive eccentricity. An increase in the normal force will then lift the tunnel segments, with lower support reactions and a larger shear force as a result (Figure 10).

This realistic modelling led to the discovery that the transverse forces and moments in the tunnel depend strongly on the temperature in the tunnel. Compared with the usual chain model, the block model used here is a better representation of reality, as it incorporates the real geometry and describes a more realistic joint interaction behaviour.

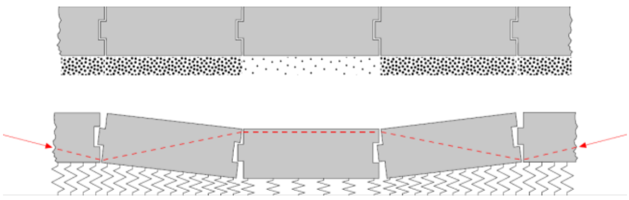

Figure 10. Normal force effects shear forces and bending moments due to eccentric contact.

5.4 *Shear keys*

The joints have been provided with shear keys to transfer transverse forces and to prevent differential deformations. The shear keys were designed for the structural behaviour resulting from the uneven loading on the tunnel and the unequal stiffness of the support. If the forces due to a sunken ship had to be transferred entirely by the shear keys, the keys would have to be extremely strong; twice as strong as was required for the other design scenarios. To avoid this, it was decided to provide the joints with additional deformation capacity for this kind of emergency loading, by allowing the teeth to fail in a controlled way. The waterstops have been designed to take these large deformations. In this way the tunnel still meets the requirements regarding accidental loads, but by means of an economic solution.

Still the shear keys turned out to be the most heavily-reinforced parts of the immersion elements.

6 CONCLUDING REMARKS

In the meantime, the Second Coen Tunnel has been opened for traffic and the renovation the First Coen Tunnel is in progress. Part of the renovation activities is the construction of a new ventilation system and the upgrading of the fire protection. Both tunnels will be opened for traffic in the summer of 2014.

PROJECT DETAILS

Project: Capaciteitsuitbreiding Coentunnel
Client: Rijkswaterstaat
Design, build, maintain and finance in the context of a public-private partnership (DBFM contract) by Coentunnel Company: consortium of ARCADIS, Besix, CFE, Dredging International, Dura Vermeer, TBI Bouw and Vinci Grands Projects
Construction: Coentunnel Construction, a joint venture of the construction companies Besix, CFE, Dredging International, Dura Vermeer, TBI Bouw, Vinci Construction and Croon Electrotechniek.

Handover: The realisation of the Second Coen Tunnel started in 2008. The tunnel elements were immersed in the spring of 2011. The tunnel was opened for traffic in May 2013. The existing Coen Tunnel is being renovated until the summer of 2014.

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