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Geotechnical design of a railway tunnel and an underground station supporting the new city hall, Delft, The Netherlands

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ABSTRACT: The Delft railway tunnel project comprises of the design and construction of a 2.4 km long, four track railway tunnel, an underground railway station and an underground parking in the historic city centre of Delft. Monuments and historical buildings are supported by shallow foundations very close to the excavations. The lowest excavation level is about 10 m below the ground surface. The ground water level is 1.5 m below ground surface. The cut-and-cover tunnel is retained by diaphragm walls in order to minimise ground deformations and to allow construction of buildings on top of the tunnel.

After construction of the railway tunnel and removal of the existing railway viaduct, 37 hectares of reclaimed public surface can be redeveloped. Public space in the city centre of Delft is scarce. Therefore several new buildings will be located on top of the tunnel. One of those buildings, a new 5 story high city hall, will be constructed on top of the proposed underground railway station. Here the diaphragm walls have a combined function; retaining soil and ground water and supporting high structural loads from the city hall. Loads from the superstructure are concentrated into columns with centre-to-centre spacing of 8.1 m. These columns are placed on top of the diaphragm walls. Minimising differential settlement is the critical structural requirement for the city hall and the tunnel. This could be achieved by assessment of required wall stiffness and tip levels using load spread models, and if necessary the application of base and shaft injection. The moment the tunnel is in exploitation the frame of the city hall is constructed.

This publication discusses the general outline of the railway tunnel project around and below the new city hall. It explains the geotechnical engineering and construction of the retaining and supporting function of the railway tunnel wall while facing the challenges of the Dutch soft delta soils.

1 DELFT RAILWAY TUNNEL

1.1 Introduction

For traffic volume and environmental reasons, the city council of Delft and the Dutch railway organization ProRail BV decided to initiate the construction of a railway tunnel through the City. The tunnel project comprises of the design and construction of a 2.4 km long four track double railway tunnel, an underground station and an underground parking. The construction of the tunnel will be mainly retained by a cut-and-cover (top-down) technique using diaphragm walls. The contract is assigned to the contractor Combination “CrommeLijn” (CCL), a co-operation between CFE NV, Mobilis BV TBI infra and Dura Vermeer Group BV. In the Design & Construct contract the engineering consultant Grontmij Netherlands BV

is responsible for the engineering. In June 2008 Grontmij started the final design. The construction started in August 2009. The project is expected to be completed in 2017.

This paper discusses the developments around the future underground railway station (Fig. 1).

1.2 Redevelopment of urban areas

After construction of the tunnel and removal of the existing railway viaduct, almost 37 hectares of reclaimed city ground can be redeveloped (Delfgaauw et al., 2010). According to the plan of the Catalan architect Joan Busquets the area will have a complete makeover. New condos with almost 1,500 apartments will be built and an extensive public transport branch point consisting of an underground railway station and a bus and tram

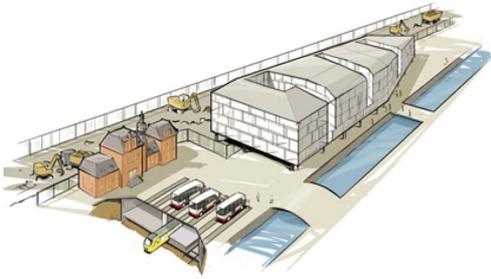


Figure 1. City hall located on top of railway tunnel.



Figure 2. Artist impression city hall, Delft, The Netherlands.

platform will be realised. A new city hall, designed by the Delft architects of Mecanoo, will be built on top of the tunnel (Fig. 2). This paper addresses the highly interactive design of this complex part of the tunnel project.

1.3 *Shallow sub/surface strata*

The shallow sub-surface strata in the western part of The Netherlands can be characterized by almost 20 m of soft soil of Holocene origin underlain by medium to dense sands of Pleistocene origin. North of the tunnel area a factory is situated that extracts large amounts of brackish groundwater from the Pleistocene sands. The drawdown results in low pore pressures in the Pleistocene sands, an advantage during construction as it reduces the risk of up-lift of the bottom of the excavation. With limited measures deep excavations can be achieved although the final tunnel construction is designed to retain higher upward water pressures in case the extraction ends.

2 ORGANISATION AND CONTRACT

The project outlined above involves complex developments of infrastructure and urban regeneration. Apart from the clients it has many stakeholders,

from owners of local stores to inhabitants, local authorities, and transport companies.

The project organisation is typical for large public works. The main client consists of a co-operation of ProRail and the city council of Delft. The clients contracted the combination DRB (DHV, Railinfra Solutions and Benthem Crouwel Architects) for preliminary design. Extensive risk assessments regarding safety and legal aspects were performed, as this is the first project in the Netherlands where large buildings are supported by and integrated with a railway tunnel. After proving the technical, financial and legal feasibility, the preliminary design was back-translated to functional specifications in a D&C contract that was awarded to CCL.

Mecanoo and ABT are designers of the city hall, which will be tendered in a separate contract. All designing parties work closely together with the contractor, the client and its advisor DRB in so-called interface meetings to integrate their partial designs.

3 CONSTRUCTION

3.1 *Main design choices*

The main design challenges are:

- Differential settlements and settlements of the tunnel due to the loads of the city hall need to be limited because of serviceability of the rail tracks, leakage of the tunnel structure itself and the stability of the city hall.
- The presence of the historical station building, supported by a shallow foundation close to the excavation.
- The limited available space.

Diaphragm walls and barrettes were selected as main foundation elements because of their stiffness in both horizontal and vertical directions and the construction method leads to minimal deformation of nearby buildings, e.g. the historical railway station.

3.2 *Construction sequence*

The building pit for the underground station comprises three boxes (northern, central and southern), separated by watertight sheet pile walls. This subdivision is implemented to allow for staged construction dictated by the interfaces with existing nearby structures and the construction itself. Below the layout of the railway station is described from north to south.

3.2.1 *Northern part*

The northern part of the railway station has two representative sections (Fig. 3). One section has a

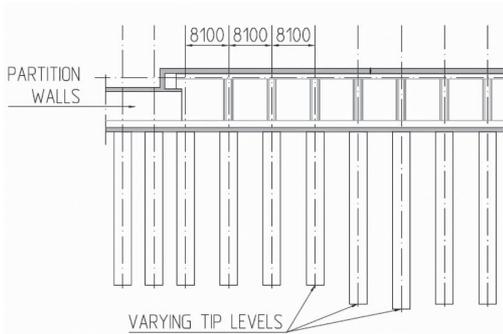


Figure 3. Longitudinal cross section northern part.

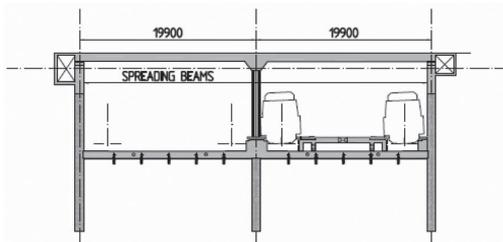


Figure 4. Cross section northern part.

lower roof and as a consequence an earth cover of several metres. The second section has a 2.7 m higher roof. Between both sections a solid concrete beam crosses (Fig. 4) the station transversally from one outer diaphragm wall to the other outer diaphragm wall. The span is approximately 40 m. The excavation depth is 10 m. To reduce the necessary dimensions of this beam, it is supported by the tunnel partition walls. This means that the generally applied top-down construction sequence had to be altered to a bottom-up sequence in the lower northern section in order to finish the supporting walls and transversal beam before the roof could be concreted. Moreover, the process of staged dewatering, as is applied in other tunnel sections, was changed to a complete draw down to the base floor level. To avoid a too high quantity of dewatering, the necessity of compartment walls was obvious.

In the second section, with the high roof, central columns with longitudinal spacings of 8.1 m support the roof, while at the far ends the roof slab is fixed to the outer diaphragm walls. As the high structural column loads of the city hall have to be supported by the diaphragm walls and the central columns, measures had to be taken to limit possible differential settlements. Due to the great variation of concentrated vertical structural loads

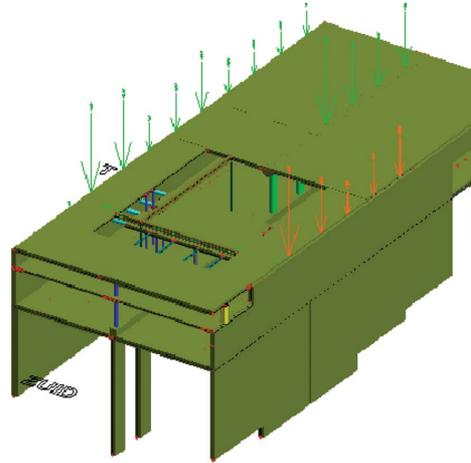


Figure 5. Structural model northern part.

(4 MN to 28 MN), large, solid spreading beams had to be casted on top of the outer diaphragm walls. Conservatively, the load spread through the joints of diaphragm wall panels by mobilisation of side friction was not taken into account. Load spread could be achieved with beam heights of approximately 3.0 m. Below the central columns, barrettes were applied with varying tip levels to achieve the required bearing capacity and stiffness. Iterative structural and geotechnical design (Fig. 5) was required with regard to scenarios with upper bound ($\sqrt{2} \cdot k_{\text{mean}}$, kN/m) and lower bound ($\sqrt{2}/k_{\text{mean}}$, kN/m) vertical stiffness characteristics.

The section with the high roof was constructed top-down. After installation of the diaphragm walls and barrettes, the spreading beams are casted after partial removal of the top section of the diaphragm walls. Reinforcement of the concrete spreading beams is connected to the reinforcement cages of the diaphragm walls. Before covering the tunnel section with the concrete roof, anchor piles are installed which counterbalance uplift below the future tunnel floor.

3.2.2 Central part

The central part of the underground railway station is characterised by two important aspects. At one side there is the historical building of the old railway station that needs to be preserved. It is located at no more than 3.0 m distance from the outer diaphragm wall. On the opposite side, the future ground surface level is equal to the intermediate floor level of the underground railway station, about 3.0 m below the tunnel roof slab. This asymmetrical load situation had to be addressed in both design and construction (Fig. 6).

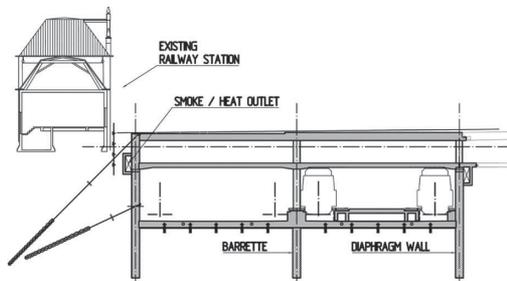


Figure 6. Cross section central part.

In order to reduce the deformations of the historical railway station, the allowable horizontal diaphragm wall deflection (1/100 of the excavated height) that was stipulated for the northern and southern part has to be limited. Therefore the diaphragm walls will be installed with smaller panel widths of 3.8 m in comparison with panel widths of 7.3 m as applied elsewhere. By doing this, the expected horizontal ground deformations during the bentonite phase of the diaphragm wall excavations will be reduced. Furthermore, pre-stressed grout anchors are applied to support the wall between the intermediate floor and base floor. After installation of the outer diaphragm walls and barrettes, the diaphragm wall near the historical railway station will be anchored at its top level. After excavation to the intermediate floor level, anchor piles will be installed and the intermediate floor is cast. The next stage is the installation of pre-stressed grout anchors at a level between the intermediate floor and the tunnel floor. The last stage comprises of casting the roof slab and supporting columns. Now, the pre-stressed grout anchors can be relieved and removed.

3.2.3 Southern part

The southern part is also subdivided into a high and a low section (Fig. 7). The higher section will be constructed top-down and the lower section bottom-up.

The construction sequence of the low section was introduced to facilitate casting of five partition walls. At the extremity of this section however, a length of 14.0 m of the roof slab is cast using the top-down method, as this slab temporary supports a compartment sheet pile wall. The reinforcement in the connection roof/diaphragm wall was designed taking into account the self weight of the slab and supports by the partition walls (top-down).

The high roof slab settlement which takes place during top-down construction requires compensation. Before placement of fill on top

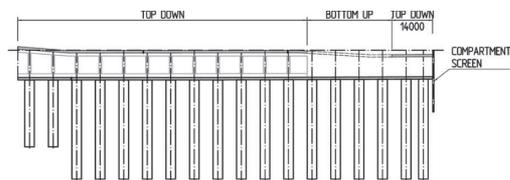


Figure 7. Longitudinal cross section southern part.

of the roof, the slab will be jacked up to reduce bending moments and shear forces at the slab/diaphragm wall connections. Reaction force is obtained from the partition walls.

3.3 Definition of concrete mixture

In the southern part, the bending moments in the connection diaphragm wall/roof slab are very high. Required spacing of vertical rebars in the diaphragm wall did not meet the Dutch code (NEN-EN1538, 2000). During the design process, it became apparent that geohydrological conditions were different than originally anticipated for. This resulted in even higher bending moments. As the required reinforcement could not be applied, a test programme was executed to find the optimal concrete mixture and the required casting method to assure a satisfactory result. The test was performed in shallow diaphragm wall shaped excavations (Fig. 8) which included representative reinforcement. Four different concrete mixtures and/or casting methods were applied to investigate the replacement of bentonite by concrete in the dense reinforcement cages. After hardening, core samples were collected for testing concrete quality and concrete distribution between the rebars and couplings. A concrete mixture with aggregate size 4/16 without any treatment after pouring (i.e. vibration, poking, etc.) proved to be the best option.

3.4 Barrettes

At midspan the tunnel roof is support by a row of columns with spacings of 8.1 m. These columns have longitudinal concrete beams above and below. The lower beam, being part of the tunnel floor, is supported by barrettes instead of a continuous diaphragm wall. As it would be too expensive to cast barrettes up to roof level and shorten them afterwards down to tunnel floor level, temporary steel columns were applied in this section. The barrettes were equipped with reinforcement cages which reach up to the future tunnel floor level. Before installation, the reinforcement cages and steel profiles were welded together at ground surface. The reinforcement was lowered into the trench.



Figure 8. Concrete mixture test.

Finally, the barrettes were casted to future tunnel floor level, and after sufficient hardening, the section between future floor level and ground surface was back-filled with gravel. The steel profiles could be used to support the roof during top-down construction. The gravel can easily be removed during the excavation. After the floor is cast, a formwork can be placed around the steel profiles to give the columns a concrete look.

4 ENGINEERING DIAPHRAGM WALLS

4.1 Combined function diaphragm walls

The diaphragm walls below the city hall have combined functions. They retain ground and groundwater pressures and they support the city hall located on top of the tunnel. In accordance with CUR166 (2005) interaction of the vertical and horizontal wall stability should be analysed where vertical loads are substantial (over 12.5 kN/m^2). In this case loads from the city hall are over 50 kN/m^2 .

The CUR166 (2005) is a leading handbook for civil engineers. The user is guided through design and construction aspects of soil and ground water retaining constructions. The handbook will become part of the Dutch national annex to Eurocode 7.

The horizontal stability of the propped diaphragm walls was assessed using the computer program MSheet (Delft Geosystems, The Netherlands) based on the Winkler theory of beams with elastic-plastic spring behaviour. Separately, assessment of the vertical stability of the diaphragm walls were performed in accordance with the Dutch standard for compression piles NEN6740 (2006) and NEN6743-1 (2006) applying the computer program MFoundation (Delft Geosystems, The Netherlands). Pile design was based on the Dutch

q_c -method and took account of stress reduction by trench excavation and a complex geo-hydrological profile. The described design routine proved to be very practical. However to verify the results of the separate assessment of horizontal and vertical stability, a finite element study was performed using Plaxis 2D.

4.1.1 Horizontal stability

According to design approach B outlined in CUR166 (2005) a finite element study (FEM) was performed using representative ground parameters, groundwater levels and geometry in the different construction stages. The hardening soil model was applied with parameters such as c' , ϕ' , $E'_{50,ref}$, $E'_{oed,ref}$ and $E'_{ur,ref}$. For critical construction stages separate calculation steps were added in the FEM using design parameters. To obtain maximum displacements, forces and bending moments, drained soil behaviour was applied to the model. In the FEM the direction of the friction angle (δ , °) at the concrete wall and soil interface is automatically determined for vertical loading of the wall.

In the elasto-plastic spring model the approach is different. The model uses parameters such as c' , ϕ' , δ' and k_h . k_h (kN/m^3) is the horizontal subgrade reaction. In the final construction stage the diaphragm wall is vertically loaded up to 1.9 MN/m^2 . The balance of vertical forces has to be achieved by rotating the wall friction angle bottom-up, from tip level to about the excavation level, at the active side of the retaining wall. Results of both calculations are given in Table 1. The presented case has diaphragm wall tip levels of NAP -23.5 m penetrating 5 m into dense to medium dense Pleistocene sand.

The verification results for horizontal stability of the retaining walls show that:

- The calculated maximum bending moments (ULS) in FEM and the spring model are comparable, but appear in different construction stages.
- Introduction of vertical loads increases maximum bending moments in the retaining wall.

Table 1. Calculation results FEM 2D and Spring Model.

Construction stage	FEM 2D	Spring model
	$M_{max,ULS}$ [kNm/m^2]	$M_{max,ULS}$ [kNm/m^2]
Maximum excavation level	1,010	1,356
Completion tunnel	1,154	993
Vertical load from superstructure	1,308	989

$M_{max,ULS}$, Maximum bending moment in diaphragm wall (ultimate limit state).

- In the spring model no clear increase is observed of the maximum bending moment between completion of the tunnel and application of vertical load on the retaining wall.

This practical design approach was maintained. However a second order bending moment was added to the results of the final spring model construction stage. This additional moment was calculated using the maximum deflection of the retaining wall from the spring model calculation and the introduced vertical load from the superstructure. For the case presented in Table 1 this was 135 kNm/m' (0.07 m × 1,9 MN/m'). The results from the FEM and the spring model become comparable, 1,308 kNm/m' versus 1,154 kNm/m'.

4.1.2 Vertical stability

To verify the design approach comparison is made between results of FEM analyses, results of spring model analyses and results of the Dutch q_c -method in the ULS.

In the FEM and the spring model wall friction was mobilised over the wall length including the Holocene soil layers. The q_c -based method took account of wall friction from the top of the deep Pleistocene sands. In the FEM and the spring model the wall friction is assessed using the slip-method while the Dutch q_c -method relates wall friction to q_c using empirical pile constants ($\alpha_p = 0.5$, $\alpha_s = 0.006$ and $s = 0.63$ according to NEN6743-1 (2006)). End bearing of the spring model was also based on the q_c -method, where in the FEM the end bearing was assessed by multiplication of effective stress below the tip of the diaphragm wall (including the effective weight of the wall) with the tip surface. A summary of calculation results is presented in Table 2.

The conclusions drawn from the verification of the design approach for vertical stability are as follows:

- Total wall friction assessed with the FEM and the spring model is comparable.

Table 2. Comparison of bearing capacities (ULS) of different models.

Model	$F_{\text{shaft,d}}$ [kN/m']		$F_{\text{tip,d}}$ [kN/m']	F_d [kN/m']
	H	P		
FEM	360	310	1,260	1,930
Spring model	200	470	1,520	2,190
Dutch q_c based method	-	480	1,520	2,000

$F_{\text{shaft,d}}$: ULS shaft resistance; $F_{\text{tip,d}}$: ULS end bearing; F_d : ULS total bearing capacity; H/P, Holocene/Pleistocene.

- Wall friction in the Pleistocene sands assessed with the spring model and the q_c -based model are equal.
- End bearing assessed with the FEM is underrated because the vertical balance is reached at a limited vertical deformation of 40 mm ($0.03D_{\text{eq}}$) in ULS. According to NEN6743 (2006) the maximum deformation in ULS is 0.1 to $0.2D_{\text{eq}}$.
- Assessment of vertical stability using the Dutch q_c -method separate from horizontal stability is possible. However, wall friction in the Holocene soil layers is neglected.

4.2 Vertical support and deformations

The city hall columns are supported by the outer diaphragm tunnel walls and the barrettes at the centre of the tunnel. Strict differential settlement criteria were prescribed in the tunnel contract to assure the structural integrity of the city hall. In transverse direction differential settlement between adjacent structural columns has to be reduced to <20 mm (Fig. 9) and in longitudinal direction to <15 mm (Fig. 10). Maximum absolute vertical column settlement has to be limited to 30 mm.

According to the Dutch standard NEN6743-1 (2006), the load-settlement behaviour is a function of the ratio of load and resistance. Load-settlement

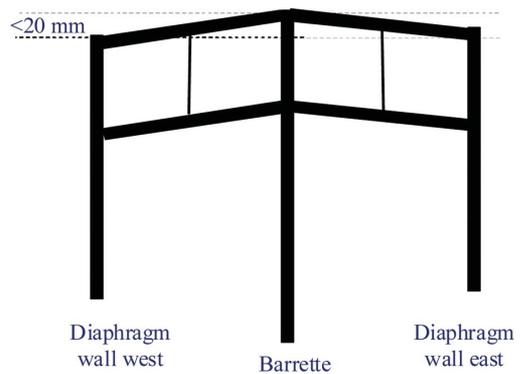


Figure 9. Transversal differential settlement.

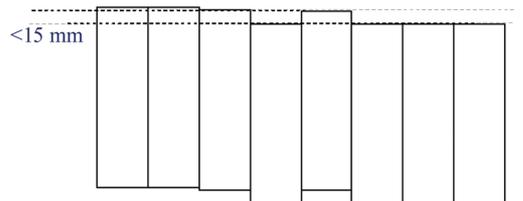


Figure 10. Longitudinal differential settlement.

behaviour of diaphragm walls and barrettes is assumed to be comparable to that of bored piles.

To control (differential) settlement three possible options were considered:

1. Post base and shaft grout injection. Application of this technique increases pile stiffness as well as the bearing capacity. The technique has not widely been applied to improve the performance of diaphragm walls and barrettes. The effect of post injection for this project could only be verified by load tests.
2. Spreading beam on top of the diaphragm walls and barrettes for steady load distribution at tip level.
3. Varying tip levels to control vertical stiffness.

A combination of options 2 and 3 was selected. A feasibility study indicated that the increase of tip levels by about 7 m is equivalent to the application of shaft and base injection. In addition, prediction of performance of diaphragm walls after injection is subject to uncertainties (Hamza & Ibrahim., 2000).

A summary of the calculation results is as follows. City hall column settlements between 5 mm and 25 mm were assessed. The maximum column loads of up to 28 MN per barrette (L/W is 3.3 m/1.0 m) requires tip levels of 46.5 m below ground surface which is comparable to Thasnanipan et al. (2001).

Because of the high ground water levels impermeable retaining walls are required. Vertical loading conditions however result in irregular settlements of the diaphragm wall panels. The spreading beam contributes to reduce differential vertical settlements of neighbouring panels to <5 mm. Joints between diaphragm wall panels have limited differential settlement capacity to a maximum vector deformation of 25 mm. Therefore, measures such as post joint injection are available for instant use during construction.

5 RISK EVALUATION

5.1 General

The design process and the construction of the tunnel are based on the method of systems engineering (Everaars et al., 2010). SE translates abstract top-requirements into more practical requirements.

The design process produced additional requirements which were introduced in SE. Specifications are systematically evaluated and broken down further into practical requirements for construction. Critical requirements for design and construction of the city hall tunnel section are outlined in the sections below.

Parallel to design of the Delft railway tunnel an expert panel produced a state of the art report on design and construction of diaphragm walls (CUR231, 2010). Best practices and main risks regarding diaphragm wall supported excavations were addressed.

5.2 Engineering

Interaction of structural loads, soil properties and foundation stiffness were very sensitive when calculating differential settlement. Therefore extensive SI was performed to derive soil profiles and soil parameters. Additional SI was performed to mark out a clay layer thinning below the city hall.

Structural engineers and geotechnical specialists combined forces during an iterative design approach, while project management focussed on coordinating with the external structural engineers for the city hall superstructure.

5.3 Construction

In order to reduce failure risk, additional measures, such as monitoring, tests and adjustments of construction methods are taken.

5.3.1 Monitoring and testing

In order to validate the applied design methodology, monitoring instruments are attached to some cross sections before the start of construction. Inclinometers are installed at close distances to the diaphragm wall and at one location directly inside the diaphragm wall. A series of settlement markers up to a distance of 25 m of the diaphragm wall is installed to monitor horizontal and vertical ground deformations. Monitoring data enables verification of design models.

As recent experiences learned that failures of diaphragm wall trenches were often caused by joint anomalies (bentonite inclusions), sonic logging equipment was installed at panel joints close to existing (historical) buildings. The equipment comprises four watertight HDPE tubes (two at both sides of the joint), fixed to the reinforcement cage. After concrete hardening, sonic logging is performed by lowering transmitters and receivers at opposite sides of the diaphragm wall for joint inspection.

5.3.2 Adjusted construction methods

During the design process and construction preparation measures were proposed to control risks. In some cases the design was adjusted. Below two examples are given.

Initially, a short sheet pile wall was anticipated for at a minimum distance from the façade of the historical railway station to improve trench stability.

The idea was that this sheet pile wall, which is necessary to construct the heat/smoke outlet conducts between the façade and the future tunnel, would reduce the required bentonite level in the stable diaphragm wall trench. Analyses, including introduction of stabilised soils between sheet pile wall and diaphragm wall, however resulted in low safety factors. For this reason high bentonite levels were required and installation of the sheet pile wall was scheduled after concrete hardening of the diaphragm wall.

Pre-stressed grout anchors in the central section of the railway station were originally planned to pass through metal tubes incorporated in the reinforcement cages. As these tubes should form an important rheological obstacle during the concreting of the diaphragm walls, this concept was left. The lower grout anchors with spacing of 1.9 m (two per panel) will now pass through drilled holes. Those holes will be cored during excavation. To avoid drilling through reinforcement special cages were designed. The holes for the top anchors, with spacing of 3.8 m (one per panel) could pass through the diaphragm wall in the unreinforced central zone. This zone is where the concrete is tremied.

5.3.3 Diaphragm wall joints

The tunnel design addressed risks related to the required impermeability of diaphragm wall joint (analyses of horizontal and vertical stiffness and the introduction of a spreading beam). Quality control of underground works is difficult and no proven in-situ test is available to investigate joint quality. For this reason extensive monitoring will be performed during the excavations. After detection of leakages immediate post joint grout injection will be applied.

5.4 Project management

When working on a project with different parties and different contracts, proper communication is indispensable. In this project this is achieved by intensive interface management. Ideally, discussions lead to solutions that are favourable for all parties involved. In some occasions, decisions have to be made for the benefit of the project as a whole. The Client's Change Control Board decides in cases where contract changes are required to improve the overall design or to minimise risks, such as:

- Contractually, the designers of the city hall had to take measures to reduce train induced vibrations. However, it proved impossible to incorporate this in their concept and it was decided to take measures underneath the railway tracks.

- It was decided to transfer parts of the scope of the city hall (an underground bicycle park directly adjacent to the tunnel) to the tunnel contract, in order to minimise construction, logistic and planning risks.

6 CONCLUSION

Application of diaphragm walls in the cut and cover tunnel reduces ground deformations and therefore minimises settlement of nearby buildings. The relatively high bearing capacity of diaphragm walls creates opportunities to build on top of the tunnel.

A new city hall will be constructed directly on top of the railway tunnel over a length of 130 m. Limiting (differential) settlement of the diaphragm walls was the critical structural requirement here. Load-settlement behaviour is controlled by using spreading beams and variation of diaphragm wall tip levels.

In consequence of construction on top of the tunnel, the diaphragm walls below have two interacting functions. They retain ground and ground water and they provide vertical support. A FEM analysis has proven that assessment of horizontal and vertical stability can be performed separately.

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