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Marieholmstunnel

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ABSTRACT

The Marieholm tunnel is a part of the Marieholm Connection Project, which connects the Port of Gothenburg and the Industries at Hisingen with the central part of Gothenburg. The closed section is 500 m long with three traffic lanes in each direction. This paper presents some of the main decisions and challenges, which shaped the construction of the Marieholm project. The challenge of the project was construction in the city with limited space and difficult transport logistics together with ongoing ship traffic. Both up and down stream an existing tunnel and bridges limited transportation width and depth. Soil conditions with 60–100 m of soft (Gothenburg) clay layer made deep excavations difficult. The immersed tunnel was constructed as three 102 m long elements constructed one after the other in a dry dock within the alignment. The construction pit was built with steel tubes as retaining walls, underwater excavation and a bottom slab cast underwater. During construction wall deflection, bottom heave and strut forces were closely monitored. The tunnel elements were temporarily supported on steel piles while being sandflowed.

1. Introduction

Gothenburg, Sweden's second largest city, is bisected from Southwest to Northeast by the Göta River. Only a small number of crossings connect the city centre to the port, industrial areas and suburbs on the north side. This is the cause of growing capacity problems.

Preparations for a new road crossing to relieve the existing immersed tunnel, the Tingstad Tunnel, were undertaken by the Swedish Road Authorities (Trafikverket). The crossing was planned as a tunnel rather than a bridge, because of the implications a bridge would have on traffic in the urban area. A low movable bridge would not be efficient for the number of vehicles passing, and there was not enough space for a high bridge in the urban area.

A location 700 m upstream from the Tingstad tunnel was chosen. The initial study was led by the Swedish Road Authorities (Trafikverket) and carried out by COWI AB.

The project was tendered in 2013 and awarded to JVM (JV Marieholm) in 2014, a joint venture consisting of the main contractors Züblin AB and Boskalis, with Arup as lead and geotechnical consultant. TEC headed the design of the immersed tunnel permanent works with MH Poly providing transport and immersion consultancy. Temporary works design was generally undertaken by Züblin's technical office. During the design and construction period COWI remained Client's consultant for Trafikverket.

The tunnel has three lanes in each direction and has been dimensioned for speeds of 50–70 km/h. Design ADT is 90,000 vehicles.

The Marieholm Tunnel is just under 500 m long, the external dimensions of the tunnel cross section are 31 m width and 9.5 m height. The lowest point of the tunnel bottom slab is 17 m below water surface.

The tunnel was opened to traffic December 2020 (see Figs. 1–3).

2. Scope

This article elaborates on the path from early evaluation of alternative IMT construction concepts to the winning joint venture's eventual choice of concept. It further discusses some of the particular and unique challenges faced in relation to the construction of the Marieholm immersed tunnel, owing to the choice of concept and the constraints and conditions (geotechnical, environmental and infrastructural) at the tunnel site. An important aspect of the winning joint venture's concept was the deep construction pit/dry dock established in the geotechnically volatile Gothenburg clay, which enabled casting of the tunnel elements at the site. This solution was selected over other concepts, which involved either casting the tunnel elements off site or excavating larger

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Fig. 1. Regional location of project (source: COWI).

Fig. 2. Project overview (blue river crossing: Existing Tingstad tunnel; red river crossing: Marieholm tunnel) (source: Trafikverket).

but shallower construction pits to reduce the length of the immersed tunnel. Therefore, in this article particular focus is given to the monitoring and deformation behaviour of the deep construction pit.

Design challenges for the permanent structures and lessons learned owing to e.g. the direct foundation on the Gothenburg clay are not discussed in the present article.

3. Limitations due to construction in the city

Already in the early stages of the project it was identified that the location of the Marieholm tunnel construction site would impose constraints on the immersed tunnel construction.

3.1. Neighbour structures

The construction site was enclosed by the Tingstad Tunnel and Götaälv Bridge downstream, and the closely spaced old and new Marieholm railway bridges upstream. The existing river crossings at construction start are shown in Fig. 5. In addition, construction works on the new Hisingen Bridge replacing the Götaälv Bridge at the same location

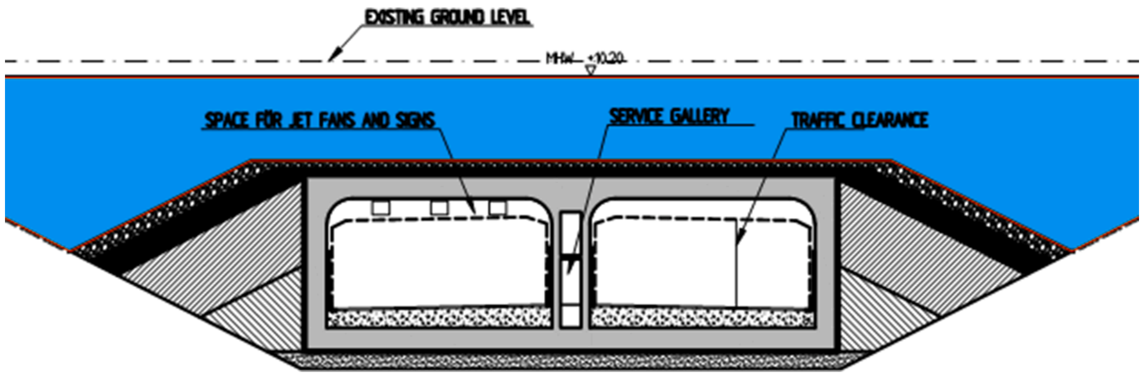

Fig. 3. Tunnel cross section.

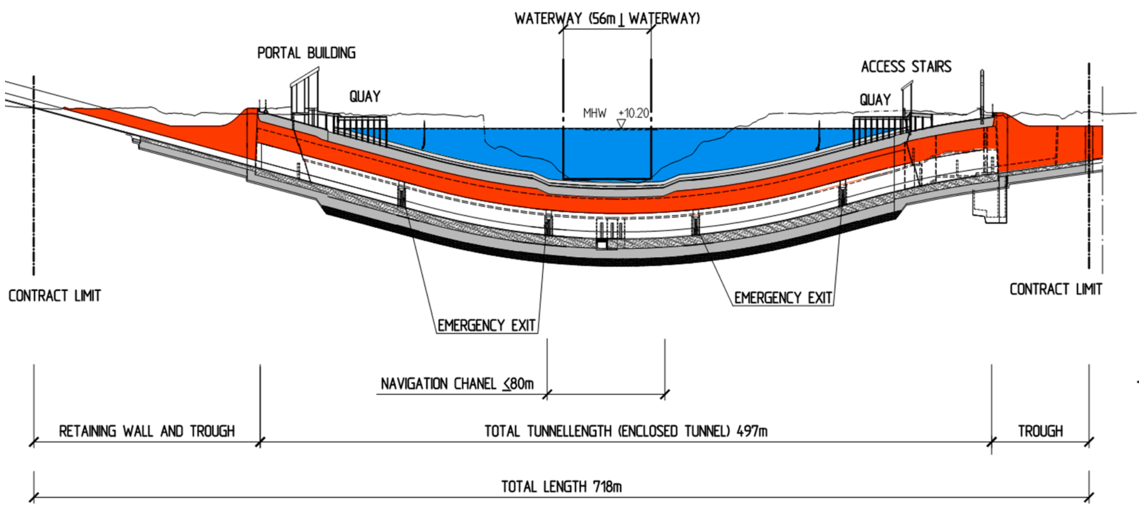

Fig. 4. Tunnel longitudinal section.

as this, were planned to run in parallel with the construction of the Marieholm tunnel.

This imposed the following limitations in case tunnel elements had to be floated in from a construction facility off site:

- Tingstad tunnel: 7.8 m navigation channel depth
- Götaälv bridge. Main span: 6.5 m clearance by 20 m width. Side spans: 5 m clearance by 31.5 m width
- Marieholm bridges. Approximately 20 m width of all spans

3.2. Navigation channel, seasonal window for dredging/earthworks, saltwater wedge

The required navigation channel at the tunnel site is 6.9 m deep and 50 m wide and placed centrally in the approximately 150 m wide river. Approximately 4900 vessels per year pass the site. Only limited closure of the ship navigation is permitted.

Dredging in the river is for environmental reasons only allowed to take place in the winter period September 15th to April 15th.

4. General conditions and specifications

4.1. Ground conditions

The valley of the Göta River is characterized by thick layers of soft clay. The thickness is about 60 m on the Tingstad (West) side of the tunnel and more than 100 m on the Marieholm (East) side. Underneath the clay is a 0 to 15 m thick layer of frictional soil underlain by rock.

The soil unit weight varies between 15 and 17 kN/m^3.

The natural water content decreases from 70 to 90% at ground surface to 50–75% at level −20 m. Below level −30 m the natural water content decreases further. The natural water content is close to and in some cases above the liquid limit above level −15 m. Below this level, the natural water content is lower than the liquid limit of the material.

The in-situ undrained shear strength is $c_u = 12$ kPa at ground level increasing with 1.2 kPa/m depth. The clay is slightly over consolidated, with OCR = 1.25.

4.2. Design conditions

The service lifetime of the tunnel is 120 years. The design is based on the Eurocodes with Swedish National Annexes. The concrete strength is C35/40 and the crack width criteria is generally 0.20 mm. The tunnel is designed for the accidental situations; sunken vessel, falling or dragging anchor, explosion, fire and flooding. The fire requirements were fulfilled by applying fire boards to the roof, sprayed fire protection to the rounded corners and prefab element on the walls of the road tubes.

5. Owner’s considerations

During the initial design stages, the Owner established the tunnel dimensions.

The Owner’s initial idea for constructing the tunnel elements was to establish a dry dock in the alignment. Tunnel elements would then be constructed, floated out and immersed sequentially. Totally five elements were foreseen. Due to the seasonal restrictions on dredging works

Fig. 5. Enclosed project site (source: COWI).

this resulted in a too long construction period. As a consequence, alternative methods were considered.

The element dimensions, when compared with the clearance limitations imposed by the nearby tunnel and bridges, showed that it would not be possible to fully construct and equip tunnel elements elsewhere, and then float them to site. In practice, only three main options remained:

- Option 1: Cast the elements in a construction pit at the location of the future Cut and Cover approach tunnel(s), i.e. in the alignment
- Option 2: Cast the elements partly in a dry dock either at the site or off site and complete the casting afloat at site
- Option 3: Cast the elements on site in a casting yard next to the alignment

For each of the main options two alternatives in terms of total IMT length and number of tunnel elements were evaluated on three criteria: Construction cost, construction time of civil works and construction risks.

In particular related to construction risk, primary attention was given to three risks:

- disturbance of ship traffic during construction
- dredging, excavation and reclamation quantities
- construction pits in the Gothenburg Clay

The six considered alternatives varied significantly in IMT length. The two alternatives of Option 1, which was cast in the alignment, were shortest and therefore resulted in small dredging quantities (only 160 m to 200 m IMT length consisting of two tunnel elements), but were associated with large reclaimed areas and deep construction pits. This in turn resulted in large excavation quantities at both riverbanks. For these alternatives the main principle was that one tunnel element would be cast in each construction pit simultaneously, thereby among other things reducing the construction period.

Option 2 and 3 varied between 380 m and 408 m in IMT length. In particular for Option 2, which was cast afloat, a main advantage was to reduce the footprint and complexity of the construction pits for approach tunnels and ramps. This reduced excavation quantities compared with other options.

The study indicated that Option 3 with construction of a casting yard next to the alignment was the most expensive solution. In addition it was found to result in both the longest construction period of approximately 48 months and the highest construction risk level related to the large construction pits in Gothenburg Clay. As a direct consequence Option 3 involved large excavation quantities.

A construction pit with the required depth in Gothenburg clay would typically involve diaphragm walls, soil improvement within the dry dock and several levels of propping or anchoring of the walls.

Options 2 and 3 were similar in expected construction time, approximately 42 months, with Option 2 cast afloat turning out to be preferable both in terms of cost and construction risk.

The conclusion of the study was the recommendation of an immersed tunnel with 4 × 102 m long elements cast partially afloat. The tunnel should be constructed over a period of totally 70 months (from January 2015 (first closure of street crossing the construction site) to October 2020).

The choice of construction method was left open to the Contractor in the Tender documents. It was made sure that all of the studied options would be covered by the permission from the Environmental authorities (Mark- och Miljödomstolen).

6. Selected solution

6.1. Immersed tunnel concept

The winning joint venture chose to develop a concept where the IMT was cast in the alignment. As opposed to the options considered during the early studies, only one production line was anticipated. This was situated in a construction pit on the Marieholm side of the river. The

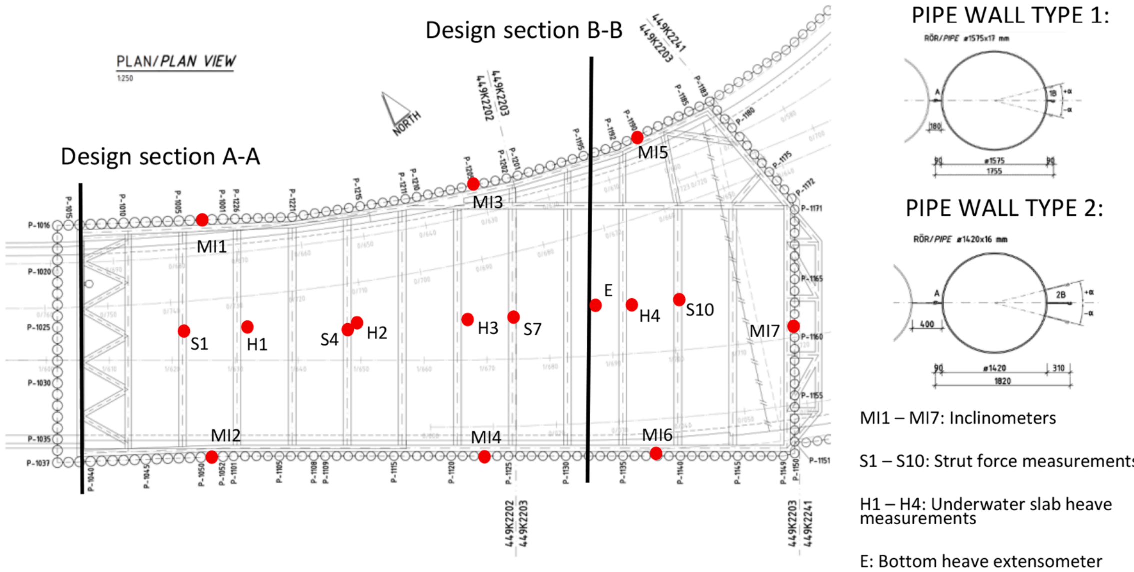

Fig. 6. Plan view of the construction pit.

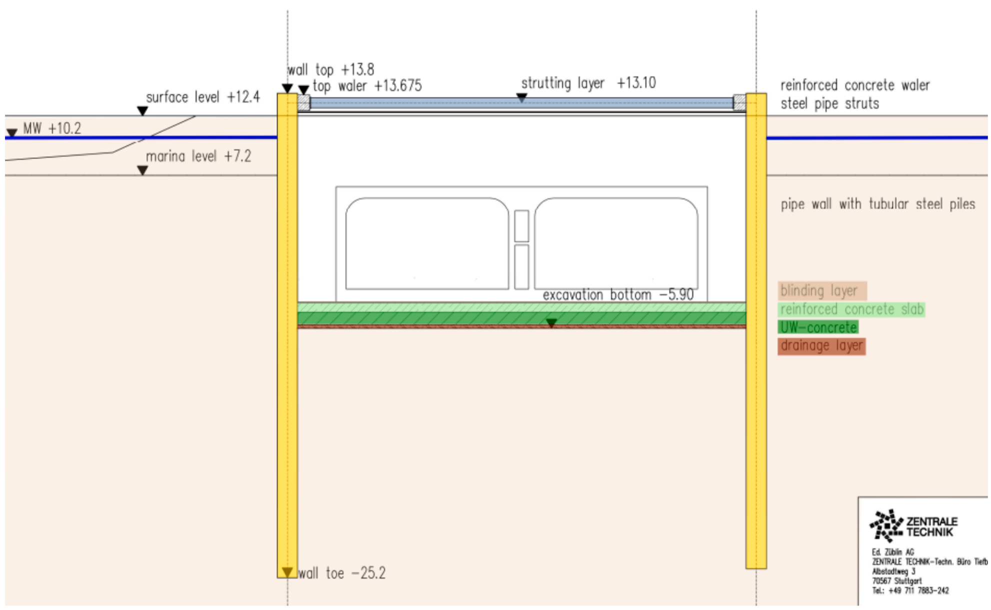

Fig. 7. Cross section of the construction pit (design section A-A).

Table 1
Hardening Soil model with small-strain stiffness parameters.

Unit weight γ	kN/m^3	16.4
Effective cohesion c'	kN/m^2	1.2
Increment c'_{incr}	kN/m^2/m	0.13
Effective friction angle φ'	°	30
Dilatancy angle ψ'	°	0
Oedometer modulus E_{oed}^{ref}	kN/m^2	5800
Secant modulus E_{50}^{ref}	kN/m^2	11,000
Un-/reloading modulus E_{ur}^{ref}	kN/m^2	31,000
power (m)	–	0.9
K_0^{nc}	–	0.55
OCR	–	1.25
K_0	–	0.625
ν_{ur}	–	0.2
R_f	–	1.0
G_0^{ref}	kN/m^2	26,600
$\gamma_{0,7}$	–	$4 \cdot 10^{-4}$
permeability $k_{h,v}$	m/s	Variable 10^{-8} to $0.5 \cdot 10^{-9}$

Table 2
NGI-ADP model parameters.

Identification		Clay	Clay 2
Material model		NGI-ADP	NGI-ADP
Drainage type		Undrained (C)	Undrained (C)
γ	kN/m^3	16.5	16.5
G_{ur}/s_u^A		1000	1000
γ_f^C	%	1.0	1.0
γ_f^E	%	2.1	2.1
γ_f^{DSS}	%	2.0	2.0
ν'		0.495	0.495
$s_u^A{}_{ref}$	kN/m^2	15	56
$s_u^{C,TX}/s_u^A$		0.99	0.99
y_{ref}	m	13.0	−13.0
$s_u^A{}_{inc}$	kN/m^2/m	1.7	2.14
s_u^P/s_u^A		0.58	0.58
τ_0/s_u^A		0.5	0.5
s_u^{DSS}/s_u^A		0.65	0.65

Fig. 8. Pictures of the construction pit (source: Trafikverket).

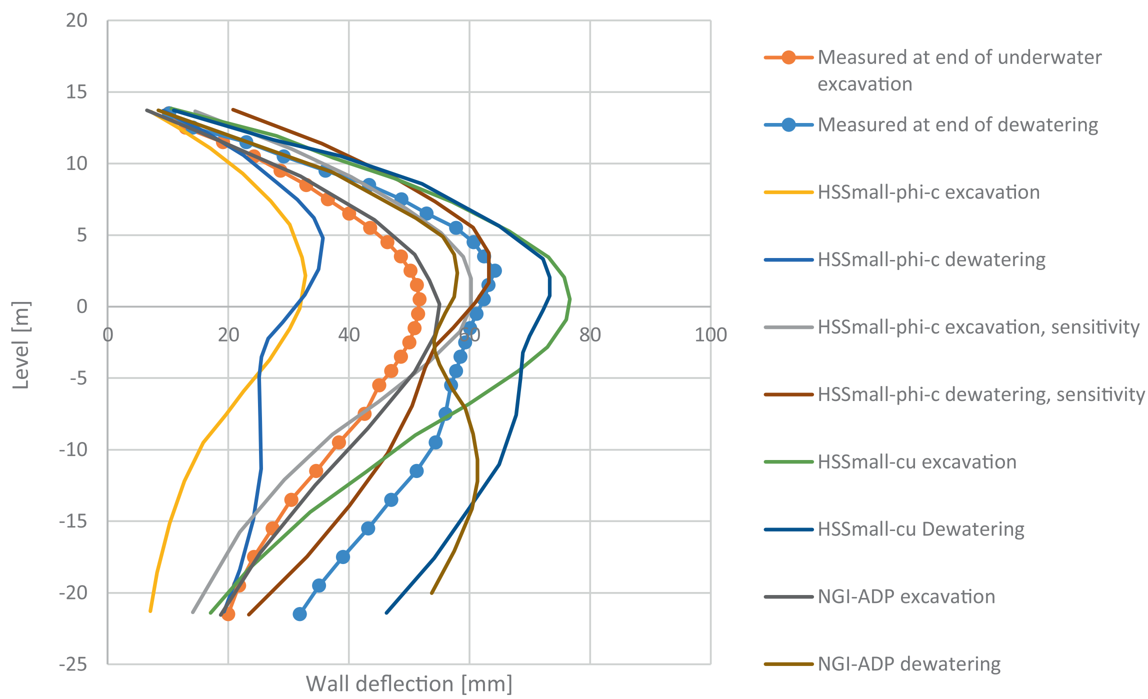

Fig. 9. Comparison between measured and predicted wall deflections (design section B-B, low water at + 9.6 m).

solution of casting the tunnel elements afloat was abandoned. It was assessed that establishing mooring areas for the tunnel elements during casting and outfitting would require extensive dredging. Based on recent experiences from the Söderströmstunnel in Stockholm it was also considered a disadvantage to cast the tunnel elements afloat. For the Söderströmstunnel the steel body of the elements were assembled elsewhere, then cast afloat on lake Mälaren and near the immersion site. One of the main challenges faced was the trim of the elements, which changed as concrete was being poured.

In order to meet the required construction period, an immersed tunnel with only three elements, each 102 m long, was chosen. This shortened the total IMT length to 306 m compared to the 408 me in the recommended solution. As a consequence, a deep construction pit partially extending into the Marieholm marina had to be constructed.

7. Construction pit (dry dock) in soft clay

7.1. General considerations

As explained above, it was chosen to cast the immersed tunnel elements in a deep construction pit on the eastern shore of the Götaälv, which afterwards became part of the construction pit for the Cut & Cover tunnel.

A dock with dry excavation was disregarded, as it would require several support levels obstructing the casting of the tunnel elements and Cut & Cover tunnel. Hence, underwater construction was considered as a better option. The dock had to be opened first in the western end in order to float out the elements, and finally in the eastern end in order to connect the Cut & Cover tunnel with the ramps. The associated demolition of the walls would have been more inconvenient with diaphragm walls than with steel walls. Furthermore, diaphragm walls were assessed to be more expensive. Therefore, the chosen solution was a dock with steel pipe walls, a strut level at the top and underwater excavation.

7.2. Chosen solution

The chosen solution was a 115 m long and between 35 and 52 m wide construction pit with an excavation depth between 16.5 and 18 m (see Fig. 6).

The retaining walls were made of 39 m long Ø1575/17 mm steel

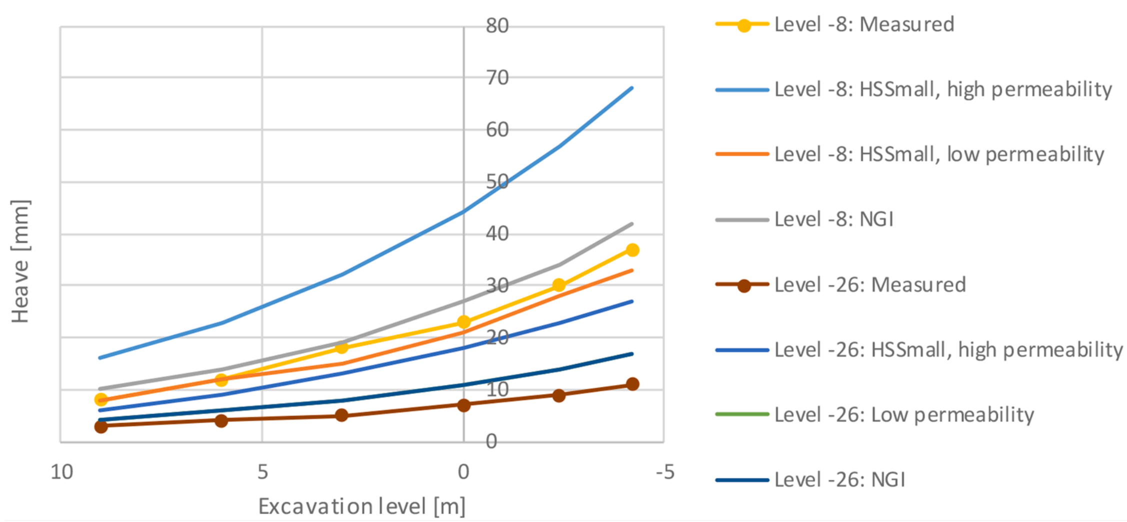

Fig. 10. Comparison between measured and calculated heave of the soil during underwater excavation.

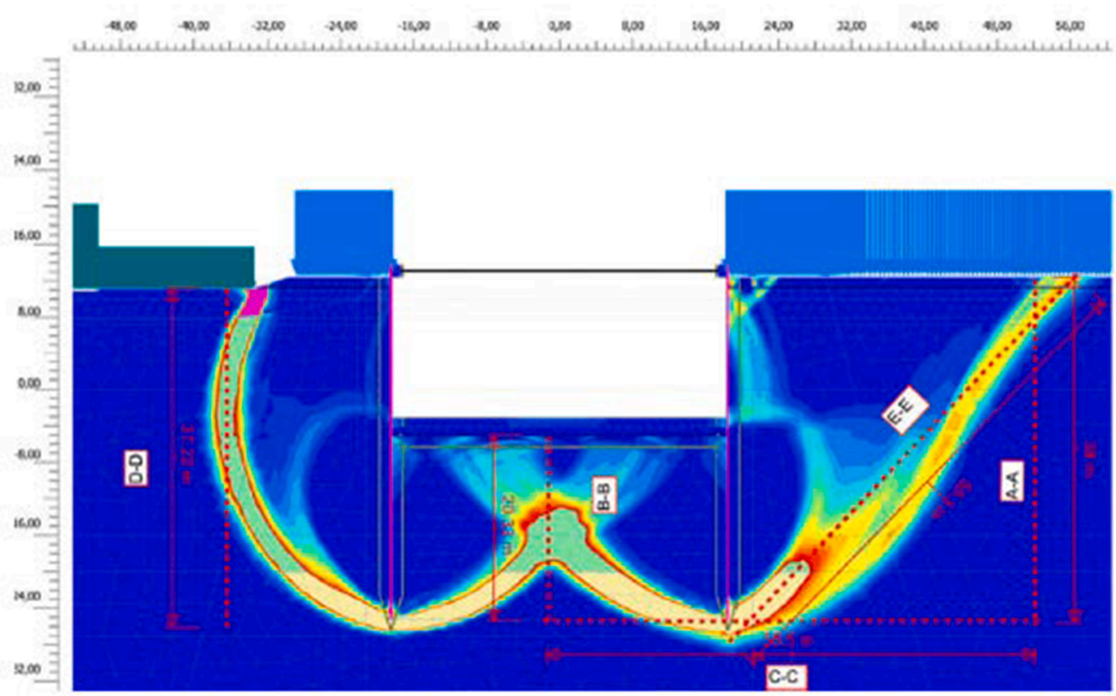

Fig. 11. Illustration of failure mechanism from a strength reduction analysis.

pipes with interlocks at 1775 mm c-c spacing in the western, deeper part of the dock and 35 m long ø1420/16 mm steel pipes with interlocks at 1820 mm cc-spacing in the rest of the dock. The steel grade was S235 JR.

The retaining walls were supported at the top by steel struts with a c-c spacing of 9 m against a reinforced concrete bracing system / waling beams (Figs. 4 and 6).

After installation of the walls, struts and waling beams (finished 22 August 2015), the dock was excavated in several steps under water with an overpressure of 2 m. After the final excavation level was reached (31 October 2015), first a 600 mm thick drainage layer was placed, then a 1.0 m thick mainly unreinforced underwater slab was cast (finished 14 December 2015) and ballasted with a 1.0 m thick gravel layer (finished 08 January 2016). Afterwards, the construction pit was pumped dry (finished 24 January 2016). Drainage pipes through the slab into the drainage layer acted as bleeder wells avoiding water pressure on the slab. A 770 mm thick reinforced concrete slab with shear dowels to the retaining walls was cast in strips of maximum 5 m width between the north and south wall onto the unreinforced slab replacing the gravel layer (finished 30 March 2016) to strengthen the unreinforced slab against heave. Ballast gravel of minimum 15 m width was left in between the first strips. The unreinforced underwater concrete slab was designed as horizontal support for the wall for the short term, allowing for limited upwards movements due to relaxation of the soil. Major vertical forces developing with proceeding consolidation were carried by the reinforced concrete slab. The drainage layer ensured that no phreatic water pressure was acting on the base slab.

After casting of the first tunnel element, a second wall (steel bulkhead) was made inside the dock just next to the western wall. The second wall was made in a way that it could easily be placed and removed and that it provided a watertight connection with the bottom slab and the walls. Afterwards, the western wall was demolished, the bulkhead was opened, and the element was floated out. Finally, the bulkhead was

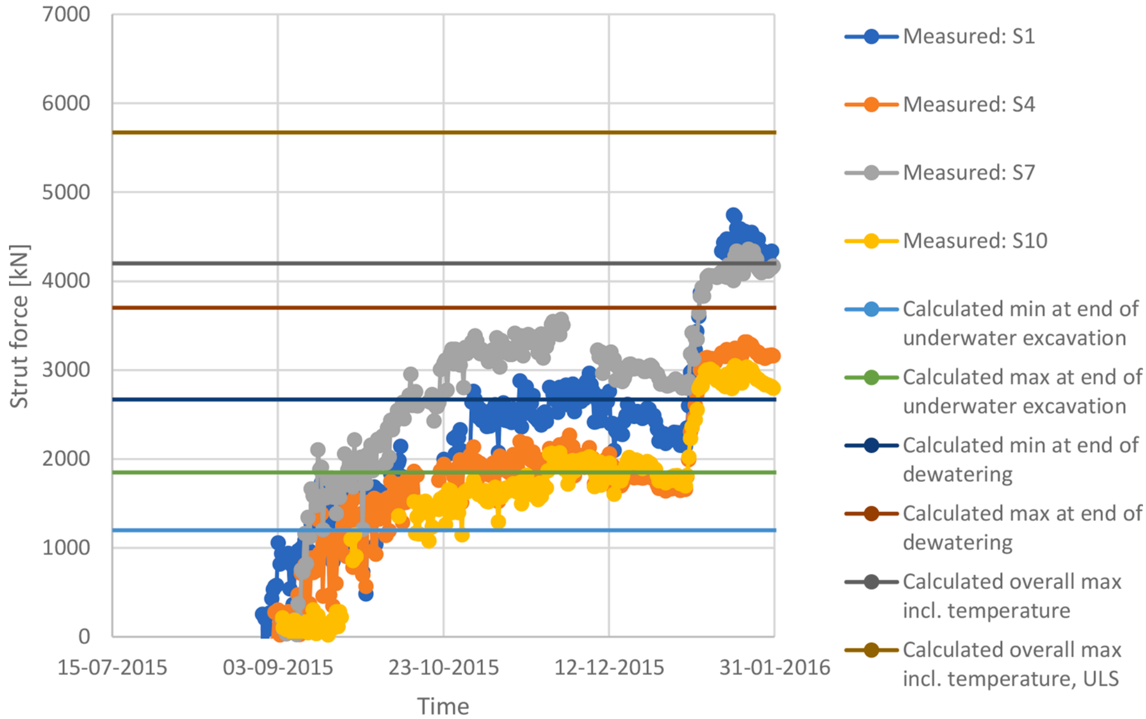

Fig. 12. Comparison between measured and calculated min (low water at + 9.6 m) and max (high water at + 12.3 m) strut forces. The struts are designed for "Calculated overall max including temperature, ULS".

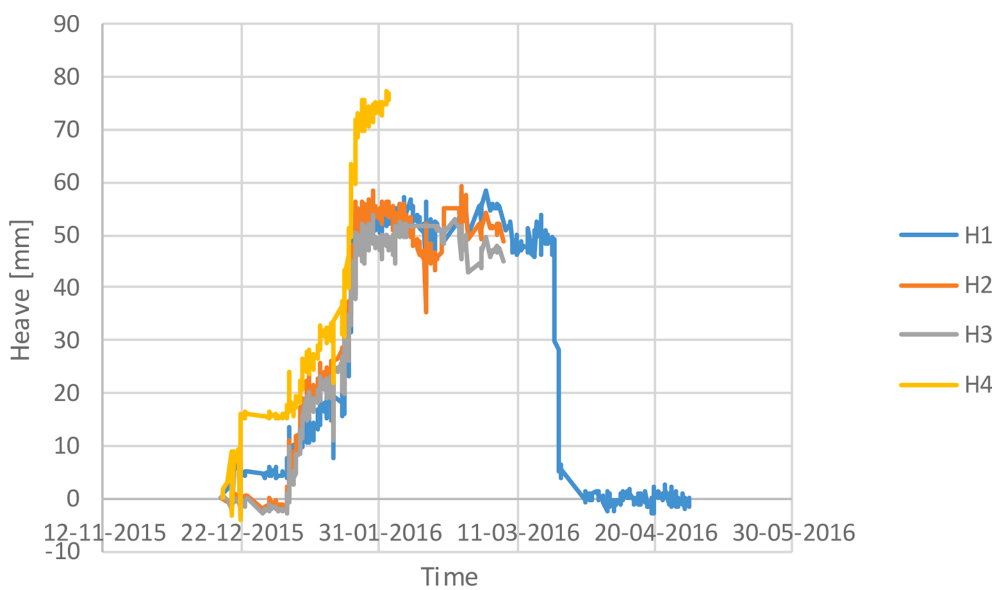

Fig. 13. Measured heave of the underwater slab.

closed and the dock was dewatered again in order to prepare for casting of the next element. The ramp construction works ran in parallel as a separate pit with the eastern wall of the dry dock shared with the ramp pit. When the permanent structures had been built and partially backfilled, then the strutting system was demounted and the steel pipe wall was cut down.

7.3. Design and monitoring

The full construction sequence was analysed with 2D finite element models in Plaxis for design sections A-A and B-B according to Fig. 7. Material modelling for soft clays in Scandinavian countries has been subject of detailed research (Karstunen and Amavasei, 2017). The design was based on consolidation analyses using the Hardening Soil model with small-strain stiffness (Bentley PLAXIS) and effective strength parameters ("HSSmall-phi-c" in Fig. 9), with the parameters based on the ground investigations on the project as shown in Table 1. The following comparative analyses were made:

a) Hardening Soil model with small-strain stiffness (Bentley PLAXIS) and effective strength parameters, with lower friction angle φ' = 22.5 deg and faster small strain stiffness degradation $\gamma_{0.7}$ = 3.0E-4 above level −20 and 2.5E-4 below level −20 ("HSSmall-phi-c sensitivity" in Fig. 9).

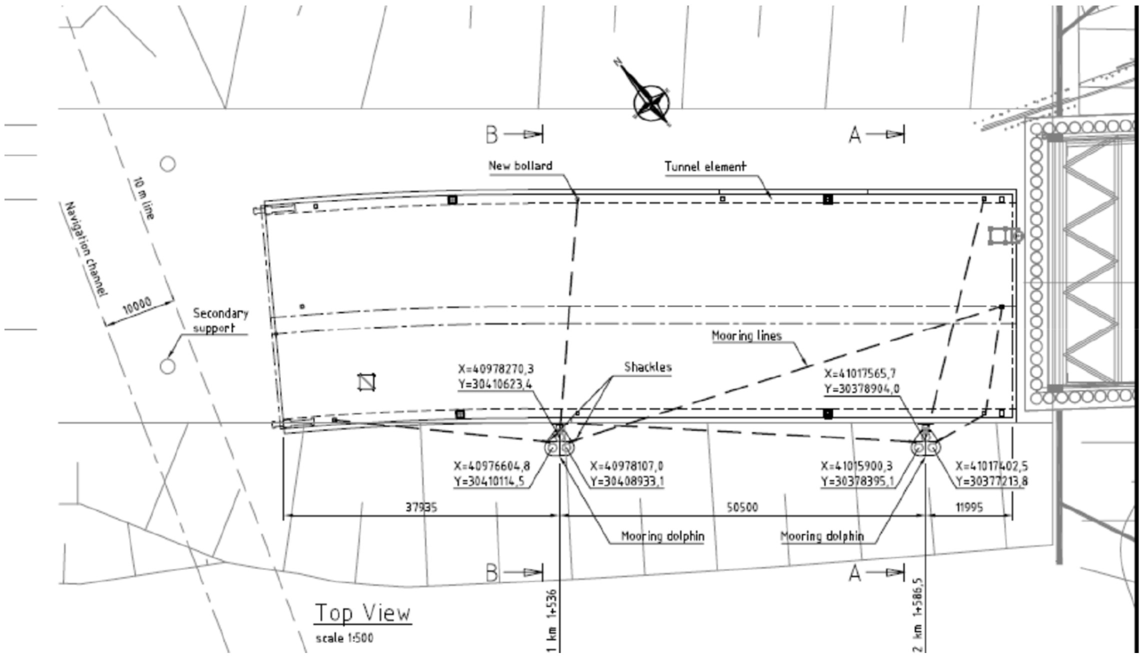

Fig. 14. Moored location of tunnel element 1 outside dry dock.

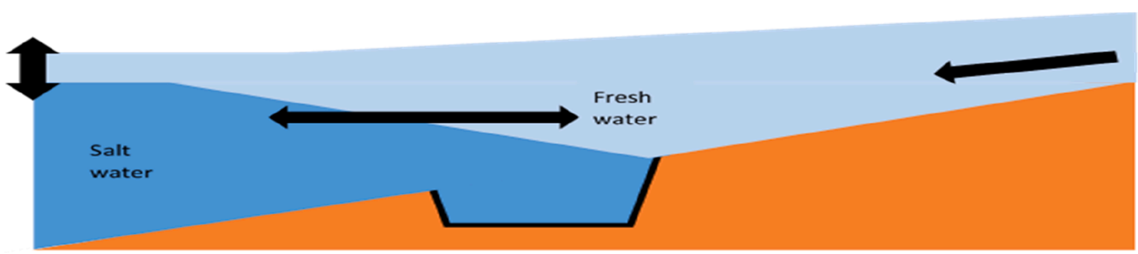

Fig. 15. Sea level (tidal) changes and discharge of fresh water determine position of saltwater wedge.

b) NGI-ADP model (Bentley PLAXIS) with the chosen parameters shown in Table 2 (“NGI-ADP” in Fig. 9). Top of “Clay 2″ at level −20 m.
c) Hardening Soil model with small-strain stiffness (Bentley PLAXIS) and undrained shear strength (“HSSmall-cu” in Fig. 9). All parameters were according to Table 1, except for the undrained shear strength, which was chosen as approximately 70 % of the active undrained shear strength in Table 2.

The interface between the walls and the soil was modelled with an interface strength factor $R_{inter} = 0.67$ (see Fig. 8).

The MI1 to MI6 inclinometer measurements all showed quite similar results. A comparison between the average of all six inclinometer measurements after underwater excavation and after dewatering of the dock with the different predictions is shown in Fig. 9. It can be observed that the “HSSmall-phi-c” used as basis for the design provided a reasonable agreement with the measurements. The variation in predictions between the different models and their deviations from the measurements demonstrates the difficulty of providing reliable predictions. MI7 showed larger wall deflections (approximately factor 2) compared to MI1 to MI6, due to installation of driven piles behind the eastern wall.

Fig. 10 shows that the measured heave of the soil (bottom heave extensometer E in Fig. 6) during underwater excavation was at the low end of or smaller than the predictions. The moderate acceleration of the heave with increasing excavation depth confirms sufficient safety for bottom heave / overall stability (Fig. 11).

Fig. 12 shows a comparison between measured and predicted strut forces. Strut forces were monitored by strain gauges with temperature measurements. The shown measured values include temperature effect. Temperature effects on the struts were minimised by covering / insulating the struts as shown in Fig. 6. The average measured temperature at end of underwater excavation was 7 degrees lower compared to the start of the measurements. The average measured temperature at end of dewatering was 11 degrees lower compared to the start of the measurements. The calculated min and max strut forces are without temperature. It can be observed that the strut forces before dewatering were underpredicted. This indicates that the earth pressure on the wall was larger than predicted. This is in line with the observation that the maximum measured wall deflections were larger than the predicted ones and that an increase of the earth pressure in the sensitivity study in Fig. 4 results in a notable increase in predicted wall deflections. The average of the measured forces after dewatering corresponds approximately to the calculated max (high water at + 12.3 m) at end of dewatering. The strut design was based on the calculated largest strut force throughout the whole construction sequence incl. temperature load (“Calculated overall max incl. temperature”) plus ULS load factor 1.35 (“Calculated overall max incl. temperature, ULS”), i.e. the struts were designed for a force larger than the maximum measured value.

The chosen alarm and limit values for the heave of the underwater C30/37 concrete slab of 55–65 and 60–70 mm, respectively, were based on the normal force eccentricity and shear force checks for unreinforced concrete. Fig. 13 shows that the measured heave remained within the defined limits (except for H4 as an outlier), which confirms that the chosen amount of ballast gravel was sufficient. The figure also shows that the heave was eliminated towards end of March due to the weight of

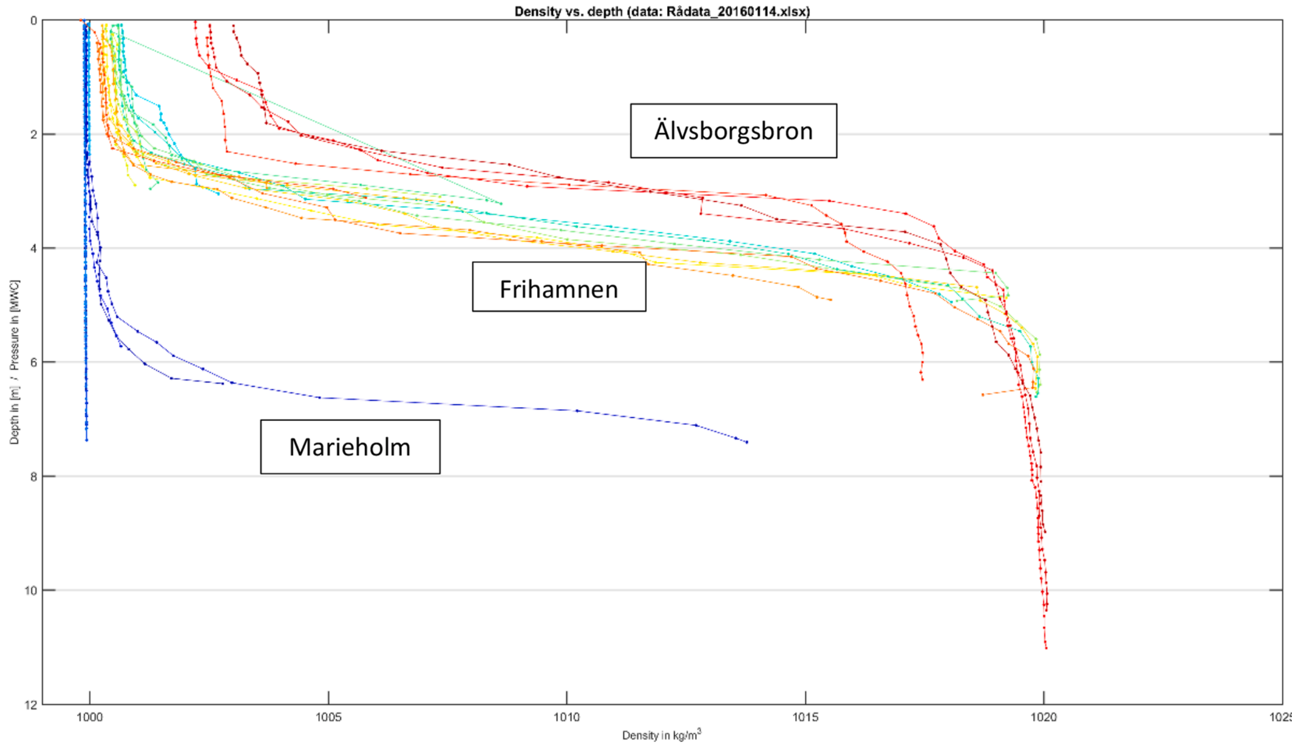

Fig. 16. Measurements of water density changes with depth measured at various moments (samples from Jan 14th 2018 at different times and locations at and downstream from the tunnel.

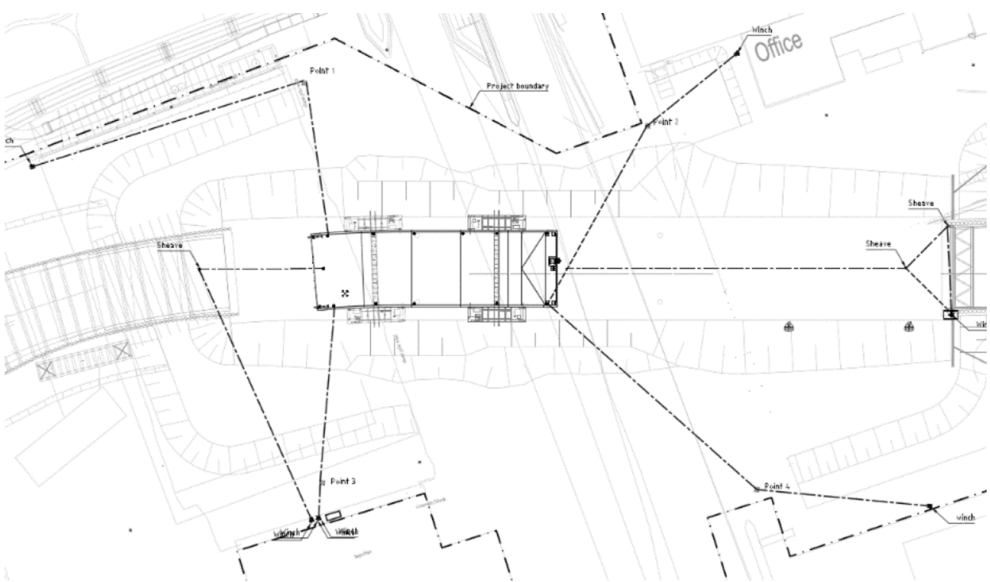

Fig. 17. Warping configuration for pontoon and tunnel element.

the reinforced concrete slab which was cast on top of the underwater slab.

8. Immersion process

8.1. Mooring

The tight construction period necessitated an effective utilization of the construction pit for tunnel element fabrication. Because there was only space for construction of one single tunnel element in the dry dock at a time this had to be floated out as soon as possible after being finished. However, the seasonal restrictions on performing the immersion operations, could result in a situation where the element had to be moored outside the dock until the immersion window opened. The water depth of the Göta Älv was found insufficient to transport a floating tunnel element to a location outside the work area, so a solution needed to be found above the trench and outside the navigation channel. Two temporary mooring dolphins, each consisting of 3 steel piles were placed just outside the dry dock. The tunnel element could then be moored to these.

In the construction time schedule there was some time between the immersion of the second and the third tunnel element, so the first

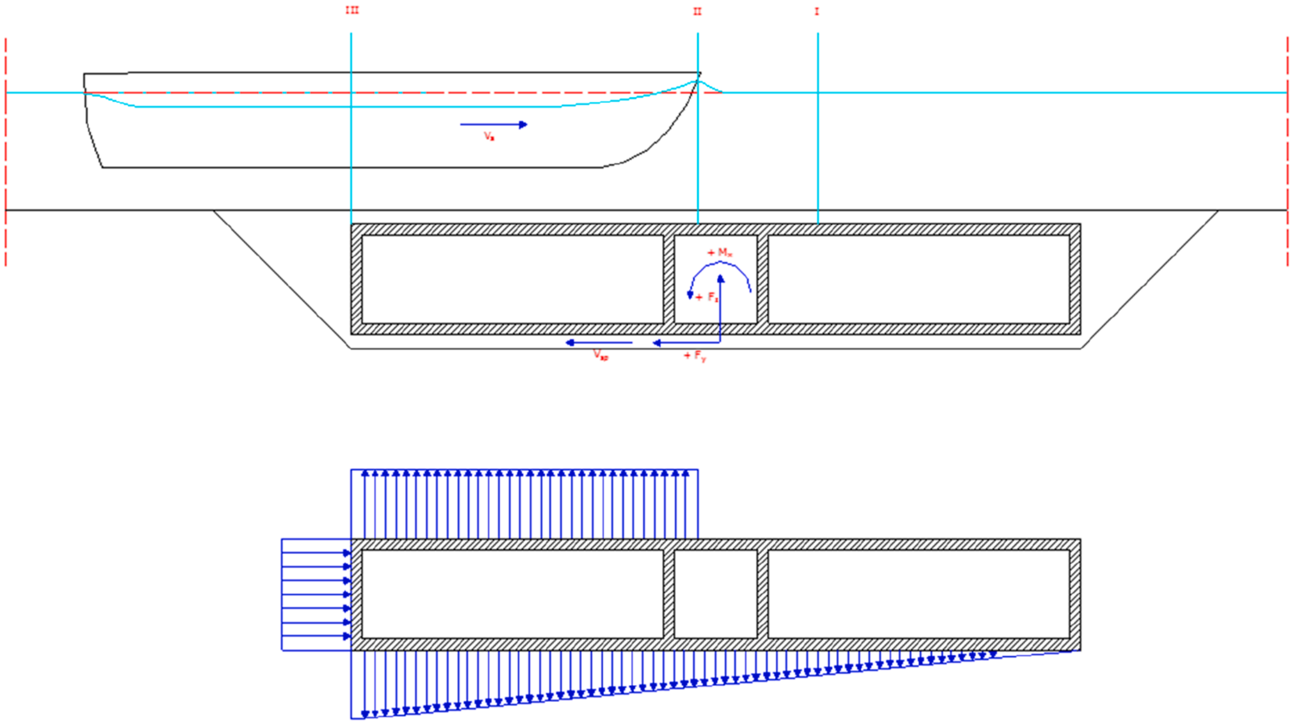

Fig. 18. Effects on tunnel element.

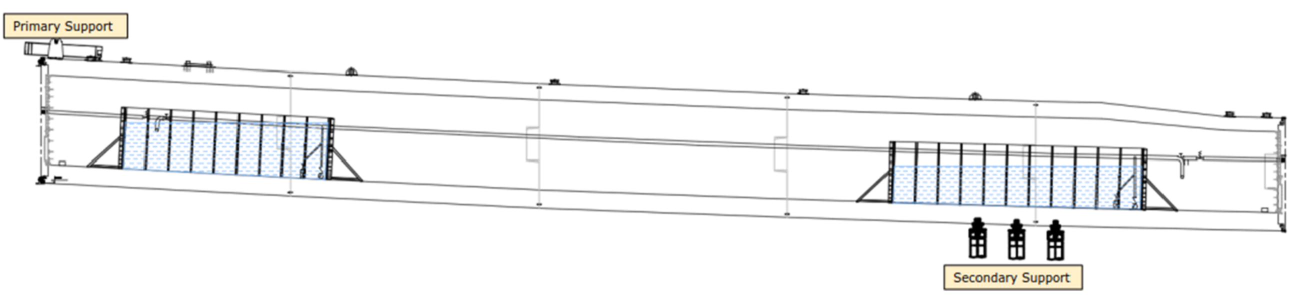

Fig. 19. Temporary support system for tunnel element 1 and 2.

finished element could be moored while the second was cast in the dry dock. After the second element was finished and floating in the inundated dry dock, the first and the second tunnel element could be immersed within a few weeks of each other. The moored location between navigation channel and dry dock is shown in Fig. 14.

8.2. Saltwater wedge

The location of the immersion trench in the Göta Älv is in an area where the salt water from the sea and the fresh water from the river meet. The denser sea water is found in the deeper areas and the lighter fresh water on top. This forms a so called saltwater wedge (see Fig. 15). The location of the saltwater wedge is influenced by the discharge of fresh water upstream and the (tidal) sea water level downstream.

Because the variation in water density is a very important input parameter in the design of the immersion system, a monitoring system was installed soon after the start of the project. A sample of the measurements is shown in Fig. 16.

These measurements combined with earlier measurements showed that the variation in salinity over the depth was more than expected and that the changes in salinity at a certain depth, especially in the immersion trench, occurred faster than expected.

This, when combined with loads from passing ships, resulted in relatively large design loads on the temporary support system as discussed in the next section.

8.3. Closure of the Göta älv

The maximum closure of the Göta Älv for passing ship traffic in order to allow for marine operations was contractually two days.

The Göta Älv is not very busy with marine traffic but with 1900 commercial ships passing per year, closure is not without impact. The largest ships, which needed to be accounted for were coasters of the type Surtemax with dimensions of 125 m × 16 m and a draft of 5.4 m.

Full closure of the river for ships was needed during warping and immersion because of steel cables from tunnel element and immersion pontoon running to winches on shore on both embankments (see Fig. 17).

Once a tunnel element was immersed, ballasted and resting on its temporary supports, ships could be allowed to make use of the river again. Larger ships passing the immersion site could, however, influence the horizontal and vertical stability of a tunnel element resting on its temporary supports as indicated in Fig. 18.

The main force acting on the tunnel element is caused by the primary wave of a ship. This wave causes a water level depression alongside a ship and a return current alongside and underneath it. It was found that

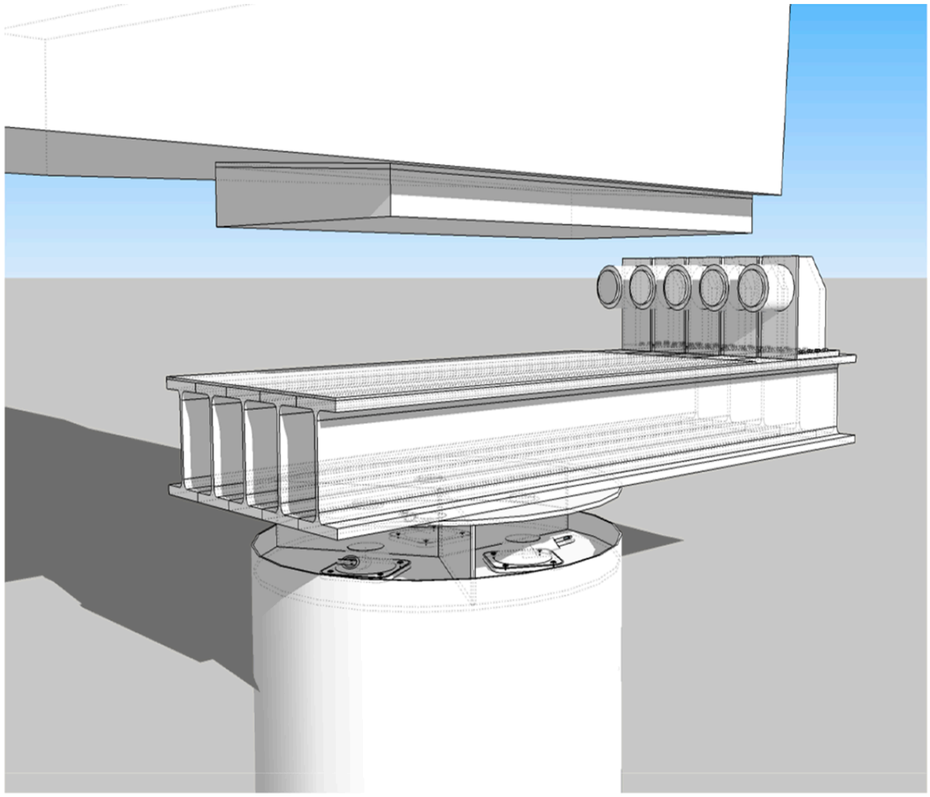

Fig. 20. Illustration of secondary support steel frame.

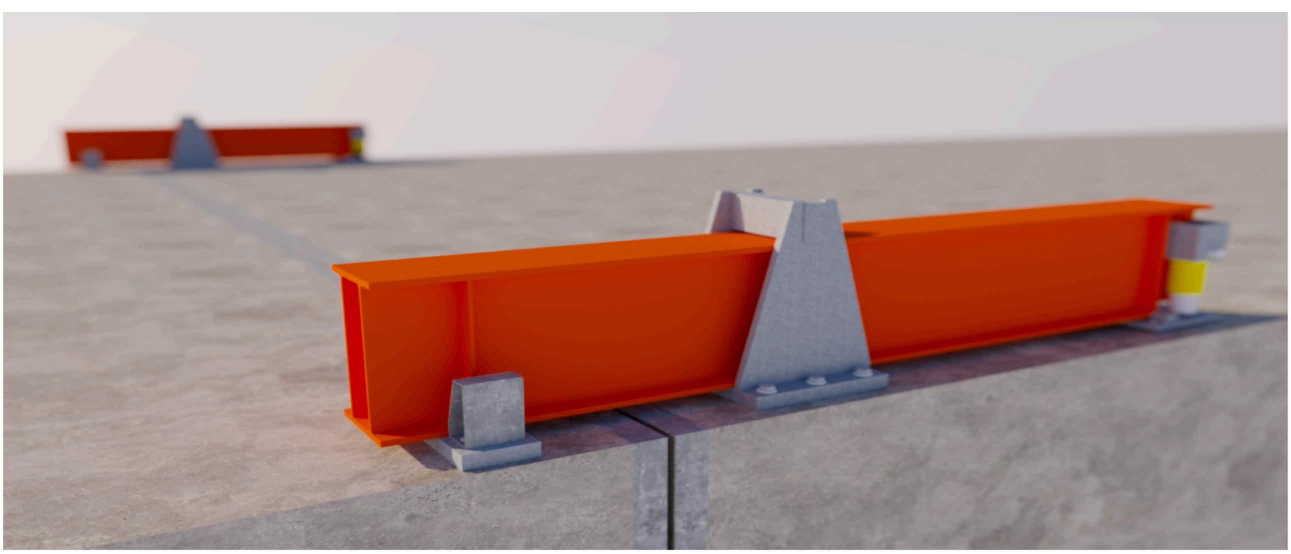

Fig. 21. Illustration of primary support.

especially when the loads from the ships were combined with changing water density and current loads, the amount of ballast needed to provide sufficient stability could exceed the bearing capacity of the temporary supports.

For this reason the closing of the Göta Älv for larger ships was extended from two days to seven days. This allowed for the sandbed underneath the tunnel element and the locking fill to be placed, and thereby ensured that the tunnel element was securely locked in place for horizontal and vertical movement before larger ships passed.

8.4. Temporary supports

After immersion, the tunnel elements were placed on temporary supports at the desired level. These ensured that there was sufficient space between the dredged trench and the underside of the element to be able to execute a good permanent foundation by means of the sand flow method. Because of the presence of the very weak subsoil and large horizontal loads from e.g. ship traffic, it was difficult to apply the traditional support pads for the secondary supports. A solution was found in using 2 × 3 steel piles with a diameter of 1168 mm and 30 m in length for tunnel element 1 and 2. The temporary support system of

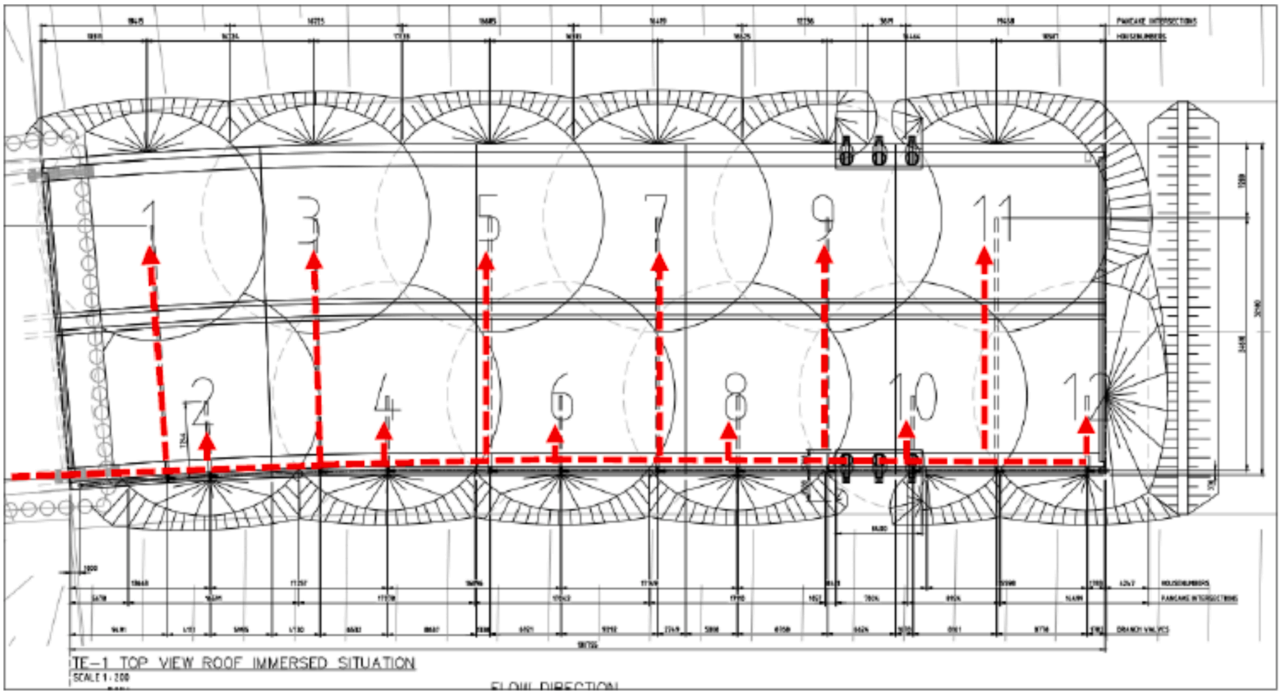

Fig. 22. Locaiton of supply lines and sandflow "pancakes"

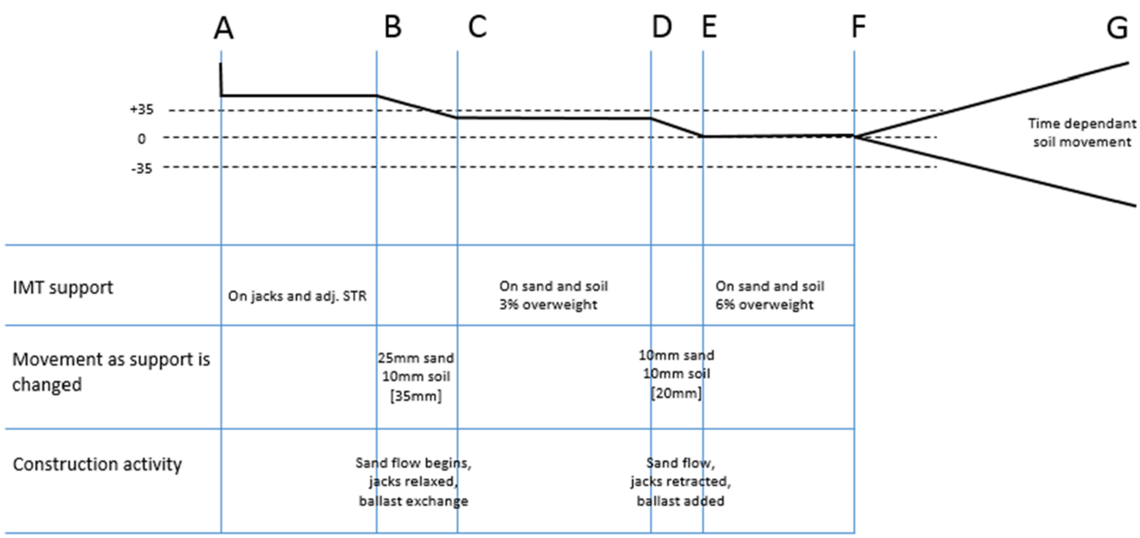

Fig. 23. Loading of tunnel and exchange of loads from temporary to permanent support conditions against initial settlement.

tunnel element 1 and 2 is illustrated in Fig. 19.

For the secondary support of tunnel element 3 a slightly different solution was chosen, utilizing some of the piles in the pipe wall around the dry dock.

The vertical hydraulic support jacks were integrated into the pre-installed steel piles because conduits penetrating slabs and walls of the immersed tunnel were contractually not allowed.

To be able to realign the element horizontally and to temporarily lock the tunnel element in place in horizontal direction, a special secondary support steel frame was developed with integrated horizontal hydraulic jacks. The support steel frame could be removed once the sand bed had been established and tunnel loads had been transferred from the temporary supports to the sand bed foundation, and the locking fill had been placed. By removing the support steel frame a vertical clearance between the steel piles and the floor of the element was obtained, which prevented that a hard point support following from tunnel element settlements. The steel frame is illustrated in Fig. 20.

The loads on the supports varied between the following design limits:

- Double primary support: $50 < P1 + P2 < 800$ ton (ALS)
- Single secondary support: $50 < S1 < 700$ ton (ALS)

The relatively large variation in support loads was due to the presence / absence of the saltwater wedge in combination with the upward loads from the sandflow process. The total ballasted overweight varied between 2.8% and 4.3% of the water displacement.

The primary support consisted of two steel beams (pin) which were installed on the top of the tunnel element. They were supported by a v-shaped construction (catcher) on the preceding tunnel element or on the Tingstad land abutment. Both beams had a hinged connection to the tunnel roof equipped with a hydraulic jack. The two jacks were in hydraulic connection to act as one support. The height of the primary

Fig. 24. Marieholm portal and construction pit (source: Trafikverket).

supports was adjustable and, after sandflow was finished, the support load could be relieved from the beams by retracting the hydraulic cylinders. The primary support is illustrated in Fig. 21.

The temporary supports performed as expected, the support loads from primary and secondary hydraulic systems were monitored and accurate, this was very helpful during the sandflow process. The realignment system at the secondary supports allowed easy and accurate correction of the horizontal position, without compromising the compression of the Gina gasket, which would be the case when a realignment system was used inside the immersion joint.

Disadvantages of this system were the necessity of underwater works for installation by divers, especially the large amount of hydraulic hoses to be connected under water was a challenge. The removal of the steel support frame after the sandflow operation proved to be more difficult than anticipated because of the presence of sand on top of the frame caused by the sandflow process.

8.5. Sandflow process

The permanent sand foundation of the immersed tunnel elements was executed by a "regular" sandflow process. The minimum height of the void between the underside of the tunnel and the dredged trench was 50 cm. The sand flow process started as soon as possible after the immersion of a tunnel element. Sand layers (circular shaped, also called "pancakes") were made under each TE via multiple outlets under a TE in two rows.

A mixture of sand and water was pumped through pipelines which were cast into the concrete floor of the tunnel elements. Underneath the tunnel elements, the sand-water mixture was discharged through outflow openings. The location of the pipelines for the sandflow mixture and "pancake"-pattern is illustrated in Fig. 22.

To ensure a controlled process only one discharge point was opened at a time to discharge the sand-water mixture. The main objective of the sand flow process was to ensure that each tunnel element would be permanently supported by the subsoil (sand) instead of the temporary supports. Calculations for the upward forces due to the sand flow process were based on empirical studies. The diameter of each "pancake" was approx. 22 m and the maximum uplift force from the calculation was 175 ton.

The sandflow for each tunnel element was completed and the element lowered onto the sandbed before the river was opened for navigation. Before the immersion of the next tunnel element the surplus of sand on the secondary side was removed and the secondary supports of the previous element were re-activated to prevent the risk on uneven settlements.

The principle of gradually loading the element and exchanging support from temporary supports to the sandflow foundation is illustrated in Fig. 23. The figure also illustrates how elements had to be placed with a large overheight due to the soft subsoil in order to ensure that they eventually came to rest at the theoretical correct location.

The tunnel elements were immersed between 2017 and 2018.

8.6. Time of engaging shear keys in immersion joints

The soft and load/time dependent behaviour of the tunnel foundation on Gothenburg clay imposed some restrictions on the timing for engaging the shear keys in the immersion joints. All additional loads applied on the tunnel resulted in both an instantaneous settlement, but also a consolidation settlement of the clay. A total vertical installation tolerance of +/-35 mm across an immersion joint was required as indicated in Fig. 23. A particular challenge was that the bandwidth for the actual clay foundation stiffness behaviour was quite wide. To meet the vertical installation tolerance, the load on the tunnel elements was gradually increased by applying backfill and tunnel protection until the relative position of the adjacent element ends was within tolerance. A live monitoring scheme was installed, which continously measured vertical tunnel movements. Consolidation settlements are normally a cause of concern for the in-situ cast concrete shear keys between elements as they can overload the young shear key concrete. For the Marieholm project this was not the case as the concrete shear keys had been cast well in advance while the actual engaging of the shear keys was done by injecting groutbag between one of the shear key bearing faces and a pre-installed neoprene bearing pad on the other bearing face.

8.7. Project status at time of writing

The Marieholm tunnel project was completed and opened for traffic in December 2020.

After finalizing the tunnel elements, the construction pit was used to construct the Marieholm side Cut and Cover tunnel before finally being demolished. The photo in Fig. 24 shows final works on the approach tunnels including the impressive portal structure.

9. Conclusion

The timely completion of the Marieholm tunnel confirms that the solutions chosen by the construction JV were suitable for the project. The employed solutions for construction and immersion works were adapted to an urban environment with challenging constraints imposed by the time schedule, available space, marine operations on the narrow waterway, environmental restrictions and the subsoil conditions. In particular the choice to only excavate one deep dry dock for casting the immersed tunnel elements turned out to right and monitoring during excavation and tunnel construction by and large confirmed the design expectations in relation to behaviour of the retaining structures of the dry dock.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Further reading

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