

# THE PORT OF OSTEND: CONSTRUCTION WORKS FOR THE WIDENING OF THE INNER APPROACH CHANNEL

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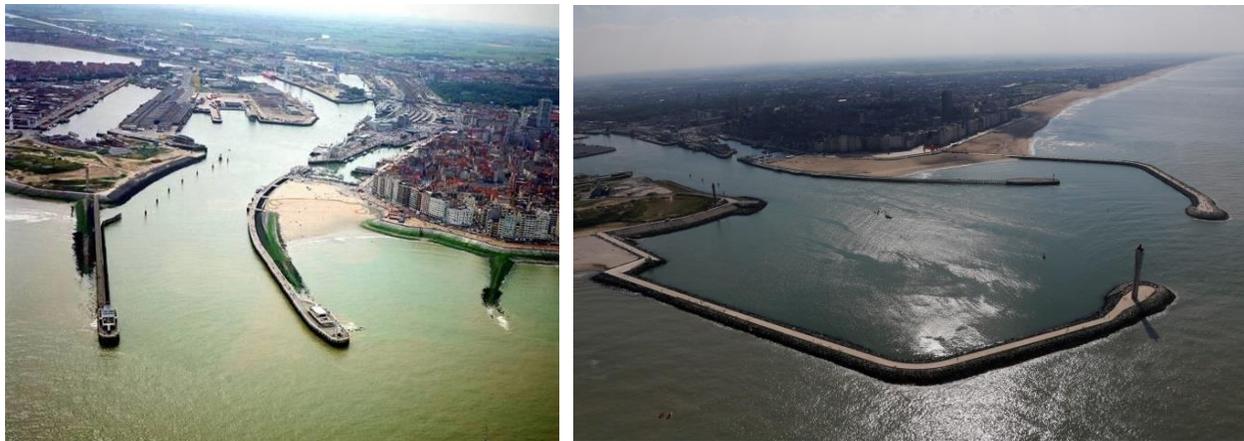
## ABSTRACT

For the last phase of the masterplan for the port of Ostend, the inner approach channel needs to be widened. As the existing timber pier at the western side is protected by law as heritage, widening of the channel is only possible towards the eastern side. To allow this significant widening of the channel, extensive works will be needed at the site 'Halve Maan'. Since the project area is located within the active port, and partially in protected habitat area, a lot of site specific requirements had to be taken into account. All these constraints had a significant influence on the design which resulted in a multidisciplinary project, that can be split in 3 different sub-projects. A new seawall and a new retaining wall were designed, and an existing quay wall had to be stabilized.

## 1. INTRODUCTION

Ostend is situated in the middle of the Belgian coastline. Although for many centuries Ostend was considered one of the most important ports at the Southern North Sea, it has a relatively small harbour. At this moment, the harbour traffic mainly consists of general cargo, but from time to time cruise ships also call at the port. And during 2018 a new ferry connection to Ramsgate (United Kingdom) will be established, which will increase vessel traffic in the port. During the last years, a changeover to 'Blue Growth' has been made by the port of Ostend. This resulted in an extensive growth of the employment related to the construction of new wind farms in the North Sea.

The harbour of Ostend has been changed a lot during the last ten years. In Figure 1 a view on the old harbour (before 2008) versus the harbour at this moment (2018) is shown.



**Figure 1: The harbour of Ostend before 2008 (left) and today (2018), the timber pier at the western side of the harbour is protected by law (monument) (picture © Department of Mobility and Public Works)**

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The construction works at the harbour entrance in Ostend fit into the larger 'Integrated Coastal and Maritime Plan for Ostend' which was set up by the Flemish Community. With the widening of the inner approach channel, this plan will be completed.

## **2. MASTERPLAN FOR THE PORT OF OSTEND**

### **2.1. General description and previous works**

The so-called 'Integrated Coastal and Maritime Plan for Ostend' was set up by the Flemish Community at the end of the previous century. The objectives of this plan were:

- to protect the city of Ostend against flooding: the maximum allowable overtopping discharge for a storm with return period  $R = 1000$  year is restricted to 1 l/s/m;
- to make the harbour of Ostend accessible for larger ships: access for ships with lengths up to 200m is required;
- to approach the harbour expansion and coast protection in an integrated way.

In order to make the harbour accessible for ships with a length up to 200m, important modification works at the harbour access were necessary.

At the outer part of the harbour, the access channel was relocated so that an access more perpendicular to the coast line was created. The existing eastern pier had to be removed to make this possible. Further, two new breakwaters were constructed to realize a more sheltered area for ships when entering the harbour (see Figure 1).

The history and extensive studies for the masterplan of Ostend, and the design of the two breakwaters are described into detail in Verhaeghe et al. (2010).

More inwards the harbour, the approach channel also has to be adapted to allow these larger ships to enter the harbour safely. At present the approach channel at that location is enclosed by a timber pier at the western side, and a seawall with the harbour site 'Halve Maan' at the eastern side (see Figure 2). As the timber pier is protected by law as heritage, widening of the channel is only possible at the eastern side.

The widening of the harbour approach channel inside the port of Ostend concerns the last phase of the works in the framework of the 'Integrated Coastal and Maritime Plan for Ostend'. The design studies for these works are described in this paper.

### **2.2. Last phase of the masterplan – widening of the approach channel**

In 2015 extra navigation simulations were performed with different vessels (cruise-ships, RoRo-ferries and cargo-ships) to optimize the design of the inner channel (Verwilligen et al., 2016). It was concluded that the inner channel should be widened from 80m to 125m at the most northern point of the site 'Halve Maan' to 145m at the most southern point of the site 'Halve Maan'. The proposed design for the new approach channel can be seen in Figure 2, together with an impression of the navigation simulation study. To allow this significant widening of the channel, extensive works will be needed at the site 'Halve Maan'.

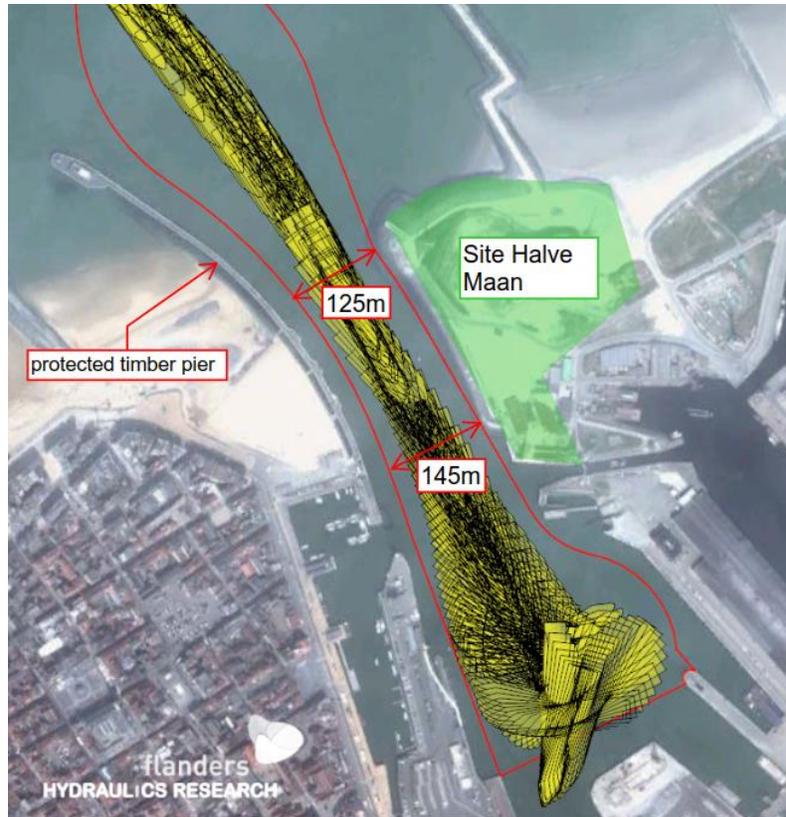


Figure 2: Navigation simulation and proposed design for the location of the widened entrance channel

### 3. SITE CONDITIONS

#### 3.1. Historical evolution of the site and relevance for the works

The project area is situated at the eastern side of the entrance channel towards the port of Ostend, and has known a rich history, depending on the historical needs of the port. The earliest relevant development of the area can be situated approximately 150 years ago. Around 1860 a basin was constructed behind a moon crested dike, together with a new lock 'Leopoldsluis' (Figure 3). This basin had to collect the seawater at high tide and release it through the lock at low tide. The goal was to use the corresponding scour phenomenon as a natural way of maintaining the required drought in the entrance channel. After World War I the Leopold basin was filled up again (Figure 3), but the existing lock remained intact.



Figure 3: 'Halve Maan' site – 1860 with basin and lock, 1950 with filled basin

At the end of the 20<sup>th</sup> century the entrance channel was deepened and widened to allow larger vessels to call at the port of Ostend, leading to the current configuration of the site with a long revetment extending parallel to the entrance channel (Figure 4). During these works, the lock 'Leopoldsluis' was partially demolished and a lot of earthworks were performed to create a new stone revetment. The revetment was repeatedly damaged during heavy storms, and an additional rock berm was constructed in the northern zone. Finally, early 21<sup>st</sup> century, following the latest developments in the offshore wind industry, a heavy-duty quay wall and corresponding pavement area for the assembly of large windmill components was constructed at the south side of the site. This heavy-duty pavement area consists of pile mattress systems and concrete slabs on deep foundation piles, capable of resisting loads up to 10t/m<sup>2</sup>.



Figure 4: 'Halve Maan' site as it is today, with existing dike and revetment parallel to entrance channel

It is clear that the site has undergone many changes during its existence, but the moon crested dike (= 'Halve Maan' in Dutch) has always remained in place and has given its name to this area situated at the east side of the entrance channel of the port of Ostend.

Due to the radical impact of the works which are foreseen, the historical development of the site is considered quite important. Indeed, as will be explained further in this paper, a lot of design constraints (geotechnical characterisation, retaining wall solutions, environmental requirements, ...) can be explained by the rich history of the project area.

### 3.2. Geotechnical conditions

An extensive soil investigation at the site 'Halve Maan' was performed, principally consisting of in situ cone penetration testing (CPT). The CPT's were mainly realized on land, but several nearshore CPT's were also performed at the location of the existing revetment (using a jack-up platform). Additionally, multiple boreholes and corresponding laboratory testing was performed. During the works, it is also foreseen to dig some exploratory trenches during the works to detect the remainders of the old lock 'Leopoldsluis'.

In general, distinction can be made between two principal geotechnical zones (Figure 5), with the old lock as a distinct transition point. The northern area can be considered as having favorable soil conditions over the entire depth, mainly consisting of medium dense up to very dense sands. A second southern area has less favorable soil conditions, with soft deposits up to a large depth. These soft deposits can be characterized as soft clay, even with an organic content at some locations. Geotechnical properties of the soil below the soft deposit correspond well to the northern geotechnical area. In general, soil characteristics were determined using Eurocode 7 (CEN, 2014), combined with the results of the laboratory testing (where applicable).

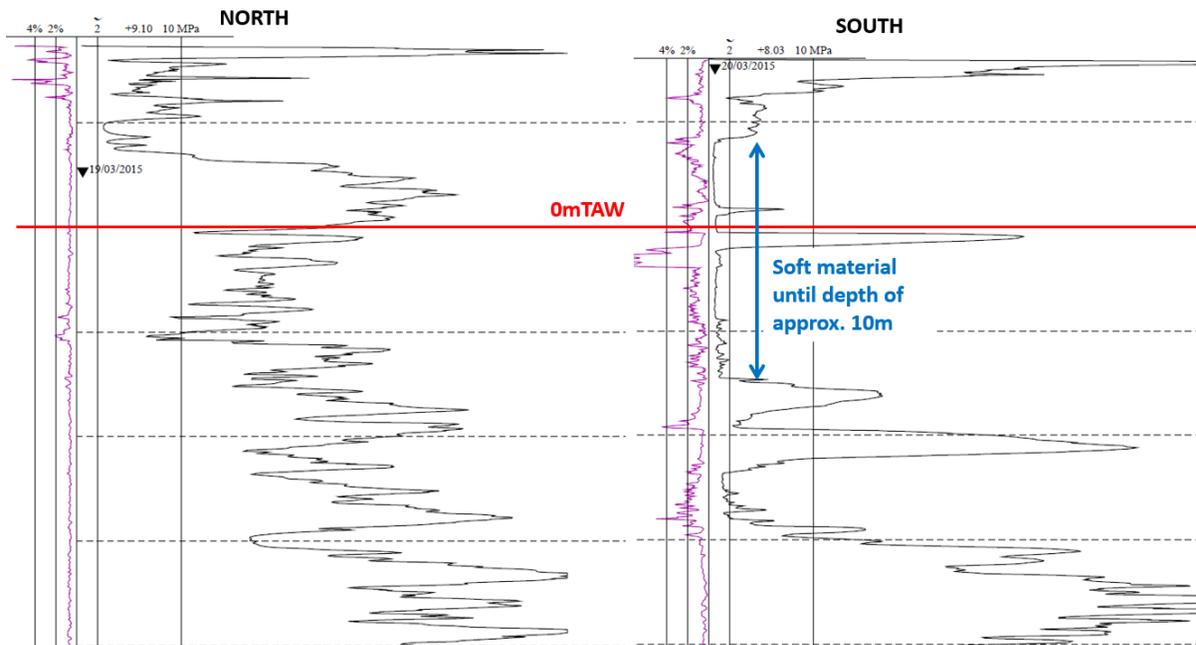


Figure 5: Typical geotechnical conditions at the 'Halve Maan' site

Knowing the history of the site, the sharp geotechnical transition between northern and southern area is not very surprising. Indeed, as mentioned in paragraph 3.1, the area south of the old lock used to be a basin and was backfilled shortly after World War 1. Most likely a backfill material of low quality was used, which explains the sudden change of sandy soils towards soft deposits when crossing the location of the old lock.

### 3.3. Metocean data

The design wave conditions at the location of the new construction were determined for a return period of 100 years. Numerical simulations were performed using Mike 21 BW with short-crested waves for different wave directions. In Figure 6 a contour plot is given, showing the calculated significant wave heights inside the harbour of Ostend for a storm with return period = 100 years with waves coming from north-west.

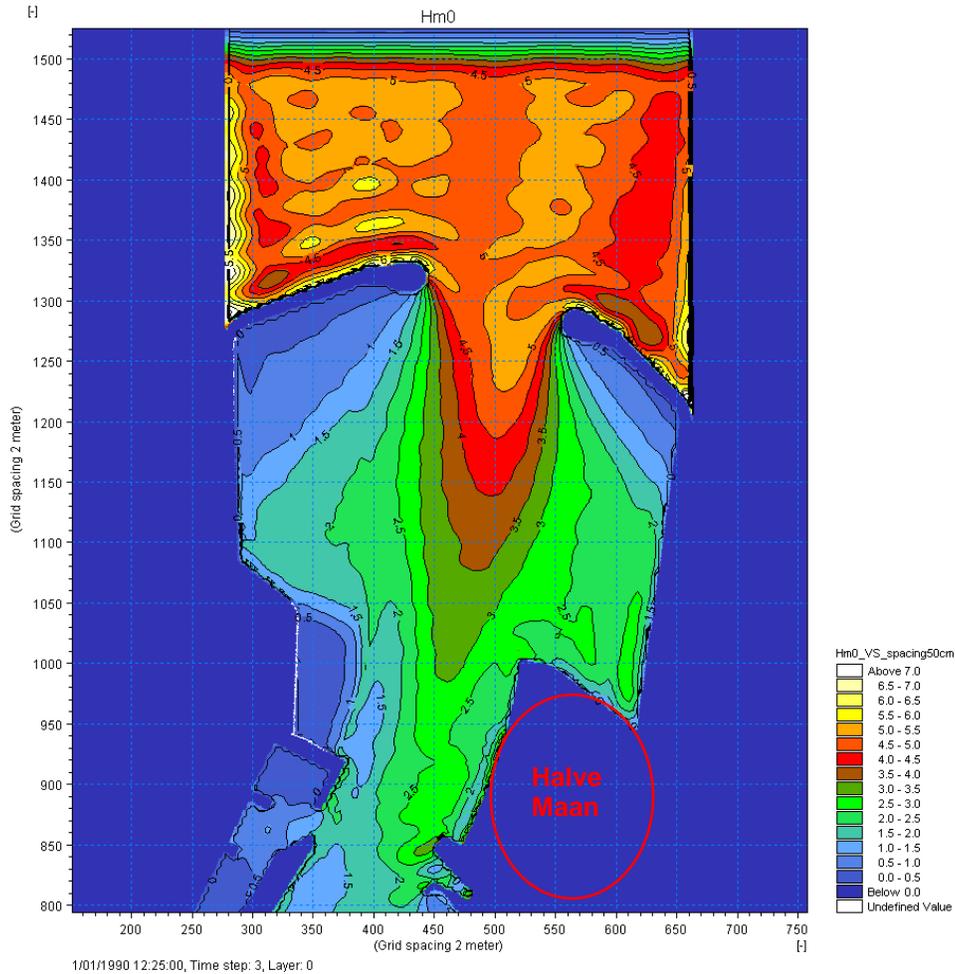


Figure 6: Contour plot of  $H_{m0}$ , return period = 100 years, waves from NW

The wave heights obtained at some distance before the 'Halve Maan' were used for the design of the new revetment. The corner of the 'Halve Maan' is exposed to the largest wave heights: design waves with wave height  $H_{m0} = 4\text{m}$  and peak period  $T_p = 10,85\text{s}$  are obtained. The corresponding 100-year water level is +6,7m TAW.

### 3.4. Archeological investigation

An archaeological study showed that the site has no particular historical value and up to today no special measures are foreseen. However, there is a small risk of unexploded ordnance (UXO) which still pose a risk of detonation. During construction, this will be a point of attention, certainly because of the many excavations that are foreseen for the works.

### 3.5. Environmental constraints

The site was assessed thoroughly from an environmental point of view and it was concluded that the environmental constraints were of utmost importance.

Study of the groundwater levels and the local topography (combined with the historical evolution of the site as described in paragraph 3.1) led to the definition of two potentially interesting areas for recovery of the original vegetation (Figure 7). First, a northern zone was identified which could be interesting for the development of brackish grassland. Secondly, contrary to the saline northern area, a southern zone with sweet water conditions was detected. This zone was evaluated to be interesting for the development of wet dune vegetation.

Based on these two observations, it was decided to perform two excavations to restore part of the project area into its original vegetation (total volume of earth to be excavated approx. 21.000m<sup>3</sup>). The development of the expected salt marsh vegetation requires a regular flooding (typically every spring tide), which is obviously not the intent of the works that will be performed. Therefore, an artificial siltation of the area will be required.



**Figure 7: Zones with a potential for recovery of original vegetation (blue = brackish conditions, orange = sweet water conditions)**

Moreover, a large part of the area is located in a so-called habitat area (dune area), which is protected by Belgian law. Due to the widening of the entrance channel, part of this habitat area will be excavated and cannot be recovered. This loss of habitat area was compensated by restoring another similar area in its original conditions. This compensating area was defined during the master planning phase and covers a surface of 0.95 hectares. This strict limitation on the seizure of protected habitat area imposed an important constraint for the design of the widening of the entrance channel.

### 3.6. Conclusion

It is clear that the works cover a range of challenges of building into an existing harbour with limited space and many boundary conditions. The interference with existing structures, the poor geotechnical conditions, the close presence of exploitation zones etc. all had a significant influence on the final design.

## 4. WIDENING OF THE APPROACH CHANNEL

### 4.1. Definition of project zones

To allow larger vessels to enter the port safely, the entrance channel should be widened from 80m to 125m at the most northern point of the site 'Halve Maan' to 145m at the most southern point of the site 'Halve Maan' (Figure 2), meaning that the entrance channel becomes wider when going deeper into the port. To allow for this widening, a new shoreline needs to be developed as well as a connection to the dike 'Halve Maan' in the north and the existing quay wall in the south.

Due to the interference between nature, infrastructure and existing exploitation zones, the works were split into three main project areas and a site-specific design was developed for each of these zones. Following project zones were defined (see Figure 8):

- Northern area: apart from the limitation on the seizure of habitat area, the edge of the approach channel could be shifted quite easily in this area. Due to the good soil conditions, a new revetment proved to be the preferable design solution. The existing dike is also reinforced to resist the high wave loads and reduce overtopping.
- Middle area south of the existing lock: this zone is characterized by unfavorable soil conditions, meaning that the construction of a new slope and revetment was not so obvious (a very gentle slope would be required, leading to very high volumes of soil to be excavated). Moreover, the existing pile foundations of the heavy-duty pavement for the offshore industry would interfere with this new slope. Therefore, it was decided to develop a retaining wall solution for this area, with the advantage that the existing heavy-duty pavement will still be functional after the works.
- Southern area: at the most southern point, the new entrance channel comes very close to the existing quay wall at the corner of the 'Halve Maan' site. This quay wall was originally designed for a water depth of 6.6m, while the new approach channel goes up to a depth almost 9m. Due to the close distance of the new entrance channel to the quay wall, measures had to be taken to stabilize this existing quay wall and allow for this increase in water depth.



Figure 8: Definition of project zones and indication of old/new approach channel

## 4.2. Northern area – construction of new revetment (?)

### 4.2.1. Geotechnical design

As explained earlier, for the northern zone there is a strict geometrical constraint related to the present habitat directive area. Therefore, instead of simply shifting landwards the existing seawall, a structure with a steeper slope was designed. Due to the favourable soil conditions, the geotechnical design of the slope was quite straightforward and a rather steep slope of 10H/4V covered with a revetment was designed, followed by a protected berm and a more gentle unprotected slope (see typical cross section in Figure 9).

The unprotected slope is somewhat steeper than what normally would be designed for an exposed slope in active water. However, experience of the port authorities in the neighbouring port of Zeebrugge indicates that no major erosion issues are expected, since a stone covered willow mattress is foreseen on the berm, extending to a level including the active zone during wave action (1.5 times the wave height below the sea level). Nevertheless, an inspection programme has been developed to monitor the actual state of the slope during the lifetime of the structure. If necessary, an additional gravel layer can be provided to stabilize the unprotected slope in case of unacceptable scour.

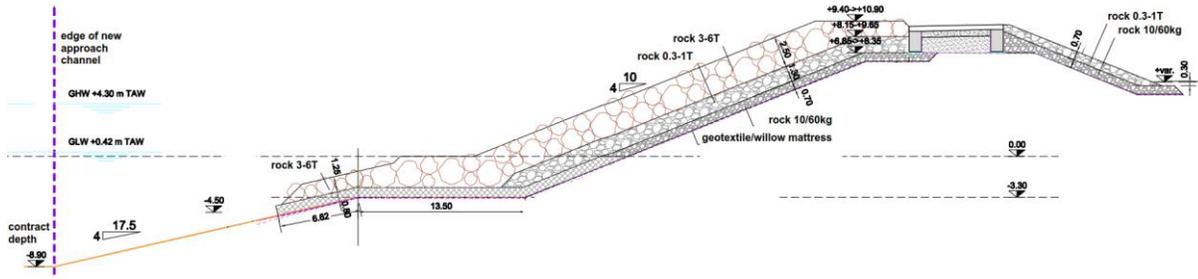


Figure 9: Typical revetment cross section (rock armour layer)

#### 4.2.2. Hydraulic design – armour stability

For the hydraulic design, distinction can be made between the seaward directed zone and the roundhead exposed to perpendicular wave attack with high waves, and the zone parallel to the entrance channel where wave heights are slightly smaller and wave attack is almost parallel to the revetment. This lead to two different designs for the armour layer, as shown in Figure 10.

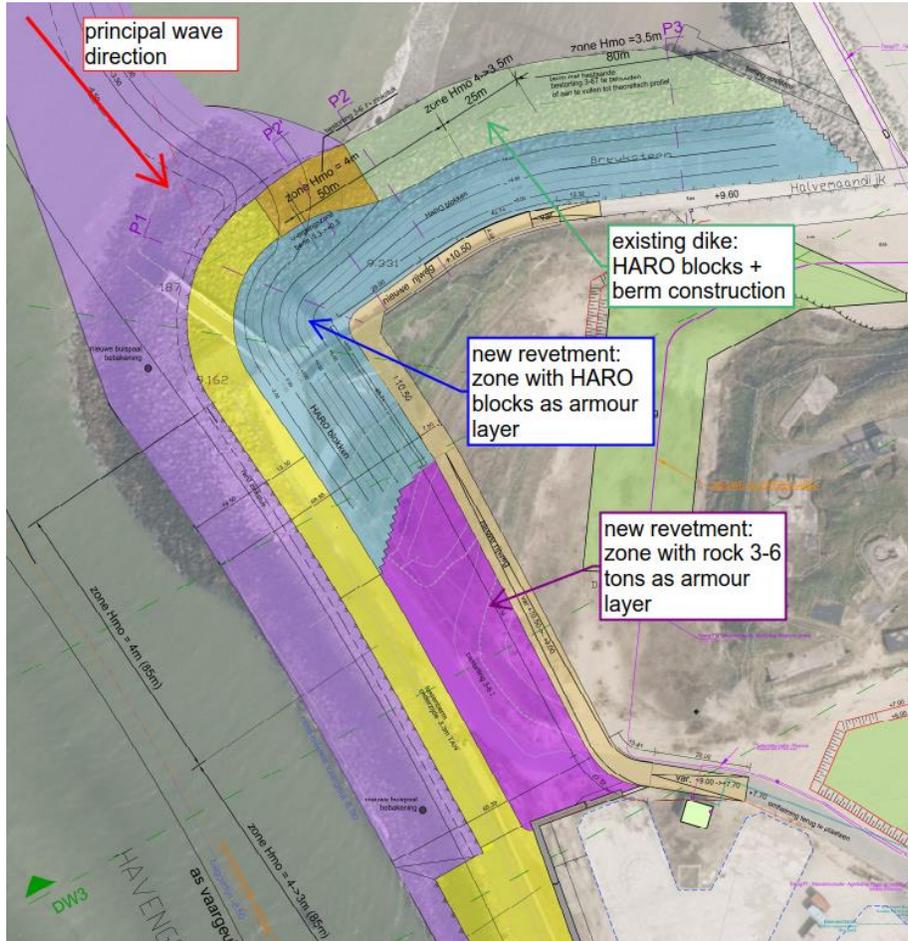


Figure 10: Design for northern project area (revetments and protection of existing dike)

The northern zone is oriented towards the harbour entrance and hence exposed to significant hydraulic loading. Due to the orientation of the two breakwaters protecting the port entrance, a quasi-perpendicular wave attack can be expected. In this area, a protection using one layer of concrete HARO blocks (weight 15 tons) was designed to resist significant wave heights up to 4m (typical cross section is shown in Figure 11). For hydraulic stability the common rules in the rock manual (CIRIA, 2007) were used. As a conservative approach a stability coefficient equal to  $K_D = 4$  was defined for the design of one layer of HARO blocks.

For the underlying second armour layer, the normal filter criteria would require a layer with 1-3 tons grading (W/10, with W the weight of the HARO blocks). However, practice has shown that these large stones can lead to issues when positioning the prefabricated HARO blocks. Therefore, as an innovative approach, it was decided to apply a smaller grading to allow for an easier construction process. Research at Gent University (D'Hont and De Kimpe, 2013) demonstrated that for a single layer of HARO blocks an underlayer with an average weight of W/20 (with W the weight of the HARO blocks) can be applied, leading to a practical grading of 0.3-1 tons armour stone.

The HARO blocks are placed on the to be excavated slope where the entrance channel is widened, and on the roundhead that will be built to make the transition towards the existing dike 'Halve Maan'. To reinforce this existing dike (consisting of smooth granite blocks), it is also foreseen to add a cover of HARO blocks (see plan view in Figure 10).

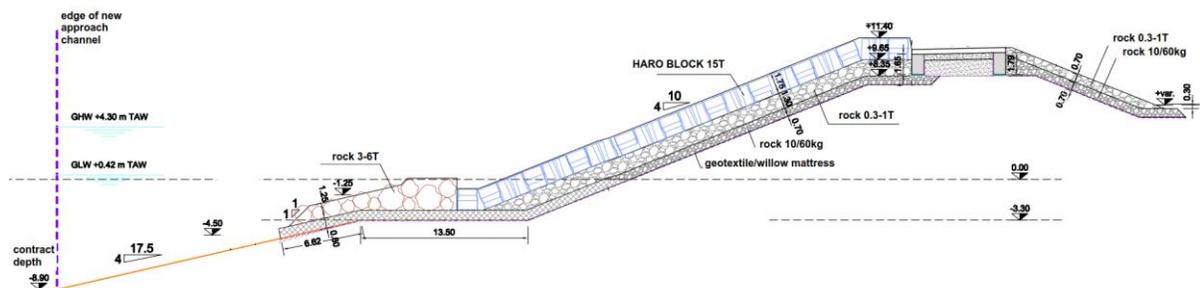


Figure 11: Typical revetment cross section (HARO armour layer)

The zone parallel to the entrance channel is experiencing somewhat smaller wave heights when going into the port. But from hydraulic perspective, the major difference compared to the northern area is the fact that the obliqueness of the waves differs significantly. Wave direction is almost parallel to the revetment (approach angle is  $80^\circ$  measured perpendicularly on the revetment), meaning that hydraulic loading of the revetment is considerably reduced. Because of this reduction a double armour layer consisting of 3-6 tons proved to be sufficient (cross section shown previously in Figure 9).

#### 4.2.3. Hydraulic design – overtopping

The height of the seawall is limited to achieve an economically (less earthworks) and environmentally (minimize use of habitat area) acceptable design. Obviously, this results in significant overtopping in extreme conditions and as a practical design criterium, it was decided to limit the overtopping discharges to 50l/m/s for 100-year metocean conditions. This high value was considered acceptable since no buildings or occupied space are located in this zone (only habitat area) and the majority of the discharge naturally returns to the approach channel. Overtopping was calculated according to the latest EurOtop manual (Van der Meer et al., 2016), which lead to the definition of sea wall height (in general, the existing terrain level needs to be increased with 1m).

Due to the considerable overtopping, the rear side of the seawall is also exposed to significant hydraulic loading. This has to be verified, since a progressive erosion at the backside of the seawall would lead to overall failure of the seawall. Based on the calculated flow velocity, the required stone size to be hydraulically stable in the overflowing water was determined, which resulted in an additional armour layer 0.3-1 ton at the backside of the seawall.

At the existing dike 'Halve Maan' it was not possible to simply increase the height of this existing seawall to limit the overtopping values. Luckily, current sea bed level in front of the dike is limited and a berm consisting of 3-6 tons armour stone is already in place, which has a beneficial effect on the overtopping discharge. Moreover, the HARO blocks that will be placed on the dike lead to an additional reduction in overtopping. Calculation showed that the combination of berm and HARO revetment is sufficient to limit the overtopping to the acceptable values, and it is only foreseen to reinforce the berm to a minimum level and width (corresponding to the geometry that was considered in the design).

#### 4.3. Middle area – construction of new quay wall

The middle project area is located near an existing exploitation zone used by the offshore industry. Intended as a handling area for heavy windmill components, pavement of a large part of this exploitation zone was designed for a uniform load of 10t/m<sup>2</sup> using deep foundation techniques. Two foundation systems were applied to allow for this high surcharge load: reinforced concrete slabs on deep foundation piles, and flexible pile mattress systems on short concrete piles. Moreover, geotechnical conditions for this area are quite challenging with soft soils (clay, peat) up to depths of 15m. Based on the above, shifting the existing seawall landwards was considered not expedient and as an alternative a quay wall was designed. The quay wall consists of a combi-wall (tubular piles as primary unit and intermediate double AZ sheet piling) with inclined ground anchors (pre-stressed grouted anchors) and a concrete capping beam, the typical cross section can be seen in Figure 12.

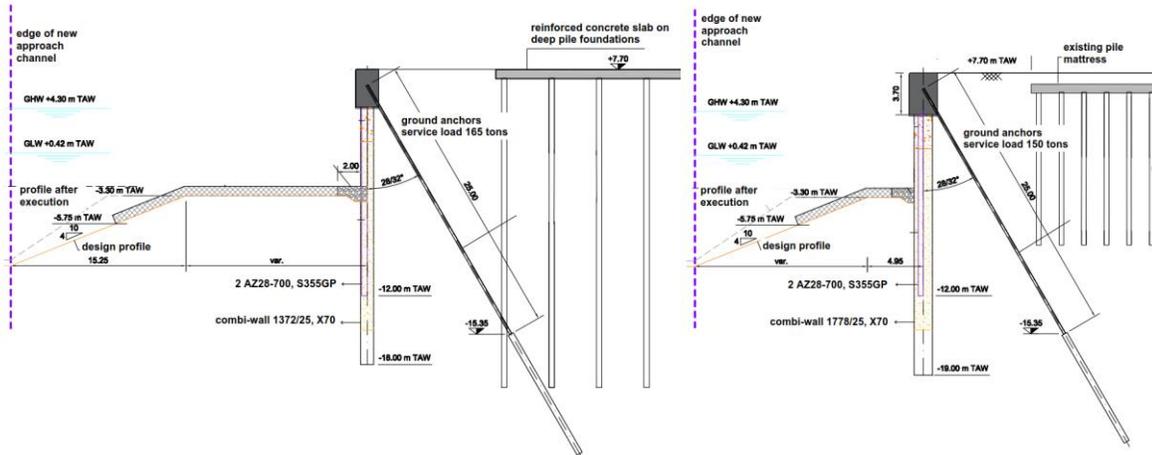


Figure 12: Typical retaining wall cross sections

To limit the retaining height of the wall, it was foreseen to maintain a berm in front of the wall since there is no requirement for mooring of vessels in this area. Although this reduced the retaining height significantly, overall load on the wall was still significant due to the considerable water pressure (tidal variation) and the poor soil conditions behind the wall (leading to high active soil pressures). Therefore, a heavy combi-wall proved to be necessary. Since the entrance channel becomes wider when going south, the available berm becomes narrower, and in the most southern zone becomes the critical section for the combi-wall design (see plan view in Figure 13). In this critical section, the combi-wall consists of large tubular piles diameter 1778mm (cross section shown in Figure 12).



Figure 13: Design for middle and southern project area (retaining wall and quay wall stabilization)

Since the berm is located at a shallow depth to have a maximum effect on the quay wall (approx. 3m below MLW), it is potentially subjected to wave action in extreme conditions. Because the berm is of key importance for quay wall stability, it was designed to be hydraulically stable under the extreme wave action. This resulted in a scour protection consisting of a willow mattress covered with stones up to 300kg. This scour protection is applied on the horizontal part of the berm as well as on a part of the slope. A design profile allowing for some scour was taken into account in the calculations, and a monitoring programme has been developed to assess the scour progression during the lifetime of the quay wall.

A particular point of interest is the anchoring of the quay wall. Due to the presence of the existing pile foundations that have to be preserved, top fixation of the quay wall was not particularly straightforward. Interference with the existing pile foundations is unavoidable but has to be limited to the absolute minimum to avoid issues during construction. Therefore, the following methodology was developed:

- It is foreseen to make an exploratory trench prior to the works to have a clear idea about the position of the existing pile foundations.
- Based on this topographic campaign, the anchor positioning will be confirmed. The design philosophy of the anchors is as follows (see Figure 12 for a comparison between the two design situations):
  - For the zone with deep pile foundations (spaced approx. 4.13m) in the north, it cannot be avoided to cross the piles with the anchors. However, with a pile spacing of 4.13m, it is considered feasible to provide two anchors between each pile row (meaning that anchor spacing automatically is 2.065). But since only the first pile row can be detected with the exploratory trench, angle of the anchors is minimized to avoid potential interference with second pile rows, leading to an inclination angle of 60° (with the horizontal).
  - For the zone with more shallow pile foundations in the south, spacing of these piles is only 2m in all directions, which was considered too small to allow for anchors to be drilled in

between the existing pile foundations (only the first pile row can be detected with certainty). Therefore, anchors are designed to be drilled underneath these piles, leading to an inclination angle of  $60^\circ$  (with the horizontal). On the other hand, this meant that spacing of the anchors could be selected without restriction, and an intermediate distance of 1.48m was selected for these anchors.

Based on the above, it is clear that the anchors have to be positioned quite vertically to avoid interference with the existing deep foundations of the pavement area. Due to this inclination, combined with the poor geotechnical conditions, design anchor loads were quite high with service loads up to 165 tons. Fortunately, the grout body can be positioned in a very dense sand layer with average cone resistances  $q_{c,av}$  up to 25MPa and more. Geotechnical capacity was theoretically demonstrated using the common rules in CFMS (1995). Additionally, capacity was verified according to latest Belgian practice (Huybrechts et al., 2008). Both methods lead to acceptable results with respect to anchor capacity. Since anchor service load is high and inclination is quite vertical, an extensive full-scale testing programme is foreseen to validate the design loads for these ground anchors. This full-scale testing programme foresees in several tests for each anchor type. For each anchor type, a separate reaction beam on pile foundations shall be constructed and additional CPT testing at each test location is foreseen to validate the theoretical calculation of the anchors. Goal of the testing programme is to validate the anticipated service loads (absolute value) and assess if creep under this maintained load remains within the acceptable limits.

#### **4.4. Southern area – stabilisation of existing quay wall**

In the southern project area, the existing seawall ends in a quay wall. This quay wall of the Danish type has to be preserved, as it is the loading area for the offshore exploitation zone mentioned in the previous paragraph. Since the widened channel comes quite close to this existing quay wall, a solution had to be found to reinforce the quay wall and allow for an increase in depth. Modifications to the existing quay wall were considered too complex, and preference was given to less invasive solutions. Since the main issue was an increase in depth at a certain distance from the quay wall, but within the passive wedge, it was decided to build a gravel berm in front of the quay wall (Figure 14). Using this berm made it possible to avoid a significant increase in bending moment in the retaining wall and allowed for sufficient earth resistance (passive earth pressure in front of the wall). It is noted that this berm solution is only acceptable because there is no requirement to allow berthing of vessels along the quay side parallel to the approach channel. The berm is hydraulically protected using the same system as for the retaining wall in the middle project area (willow mattress covered with stones).

However, for the corner section of the quay wall the widened channel comes so close to the quay that a simple berm proved to be insufficient to provide the required degree of safety with relation to ground failure. Moreover, vessels need to be able to berth at the part of the quay wall perpendicular to the approach channel (required draught approx. 6m below MLW), meaning that at the corner point of the quay wall a berm solution is not possible. The design was developed further to take the 3-dimensional spatial distribution of soil pressures into account in the corner section of the quay wall. An analytical solution for the problem was derived, leading to a slight reduction in the active soil pressures acting on the sheet piles in the corner section. This was also verified with a simplified 3D numerical model, which confirmed the analytical results. Calculation showed that using this approach, bending moments in the quay wall remain within the acceptable values w.r.t. resistance of the sheet piles themselves. Nevertheless, an additional reinforcement was necessary since a sufficient earth resistance (passive earth pressure in front of the wall) could not be guaranteed. Therefore, it was decided to increase the passive resistance by construction of vertical gravel columns in front of the retaining wall as shown in Figure 14. The gravel columns have a diameter of 70cm and are placed in a triangular grid, with inter-distance of 1.80m. Due to the gravel the average shear characteristics of the soil inside the passive wedge are improved, to the level that is required to guarantee the necessary factor of safety on the passive resistance in front of the wall.

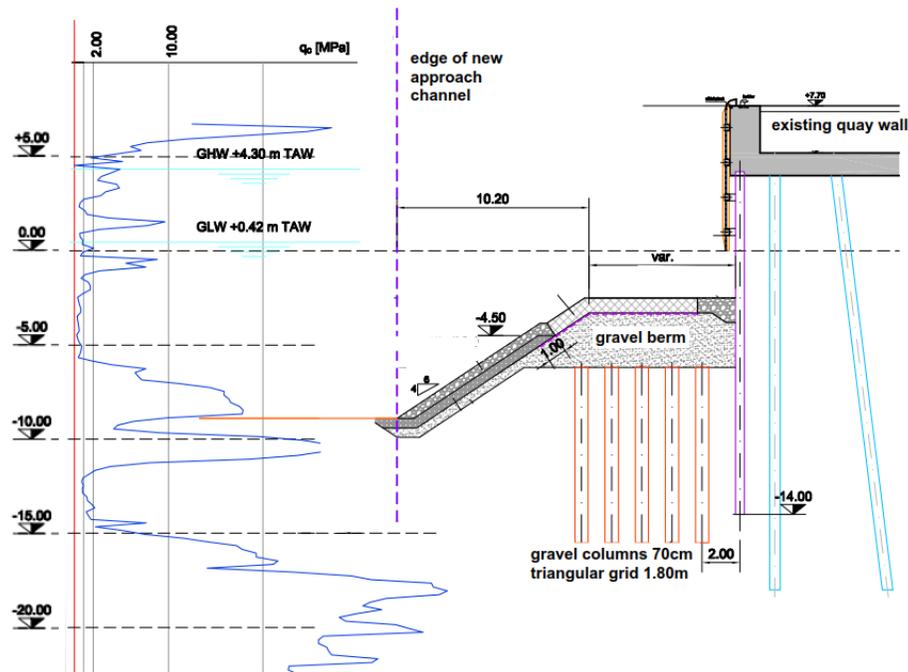


Figure 14: Typical cross section stabilization of quay wall with stone columns and gravel berm

#### 4.5. Execution of the project

The project will be launched for tender in spring 2018, construction works should start towards the end of 2018 and should be finished in the course of 2020. When the envisaged works are finished, the last phase of the masterplan for the port of Ostend will be completed.

### 5. CONCLUSION

For the last phase of the masterplan for the port of Ostend, the entrance channel needs to be widened to make the harbour of Ostend accessible for ships with lengths up to 200m. To make this expansion possible, the existing site 'Halve Maan' needs to be modified thoroughly. An extensive research was performed to determine all site-specific conditions which lead to an exhaustive list of requirements that had to be taken into account: interference with existing structures, the poor geotechnical conditions, the close presence of exploitation zones, the strict environmental constraints etc.

All these constraints had a significant influence on the design which resulted in a multidisciplinary project, that can be split in 3 different sub-projects. In the northern area, a new revetment was designed using the latest design guidelines, completed with practical experience for this field of application. For the middle area, a new combi-wall was designed using heavy ground anchors with service loads up to 165 tons. Finally, in the southern corner section, the existing quay wall was reinforced using a gravel berm and gravel columns in front of the quay wall.

As the works cover a range of challenges related to building into an existing harbour with limited space and many boundary conditions, the project is a typical example of modern day port expansion works. It is demonstrated that it is possible to realize this type of project successfully, but a significant design effort is necessary to fulfill all stakeholder needs in agreement with all site-specific requirements

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