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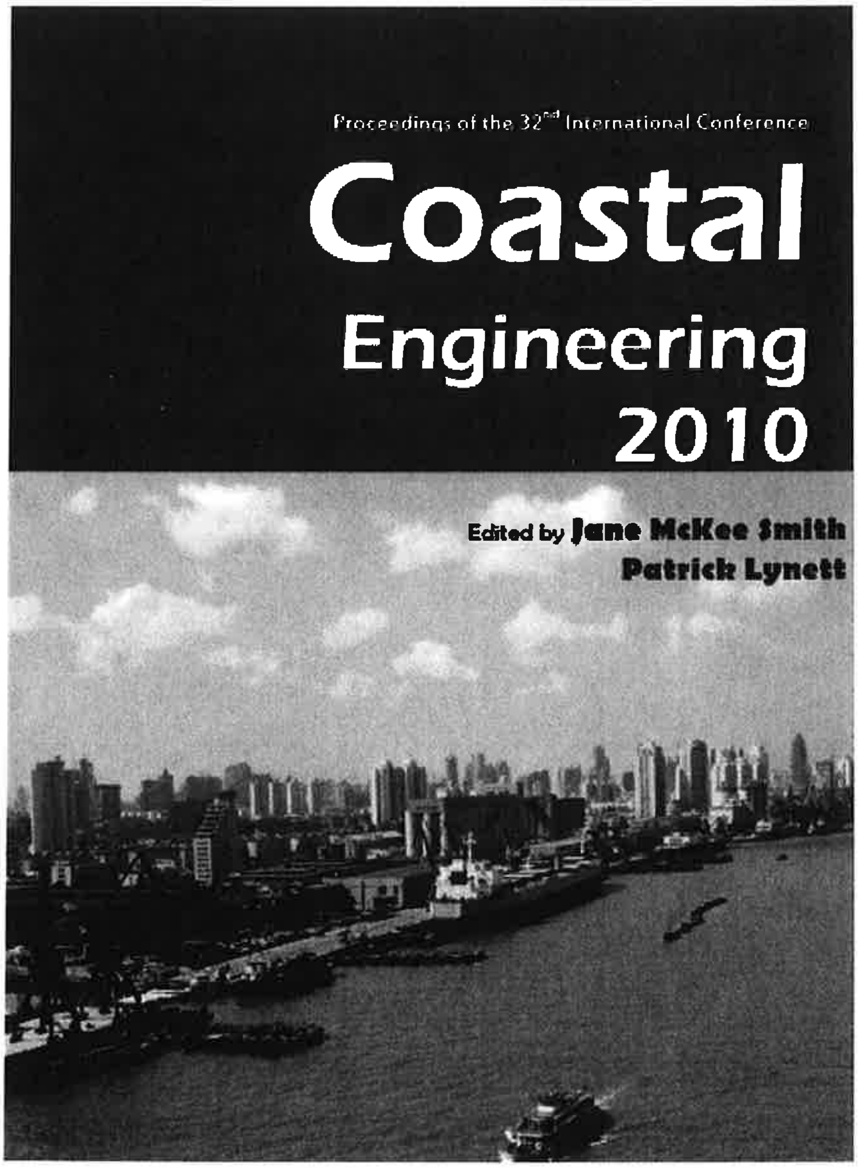

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Abstract: This *Proceedings* contains 363 papers and 29 posters presented at the 32nd International Conference on Coastal Engineering, which was held in Shanghai, China, 30 June to 5 July 2010. The *Proceedings* is divided into seven parts: Keynote; Waves; Swash, Nearshore Currents, and Long Waves; Sediment Transport and Morphology; Coastal Structures; Coastal Management, Environment, and Risk, and Posters. The individual papers cover a broad range of topics including theory, numerical and physical modeling, field measurements, case studies, design, and management. These papers provide engineers, scientists, and planners state-of-the-art information on coastal engineering and coastal processes.

Foreword

The 32nd International Conference on Coastal Engineering (ICCE 2010) was held in Shanghai, China, 30 June to 5 July of 2010. The Local Organizing Committee, led by Xie Shileng, Ge Jiufeng, Dou Xiping, and Zuo Qihua, is acknowledged for their dedicated preparation over many years that led to a successful conference with broad participation. Six-hundred attendees from 38 countries gathered at the Shanghai International Convention Center to discuss research and applications in coastal engineering. The papers contained in this *Proceedings* cover a wide range of topics including waves; swash, nearshore currents, and long waves; coastal management, risk, and environmental restoration; sediment transport and morphology; and coastal structures. The authors have provided state-of-the-art contributions, and this volume could not be produced without their commitment to solving coastal engineering challenges. The members of the ASCE/COPRI Coastal Engineering Research Council (CERC) and the ICCE 2010 Technical Review Committee reviewed 725 abstracts and selected the 418 paper and 55 posters that were presented at the conference. The dedication of the Council members has led to the continued high quality and popularity of the International Conference on Coastal Engineering.

Preparation of these proceedings would not be possible with the assistance of many colleagues. Thank you to Prof. Robert A. Dalrymple, CERC Chairman, and Prof. Billy L. Edge, CERC Vice Chairman, for their guidance and encouragement. Thanks to Ge Jiufeng for answering my many requests for information and for his gracious hospitality in Shanghai.

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GEOTECHNICAL DESIGN OF BREAKWATERS IN OSTEND ON VERY SOFT SOIL

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Two new breakwaters are being constructed to protect the renewed harbour entrance in Ostend, Belgium. In this paper, both the hydraulic design and geotechnical design are discussed. For the hydraulic design, model tests have been carried out in a 2-D wave flume to optimise the cross section. Stability of the final breakwater, with special attention to the crest element and the armour unit HARO, and stability during the construction phases were tested. The geotechnical design, especially of the north-western breakwater, was an engineering challenge since it is located above a thick layer of very soft soil. A combination of building in stages (consolidation time) and providing a strong reinforcement (geotextile) solved this issue.

Keywords: hydraulic stability, HARO, geotechnical stability, geotextile, soft soil, consolidation time

INTRODUCTION

Despite being a relatively small harbour, Ostend, a city situated in the middle of the Belgian coastline, plays an important role in the southern North Sea. The harbour of Ostend is a versatile short sea port, but it mainly focuses on passenger and car-ferry transport. Due to the economical progression, port infrastructures need to be improved and enlarged. Until 2008, the harbour entrance was protected by 2 jetties that were oriented to the north-west. Since this was not perpendicular to the harbour mouth, ships were forced to make an S-curve to enter the harbour. The harbour of Ostend therefore was limited to ships of 120m. To make the port accessible to ships up to 200m, the harbour entrance is reoriented to the north and two new breakwaters are being constructed on both sides of the access channel (Figure 1).

Figure 1. Old harbour entrance (grey lines) is replaced by the new entrance (blue lines) and protected by 2 new breakwaters (yellow lines).

The western and eastern breakwaters are 700m and 800m long, respectively. The breakwaters are typical rubble mound breakwaters armoured with HAROs.

The site investigation showed a sandy sea bottom at the location of the eastern and the south-western breakwaters. However, at the location of the north-western breakwater (NW breakwater) very soft soil layers were found from the corner to the head of the breakwater. Intensive research and engineering skills were necessary to address this problem.

This paper deals mainly with the geotechnical design of the northern part of the western breakwater. Also, the hydraulic design of both breakwaters is shortly discussed.

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Figure 2. Renewed harbour entrance, protected by the Eastern and Western breakwaters (to be finished by august 2012)

HYDRAULIC DESIGN

Scale model tests (Froude scale 1/30) have been carried out in the wave flume of Ghent University (30m length x 1m width x 1.2m height) to define the cross sections of the eastern and western rubble mound breakwaters. Stability of the armour layer (both seaside and harbourside) and of the crest were of primary importance during the tests. After finding the final cross section, the construction phases were tested and optimised. The most important conclusions of this study are summarised here.

Boundary conditions

The final layout of the breakwaters is designed for a storm with a return period 100 years. This storm is characterised by a still water level of TAW + 6.7m (TAW = MLLWS + 0.388m in Ostend), a design wave height of $H_S = 4.8$m and a peak period of $T_P = 10.6$s. The seaside of the breakwater must be designed to resist this wave impact.

The view of the open sea when looking at the harbour entrance from the esplanade or the beach is very important from a touristic point of view. Consequently, the crest level of the breakwater has to be low, namely TAW + 8m. This results in a small crest freeboard leading to large overtopping during the design storm. Therefore, both the crest and the armour layer at the leeside of the breakwater have to withstand the impact of overtopping waves.

Another boundary condition is the wave penetration through the harbour entrance. The leeside of the breakwaters needs to be designed to resist such a direct wave attack.

Crest

In the originally proposed design, the crest consisted of an L-shaped concrete element, as shown in Figure 3 (top left). The front faced the wave impact. The model tests showed that this crest element was not stable. Adding a heel to create extra resistance (Figure 3, top right) or changing the vertical wall into a triangularly shaped one in order to direct the resulting force downward into the breakwater core (Figure 3, bottom left), were separately not sufficient to result in a stable configuration.

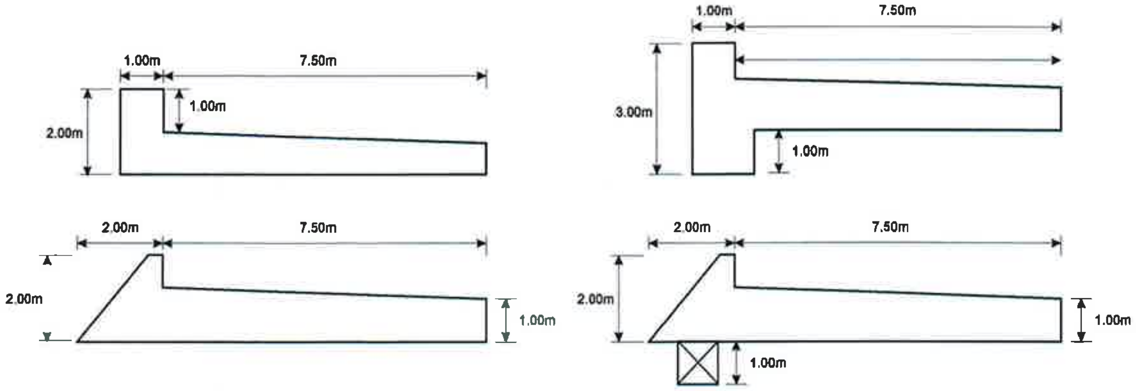

Figure 3. Original crest element (top left), a heel under the crest element (top right), sloped front wall (bottom left), combination of heel and sloped front wall (bottom right).

A combination of both measures together resulted in the final design (Figure 5). Instead of a permanent heel, a "tooth" (1m x 1m x 1.5m) was constructed every 7m.

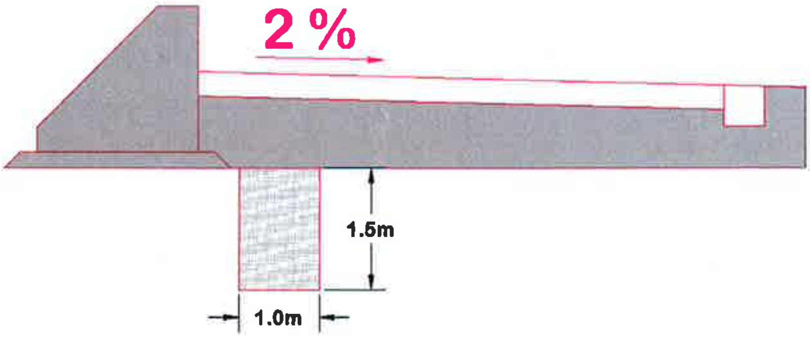

Figure 4. Detail of toe final crest element

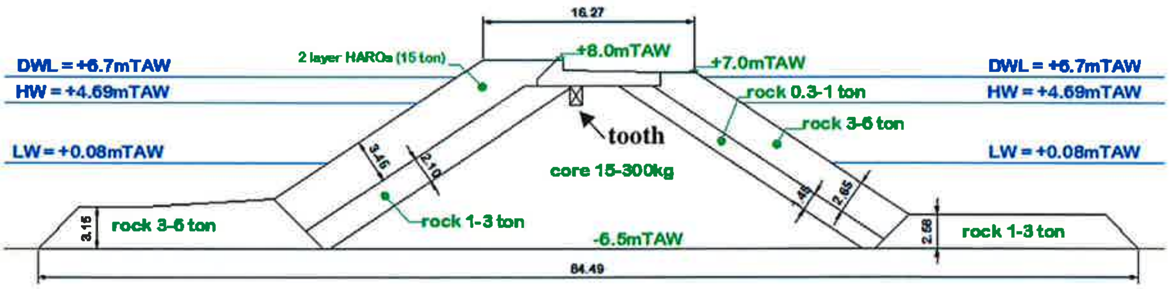

Figure 5. Final layout of the northern part of the Ostend breakwaters.

Final Cross Sections

Going from the coast (south) to the sea (north), the armour layer of the seaside of both breakwaters consists of 3-6 ton rocks and a transition zone with 1 layer of HAROs. Finally, the most northern part of the breakwaters is armoured with 2 layers of HAROs (Figure 6). Model tests show a very high stability of the HAROs under wave attack of 5m and higher, with only 1% of the blocks moving more than half of their width, and no blocks moving over a distance equal to their width (Figure 7).

Figure 6. Double layer HAROs as armour layer on the seaside of the breakwaters

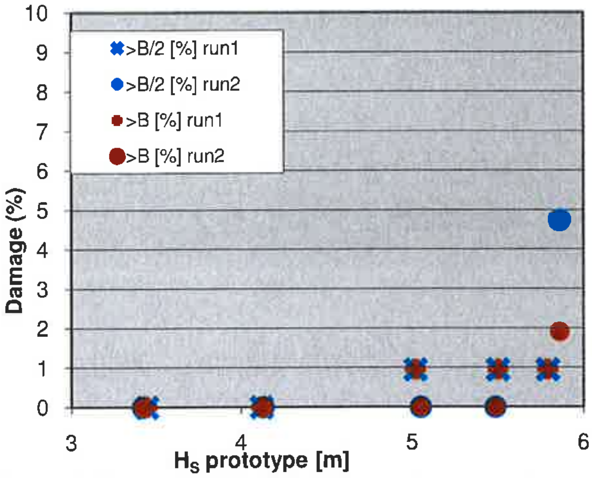

Figure 7. Damage graph of the double layer HAROs under increasing wave impact.

Due to the wave penetration through the harbour entrance, an armour layer consisting of 3-6 ton rocks is necessary for the main part of the harbourside of the breakwater. The low crest freeboard makes this zone also exposed to severe overtopping. Model tests have shown that the crest element with horizontal parts of 3m on both sides fulfill the stability demand with a damage of < 5%.

Construction phases

As described in the tender document, construction of the breakwaters will be completed in 4 stages:

- placing the willow mattresses and breakwater toes
- construction up to TAW +3.0m
- construction up to TAW +6.0m
- finishing at TAW + 8.0m

For every construction stage, model tests were carried out in the wave flume. The results are summarised in Table 1.

Name	Crest level	Critical water level	max H_{m0} (measured)	remark
	mTAW	mTAW	m	
Phase 1 (willow mattress & toe)	Variable (dependent on bottom level of cross section)	1m above crest level	2.10	Some movement of the 15-300kg core, acceptable
Construction to TAW + 3.00m	+3	+2	4.30	Covering the core with 1 layer of 1-3 ton is necessary
Construction to TAW + 6.00m	+6	+5	4.70	Covering the core with 2 layer of 1-3 ton is necessary

Table 1 - Construction phases

The design storm during the construction phases is a storm with a return period of 10 years (while R = 100 years is valid for the final construction). This storm is characterised by a wave height of 4.0m. In the physical model, tests series with H_{m0} up to 120% of the design wave height are carried out. This value cannot be reached at all locations of the breakwater due to limited water depth. In the wave flume, waves break when they reach a height of about 45% of the water depth, leading to the lower values in Table 1 (2.10m, 4.30m and 4.70m).

The crest level in phase 1 depends on the level of the sea bottom at the location of the cross section. A number of cross sections have been tested with differing water levels. Most rock movement was visible when the still water level was about 1m above the core material. This only occurs in the cross sections closest to land with shallow bottom levels. Due to this low water depth, waves of only 2.10m could be achieved. The rock movement, however, was still acceptable, and none of the core material had moved outside the toes of the breakwater.

Cross sections deeper into the sea always have higher water levels above the core material. Even with higher waves, no rock movement was observed during these tests. These findings are confirmed by (van der Meer et al, 1996).

For the second phase, building up to TAW + 3.00m, the critical water level was 1m below the crest level, according to the model tests. Waves broke directly on the structure, washing away the relatively fine core material. Protection by 1-3 ton rocks was necessary and one layer (1.05m thick) proved to be sufficient to secure the stability of the core during wave impact.

In construction phase 3, building up to TAW + 6.00m, a freeboard of 1m was again critical for achieving stability. This time, model tests showed that a 2m thick layer of 1 to 3-ton rock was necessary to guarantee that the core material was covered during a design storm with a return period of 10 years.

GEOTECHNICAL DESIGN

Soil investigation

Regarding the geotechnical design of the breakwaters, an extensive soil investigation campaign was carried out on-site, employing cone penetration tests (CPTs), boreholes, geo-electrical research and sub-bottom profiling. Furthermore, laboratory investigations and a historical research were carried out.

At the location of the eastern breakwater, a thick layer of dense sand with an intermediate layer of clayey sand around TAW -15m is present (Figure 8).

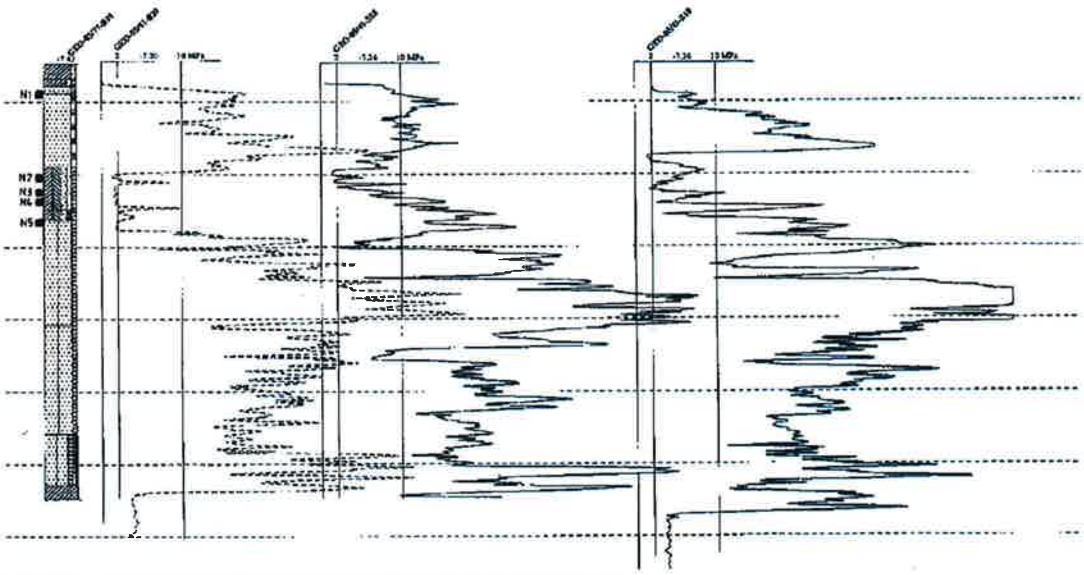

Figure 8. Typical CPTs at the location of the eastern breakwater

At the location of the south-western breakwater, soil layers were found that are similar to those at the eastern breakwater. However, at the location of the northern part of the western breakwater, very soft soil layers were discovered over a length of 350m. The thickness of these layers varies drastically from one CPT to another (Figure 9).

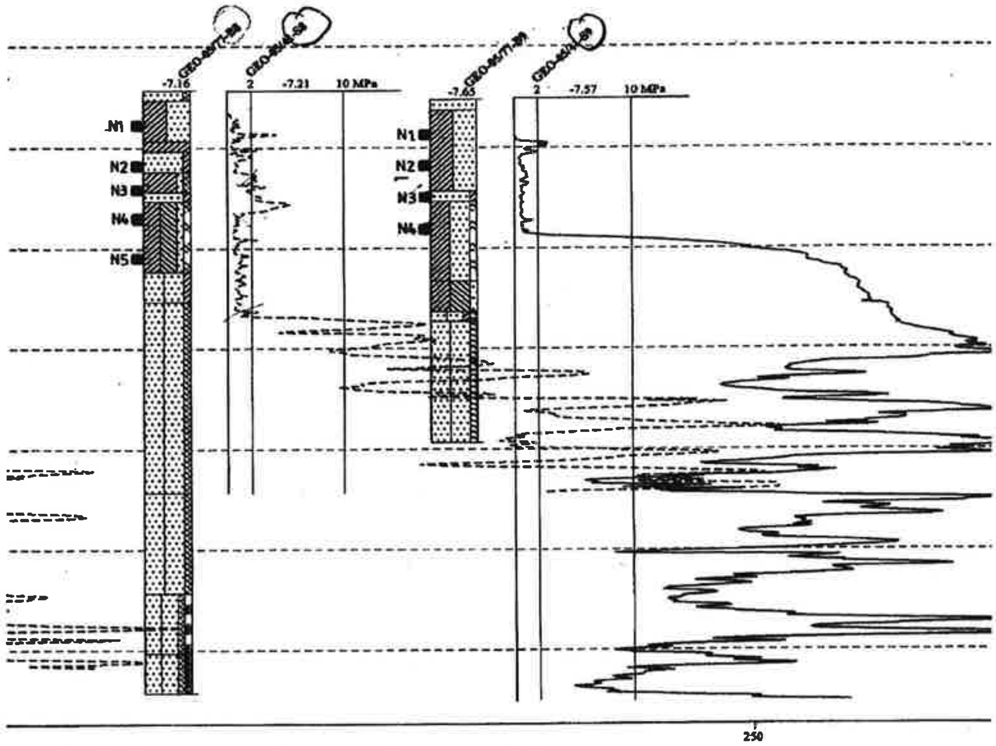

Figure 9. Two CPTs close to each other, at the location of the NW breakwater

The boreholes showed an alteration of soft, sandy clay and loosely packed clayey sand. This is in clear contrast with the eastern and south-western breakwaters. The origin of these soft layers is former dredging works. About 25 years ago, dredging works were performed to maintain the access channel and to acquire clean sand for building purposes. These works were carried out with suction dredgers, creating deep holes in the sea bottom. Due to natural sedimentation, these holes became filled with soft sediments over the subsequent years. This created a very heterogeneous soft soil of 8 to 10m thick at the location of the NW breakwater.

Consequences for the geotechnical design

Generally speaking, the presence of thick soft soil layers is of major importance for both the overall geotechnical stability of the slopes and for settlement behaviour. First we must solve the problem of the stability, and then we will consider the settlements.

Several techniques are possible to tackle the problem of slope stability, including:

- Placing a heavy toe: this has little influence on settlements
- Strengthening soil through, e.g., a strong geotextile: this will minimally influence settlements
- Applying soil improvement techniques through, e.g., gravel piles: settlements will decrease
- Soil replacement (the soft soil is dredged and replaced by sand): settlements will strongly decrease

With regard to the geotechnical design of the NW breakwater, the above techniques were investigated taking into account technical quality, influence on timing of works and, of course, cost price.

Soil replacement is well known in Belgium, and has been applied to the breakwaters in Zeebrugge for more than 7 km. However, a permit is needed to dump the excavated material, which is not easy to obtain. Moreover, the cost price for this relatively short trench (only 350m) is high due to mobilise/demobilise cost of material.

In soil improvement, the efficiency of gravel piles is questionable, as they are cut horizontally by a potential slip surface, so the contribution to the shear resistance along the slip surface is limited. Also, the cost is high.

Soil strengthening with a strong geotextile can be done with all the materials available on-site. Further, producing and placing a willow mattress with a strong geotextile is similar to producing and placing a willow mattress with a normal geotextile.

Finally, using a heavy toe is only an extension of the existing toe, so no additional construction phases are needed. A heavy toe only uses more rock material.

Based on the preceding evaluations, the designer and the client have decided to apply a strong geotextile. This measure has been combined with the use of a heavier toe at critical locations.

Geotechnical characteristics

The weakest subsoil is found at the locations of CPT S8 and CPT S9 (Figure 9). Calculations have been performed using these CPTs.

The basic soil characteristics of the soft layer were determined by extensive laboratory investigation at the Geotechnics Division of the Ministry of the Flemish Community. The most important characteristics with regard to slope stability, consolidation and settlements are:

- wet unit volume weight: $\gamma_w = 16{,}5$ kN/m³
- shear resistance: characteristics c' and φ' are determined by a consolidated undrained triaxial test with measurement of pore water pressures: c' = 3kPa, φ' = 22°
- permeability coefficient $k = 10^{-9}$ m/s
- compression constant (law of Terzaghi) C = 20

In the geotechnical design, both the final fully consolidated situation and the intermediate situations (construction phases) need to be considered.

The main part of the stability analysis is carried out using the software code GEO-SLOPE. For some detailed analysis and settlement analysis PLAXIS is used as well.

Fully consolidated situation

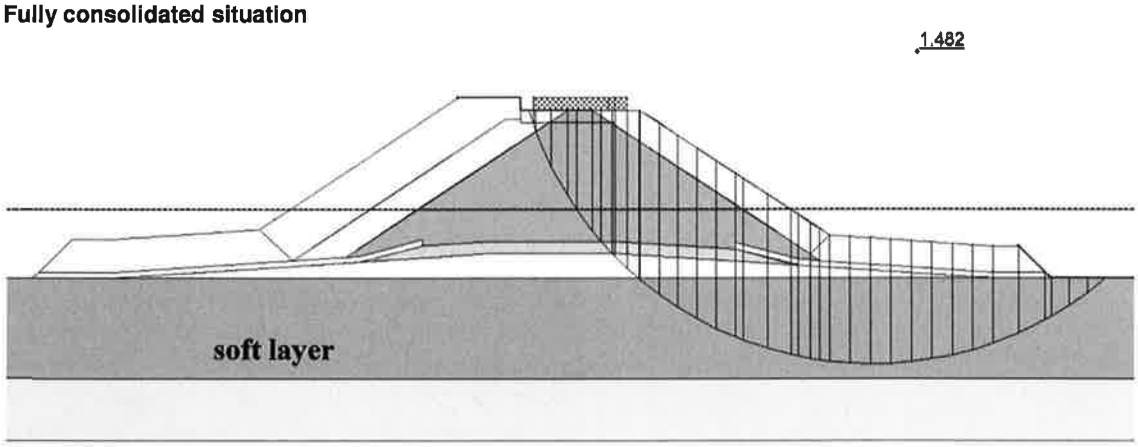

Figure 10. Slip surfaces at leeside of the breakwater

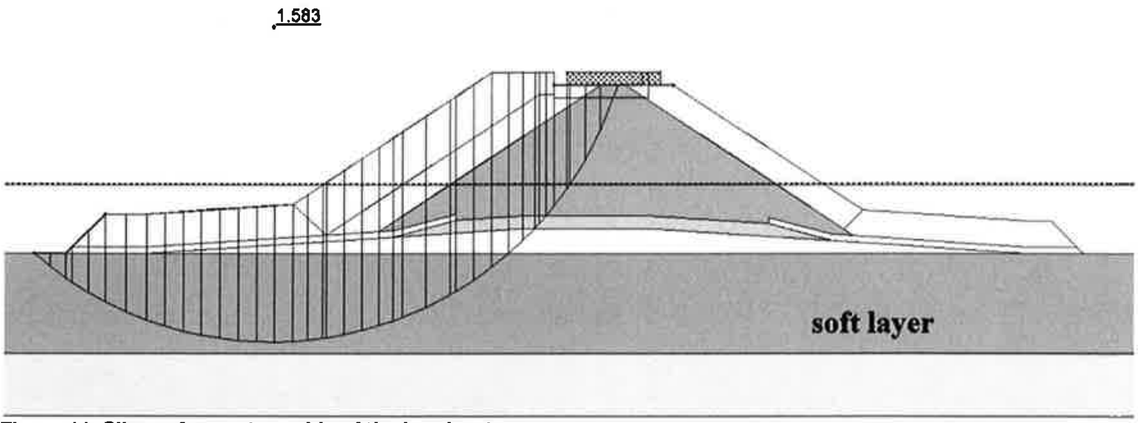

Figure 11. Slip surfaces at seaside of the breakwater

Figure 10 and Figure 11 show the results of the fully consolidated situation. As the tidal range in Ostend is up to 4.5m, low water is the determining situation. A surface load of 20kN/m² is applied.

The overall safety factor equals 1.48 for the harbourside and 1.58 for the seaside. These numbers are higher than the required value of 1.30 for final situations (fully consolidated without wave action). No measures are necessary for the final situation.

It is logical that for the seaside a higher safety factor is found because the unit volume weight of the HARO armour layer (concrete ρ = 2,30t/m³, porosity = 50%) is clearly lower than the unit volume weight of the rock armour at the harbourside (ρ = 2,65t/m³, porosity = 40%).

In the fully consolidated situation, slope stability analyses that take into account the dynamic effect of wave action have been considered as well (De Rouck, 1996). Wave action in the clay layer is damped due to clay's low permeability and high compressibility. Piezometric lines are defined for the bottom layer (red dotted line) and for the breakwater (black dotted line).

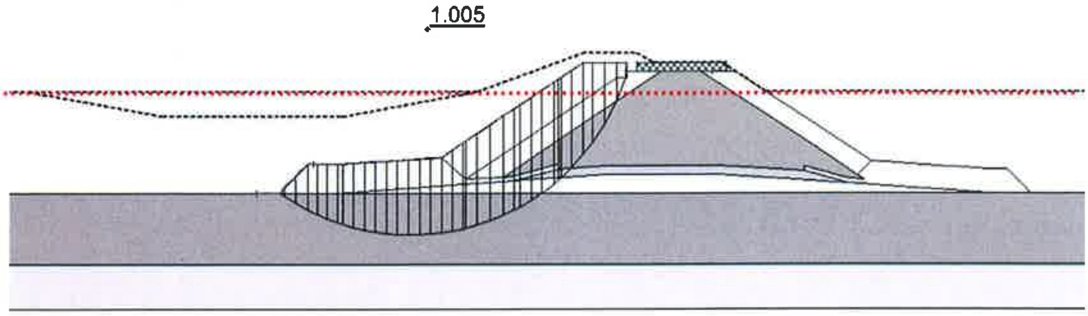

Figure 12. Modeling wave action in GEOSLOPE

Figure 12 shows the result. The safety factor equals 1.005, which is lower than the required safety factor of 1.15. Additional measures to increase stability are thus necessary. As mentioned previously, a geotextile will be applied.

Construction phases

During construction, an overall safety factor FS = 1.15 is required. The construction phases for the eastern breakwater are

a) placement of willow mattresses and toe
b) building core, filter layer and armour layer up to TAW +4.50m
c) building the breakwater up to TAW +6.00m
d) in a later phase finishing the crest of the breakwater to TAW + 8.00m

In a first approach, the same phases were considered for the western breakwater. Slope stability analysis, however, showed that the overall slope stability coefficient was clearly lower than 1.0 at the NW breakwater. Certain measures were necessary to ensure a safe construction of the NW breakwater.

As mentioned, it was our objective to solve the problem using a strong geotextile and, if necessary, heavier toes. Even with only these two measures, it was necessary to construct the NW breakwater in several steps, each step with a clearly defined consolidation time.
Our analysis has led to the following construction phases for the NW breakwater (Figure 13).

- T_0: placing the foundation (approx 4.0m thick, from approx. TAW -7.50m to TAW -3.50m)
- T_0 + 8 months: breakwater till TAW +3.00m
- T_0 + 12 months: breakwater till TAW +6.00m
- T_0 + 18 months: crest

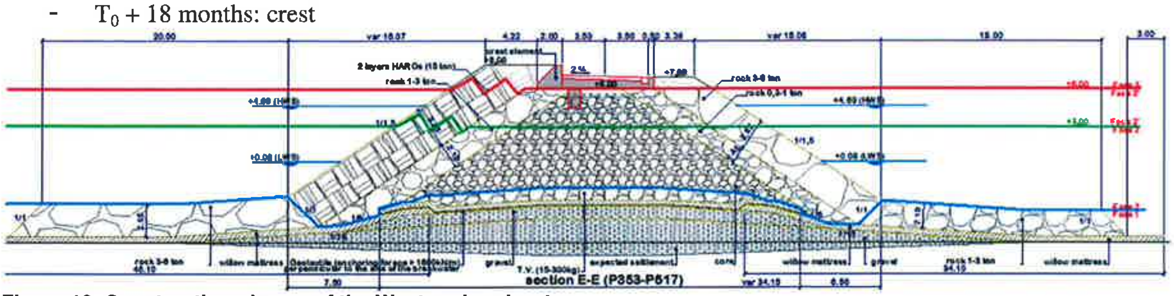

Figure 13. Construction phases of the Western breakwater

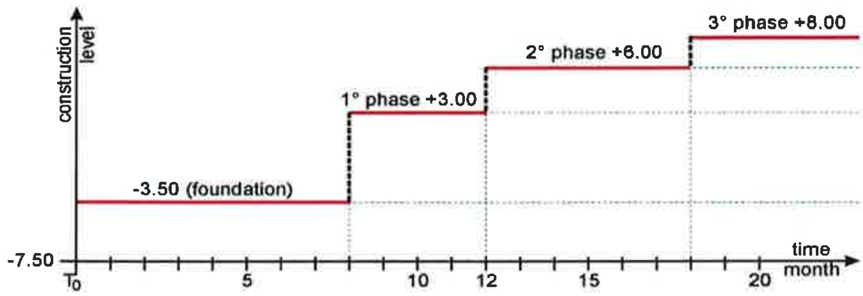

Figure 14. Construction steps in time.

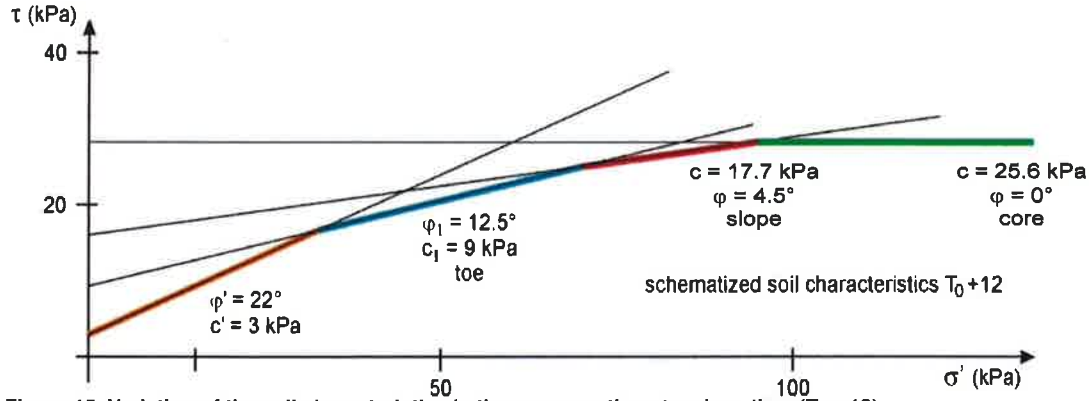

Figure 15. Variation of the soil characteristics in the cross section at a given time (T_0 + 12)

Figure 14 schematically shows the construction phases in terms of time. The construction time needed to pass from one construction phase to another at a certain cross section is schematised as immediate loading: a vertical line in Figure 14. Figure 15 shows the calculated soil characteristics at T_0 + 12 at different locations of the cross section: seaward of the toe, underneath the toe and the slope, and underneath the core.

Analysis with Geoslope

GEO-SLOPE is a typical software code for slope stability analysis based on the method of vertical slices. Geotextiles, or other reinforcement loads, can be applied in this software. The force generated in the geotextile during construction phases and in the finished breakwater under wave action, is listed in Table 2 as "fabric load". This is the primary anchoring force present in the geotextile under the given conditions. This value needs to be multiplied by several factors that reduce the initial strength of the geotextile, such as installation, creep and chemical degradation. The total multiplication factor becomes 3.4. A total design load of 1462kN/m is necessary to obtain a safety against sliding of 1.15. This value occurs during the 3rd construction phase, building up to TAW + 6.00m. When the soil is fully consolidated (c = 3kPa, φ = 22°), the total design load reduces to 731kN/m under wave impact.

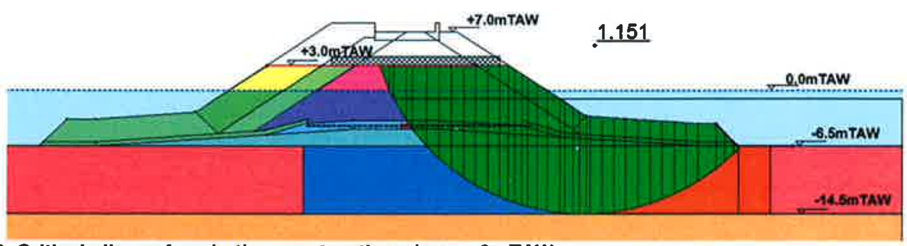

Figure 16. Critical slip surface in the construction phase +3mTAW

Construction phase	Critical water level	Safety factor [-]	Fabric load [kN/m]	Design load (= fabric load x 3.4) [kN/m]
TAW + 3.00m	TAW + 0.00m	1.152	40	136
TAW + 6.00m	TAW + 0.00m	1.151	430	**1462**
Finished, incl. waves	TAW + 4.70m	1.150	215	731

Table 2: Geotextile loads for 3 critical phases during/after the construction of the Western breakwater (geoslope results) at the location of CPT S9

Analysis with Plaxis

In order to study the influence of the construction phases on the tensile forces acting on the geotextile, and to gain insight into the progress of settlements, it was decided to model the phased construction using the finite element software PLAXIS.

Deformations and overall stability were checked. The safety approach of the Eurocode7-method with partial safety coefficients was applied.

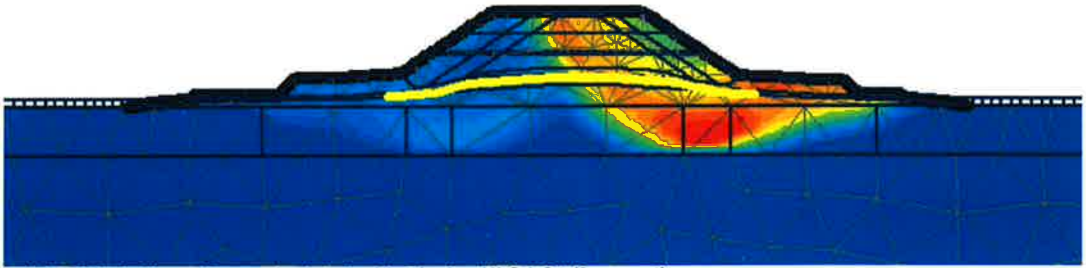

Figure 17. Critical slip surface calculated with Plaxis , ULS LC1 (frequent)

Results were shown to be highly dependent on the chosen type of geotextile. Different combinations of characteristics (tensile strength, material type, strain stiffness) led to different combinations of tensile forces in the geotextile and settlements. The analysis performed showed that for the specific type of geotextile used on-site, a nominal tensile strength of 1442kN was needed to obtain the required safety upon the acting tensile forces.

Tensile forces [kN]	LC 1 Frequent	LC 2 Occasional	LC 3 Accidental	Max	Design load (= fabric load x 3.4)
SWL	TAW + 0.08m	TAW -1m	TAW + 6.70m		
After construction until +6m	263	301	116	301	1023.4
After full consolidation	371	424	176	424	**1441.6**

Table 3. Geotextile loads (PLAXIS-results)

The phased construction analysis showed a settlement of approx 1.0m after consolidation of the underlying soft layer.

Characteristics of the geotextile

The main function of the geotextile is to strengthen the structure in order to avoid overall slope instability. The above analyses in GEO-SLOPE and PLAXIS demonstrated that the necessary strength of the geotextile needs to be at least 1462kN. This high value, combined with a limited allowable strain, led to the selection of a polyester geotextile. Namely, a stabilenka® 1600/250 geotextile was chosen, with 1600kN/m strength perpendicular to the axis of the breakwater and 250kN/m by the axis. The use of such a 'super' geotextile across a length of 350m is new in open seas engineering works.

It is much thicker and stiffer than a common geotextile, but the on-site processing occurs in a similar manner. The 5m wide super geotextile is delivered by the manufacturer and processed into mats of 20m wide and 40 to 50m long. Next, willow branches are attached in a crossed pattern to the super geotextile. Once ready, this mattress is dragged into the water at high water tide, and towed into position at the moment of minimal tidal currents. Rock gradations 10-60kg and 60-300kg are used as ballast (800kg/m²).

Figure 18. Willow mattress ready to be towed into the sea

The strong geotextile underneath the core of the breakwater plays a dominant role in slope stability during the construction period. When applying a geotextile, the following two issues are very important:

a) the so-called "anchor length" of the geotextile, both inside and outside the sliding mass, should be determined carefully. The tensile forces occurring in the geotextile have to be transmitted to the surrounding rock mass of the breakwater
b) resistance against sliding comes from two sides – first through shear resistance in the soil and rock, and, second, through an armouring (pulling) force in the geotextile. The strain of the geotextile has to be compatible with the deformation of the soil and rock.

Analysis in both GEO-SLOPE and PLAXIS have shown that the super geotextile needs to placed under the full width of the core. By doing this, there will be sufficient anchor length at both seaside and harbourside.

Underneath the toes, willow mattresses with standard geotextiles are placed.

SETTLEMENT

The soil conditions vary clearly from one place to another. Consequently, one must consider both the overall (final) settlements and the differential settlements.

The expected final settlements will be between 1.00m and 1.30m. Since the breakwaters will be built in several steps, a large part of the settlements will occur during their construction. These will be compensated for by building up to the specified level.

Differential settlements are not a real problem as far as the rubble mound breakwater itself is concerned. For the crest element, however, a solid concrete structure is foreseen and on the western breakwater a promenade for tourists will be built. The design and actual building of the crest will take into account the settlement behaviour of the NW breakwater.

The geotechnical design of the NW breakwater is based on an extensive investigation campaign, both on-site and in the laboratory, to define the soil characteristics. In order to check the theoretic settlements, the construction of the breakwater is closely monitored. Three piezometers have been attached to concrete blocks, which have been placed at the location of the previous CPTs. A fourth piezometer has been positioned at a fixed location, to serve as a reference.

We have measured the settlements, and are comparing them with the theoretical settlements.

As soon as the construction reaches TAW + 3.00m, a plate will be installed to monitor further settlements. The results of this process will determine when the construction of the stiff concrete crest element may begin.

CONCLUSION

In this paper, two main topics related to breakwater design are studied: hydraulic and geotechnical stability. To study the hydraulic performance of the breakwater, model tests have been carried out in Ghent University's 2-D wave flume. The crest element has evolved from an L-shaped, concrete element to a triangular crest block with a tooth underneath the crest element. The armour layer of the breakwater consists of 2 layers of HAROs, who appear to be very stable, with only 1% of the blocks moving more than half of their width under wave impact $H_S = 5m$. Between the different construction stages, measures are proposed to cover the core for a period of months with no further works being performed.

For the geotechnical design, intensive soil investigations were carried out to determine the characteristics of the subsoil. A very thick soft soil was found at the location of the NW breakwater, caused by dredging works occurring 25 years ago. Several techniques to solve the problem of slope stability were investigated, leading to the use of strong geotextiles and the decision to construct in phases to take profit of subsoil consolidation. Where necessary, enlarging the berms of the breakwater was also suggested. Calculations using the software code GEO-SLOPE were carried out to determine the necessary tensile strength of the geotextile, both during construction phases and in a fully consolidated situation under wave impact. A "super-geotextile" of 1600kN/m was recommended and confirmed through independent calculations in the finite element software code PLAXIS. The expected settlement at the location of the NW breakwater amounts 1.00 to 1.30m. Building up to the prescribed level will solve this issue. Settlements are being measured during construction to verify calculations.

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