DESIGN OF THE SCOUR PROTECTION LAYER FOR A BREAKWATER IN AN ESTUARINE ENVIRONMENT

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SUMMARY

This paper is a case study covering the hydraulic and geotechnical design of the scour protection of a rubble-mound breakwater designed as a protection of an offshore LNG regasification maritime terminal in the Rio de la Plata, Uruguay.

The paper focuses on the design strategies developed to deal with the challenges raised during the project of this element, due to the high safety standards imposed by the nature of the terminal, and the special hydraulic and geotechnical circumstances involving this marine infrastructure, where different failure modes (of different nature) are definitely interrelated and can be approached from different directions. As a result, a probabilistic approach was proposed to be combined with physical modelling, as well as with the establishment of operational rules related to the inspection and maintenance of the scour protection system.

INTRODUCTION AND PROJECT REQUIREMENTS

A detached rubble mound breakwater of 1,5 km was foreseen as a protection for an offshore LNG regasification terminal in the Rio de la Plata, at two km off the coast of Montevideo, in a fairly uniform water depth of 6 m.

Figure 1: Location of the project (left) and approximate layout of the required scour protection layer (right): breakwater in red; scour protection layer in green.

The project has been developed to assure compliance with high standards for this type of infrastructure. In order to define the general project requirements, the Spanish Recommendations (ROM) have been followed, in particular ROM 1.0-09. In accordance with this recommendation, a maximum joint probability of failure of 1% and a lifetime of 50 years have been considered, both associated with ultimate limit states (ULS). The probability of failure in ULS has been split up between the different failure modes taken into account for the design.

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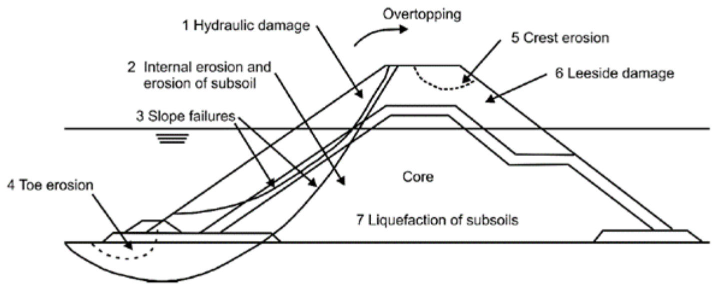


Figure 2: Standard Rubblemound breakwater failure modes (source: The Rock Manual)

The design methodology proposed in the Recommendations for Maritime Works (ROM, Puertos del Estado, Spain) differentiates failure modes between hydraulic and geotechnical on one hand and between principal and non-principal on the other hand. In general, non-principal failure modes can be treated in the design so that negligible failure probabilities can be achieved under moderate costs.

In our case, some of the failure modes shown in Figure 2 were explicitly addressed in the failure analysis of the breakwater as principal failure modes, whereas some others were treated as non-principal failure modes through safe rules of practice (liquefaction, e.g.).The design of the scour protection in front of a breakwater is usually performed assuming a non-principal hydraulic failure mode. However, the special combinations of shallow waters, severe currents and moderate waves, together with very low bearing capacity soils waves presented in the mid Río de la Plata estuary, make that scour protection of this structure should be designed assuming a principal hydraulic failure mode that in turn affects geotechnical failure modes. Additionally, the cost of reducing the failure probability of the scour protection, as a failure mode, to negligible levels is very high.

SITE CONDITIONS

Geotechnical conditions

The Rio de la Plata is the confluence of various (and long) South American rivers, full of sediments, that have been deposited all along the estuary during thousands of years. At the location of the project, this sedimentation generates a shallow platform with a few meters of mud, resting over a sequence of alternating very soft cohesive layers and granular deposits, with variable thicknesses but slightly improving in capacity with depth until -20 m to -30 m Local Chart Datum, resting on a very to moderately weathered rock.

In these adverse geotechnical conditions an intensive geotechnical campaign covering the full extents of the project was performed, including several cone penetration tests and boreholes, from which many samples were recovered to allow for an important set of laboratory tests.

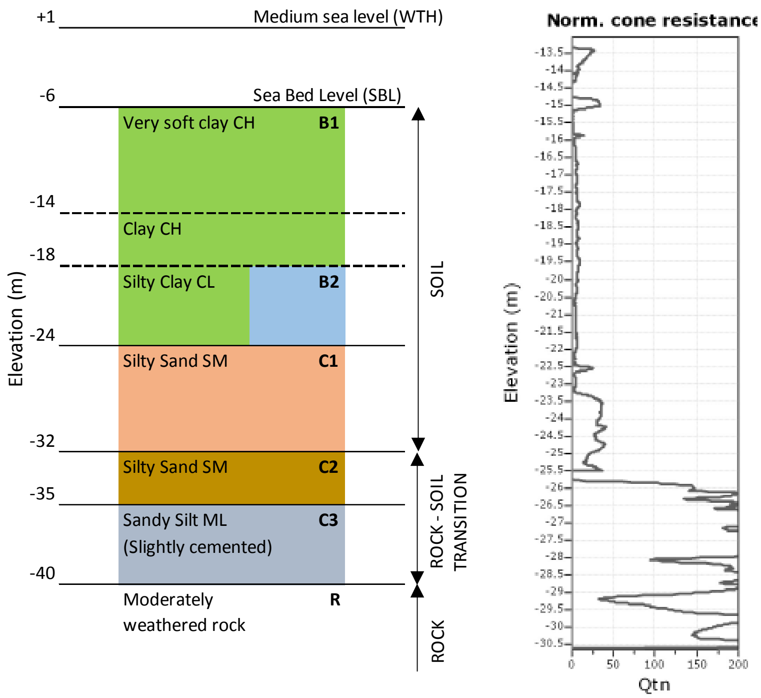


Figure 3: Indicative ground model (left) and typical CPT signal (right)

The results of this campaign allowed for a reliable characterization of the estuary soils and demonstrated the relevance of the very soft cohesive layers (B1 and B2 in figure 3) for the geotechnical design. The extremely low strength properties and the large thicknesses of these layers make them critical in terms of geotechnical stability and confirmed the early stage decision taken for the project to substitute them largely by an extensive backfill of sand (see figure 8 below).

During the detailed interpretation process for the soil properties, care was taken not to mix properties originating from different types of tests. Due consideration was given to use the type of parameter which was judged to be suitable, taking into account the applicable stress state in the actual geotechnical failure mechanism studied, in analogy with the principles below:

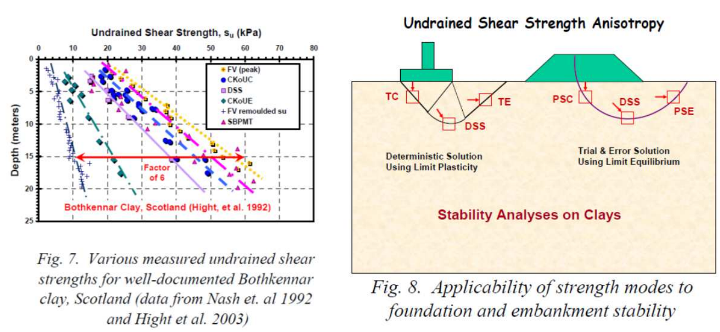


Figure 4: Principles for geotechnical failure mechanism
(Source: Paul W. Mayne, 2008)

An increase in the reliability of the soil parameters in this poorly known estuary was achieved by their determination from varied sources (e.g. the execution of SPT's in close proximity to CPT's). In respect of the actual type of parameter to be used for the particular failure mechanism under consideration, due care has been taken to evaluate mechanical properties from laboratory tests as well as from correlations with field testing. In particular, the undrained shear strength of the principal clays, apart from its triaxial determination, has been deduced from correlations with CPTu registration (Mayne, 2008), in an attempt to find reliable cautious values. The work of Paul W. Mayne, Larsson and Ladd has also been used to incorporate Bayesian knowledge of similar clay materials encountered in other parts of the globe (and their registered coefficients of variation).

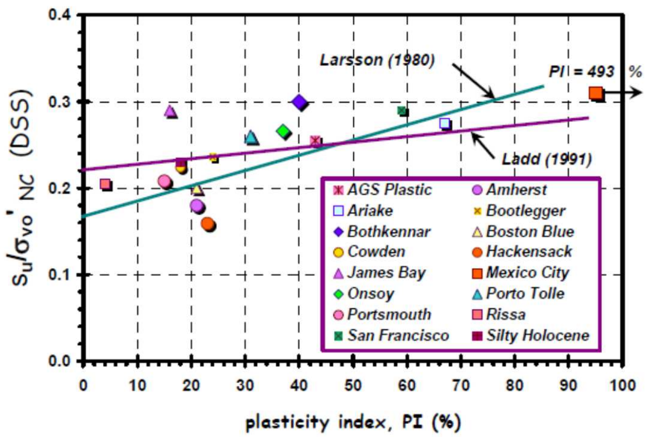


Figure 5: Principles for geotechnical failure mechanism
(Source: Paul W. Mayne, 2008)

Maritime climate

The most relevant environmental agents for the design of the breakwater are those related to the maritime climatic: water level, currents and waves. Although the wind is not treated here as an agent acting upon the breakwater, it was indirectly evaluated, as a source of the rest of the agents. In the case of Montevideo in particular, the three mentioned agents have some degree of statistical dependence, as waves depends on local wind conditions (see e.g. Solari et al. 2014) and sea level and currents depends on local and regional wind conditions (see e.g. Santoro et al. 2011). In addition to this, in some cases statistical extrapolation of extreme condition leads to physical unfeasible values; thus it was required to stablish physical limits for storm surges and waves taking into account local depth and fetch characteristics.

Water Levels

Total water level in Montevideo is influenced by both astronomical tide and storm surges. While the range of the astronomical tide is fairly small, storm surges are relatively strong due to the shallow, funnel-shaped mouth of the Río de la Plata and the wide and shallow continental shelf (see Santoro et al. 201X), so that in some cases the water level at the moment of high tide can be lower than the mean low water level.

Total water level was analyzed based on local long-term records as well as on a water level hindcast. Despite the limited tidal range usually present in the project location, the statistical analysis performed for large return periods (in agreement with the safety level requirements) led to design water levels ranging from -2.5 m to +6.5 m.

Currents

Storm surges do not only influence the water level significantly, but also affect the currents. The current pattern in the Río de la Plata is quite complex, affected not only by the astronomical tides, storm surges and local wind, but also by rivers discharges and salinity gradient and stratification. Characterization of the design current values in the area of study was obtained through downscaling of a regional hindcast, calibrated and validated with in situ data. Finally, design current values have been calculated from a statistical analysis from modelling data.

The presence of the breakwater results in slight adaptations of the current pattern in the vicinity of the toe, leading to high speeds, up to 4,5 m/s in the seaside for an approx. 1000 years return period, but much lower in the sea side.

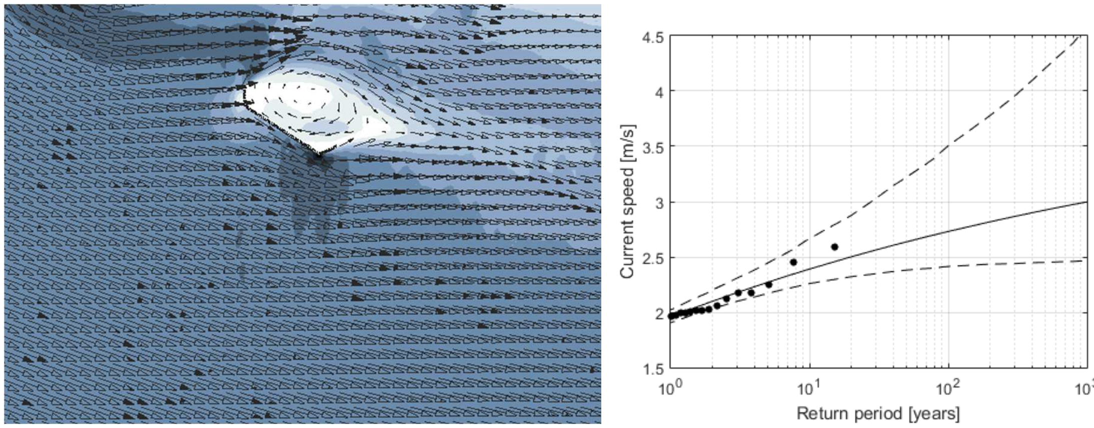


Figure 6: Example of ebb currents field (left) and currents extreme regime near southern elbow.

Waves

The extreme waves that reach the terminal are conditioned by the length of the generation area (fetch), wind intensity and water depth. There is no real fetch limitation for the SE direction, however shallow waters dissipate most of the swell and limits wave growth. Therefore, wave height gradually decreases along the estuary entrance towards the port. On the other hand, the maximum wave heights at the site might be limited by wave breaking by depth.

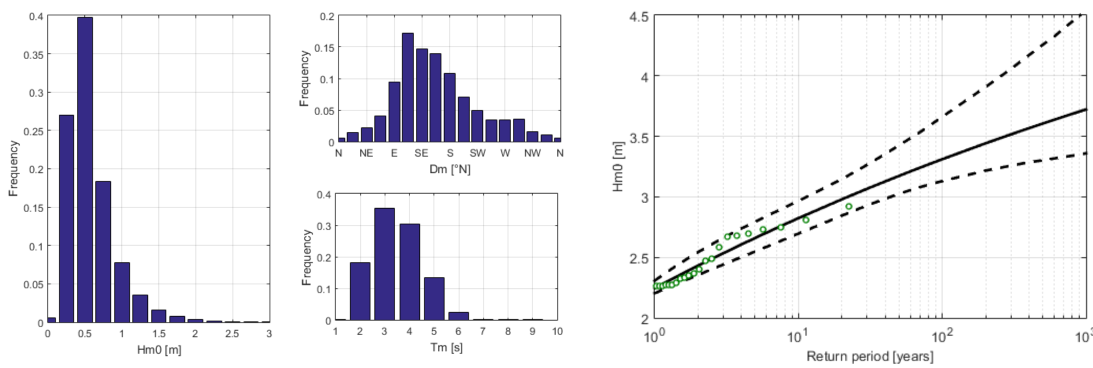


Figure 7: Wave climate: histogram of significant wave height (left), mean wave direction (center top), mean wave period (center bottom) and significant wave height extreme regime (right).

THE PROJECT

Breakwater design. Typical cross section

As expressed above, the project location is subject to very specific geotechnical conditions. The low strength properties of the top layers of the soil have made for a geotechnical design which is very much governed by a global sliding failure mode. In order to improve the geotechnical conditions, it was decided to carry out a global replacement of the soft soil.

The volume of the soil replacement has therefore been a critical parameter in the successful development of the project. From the transverse section presented below (figure 8), the large extent of the sand replacement relative to the section of the breakwater can be appreciated and its economic impact on the project can be understood.

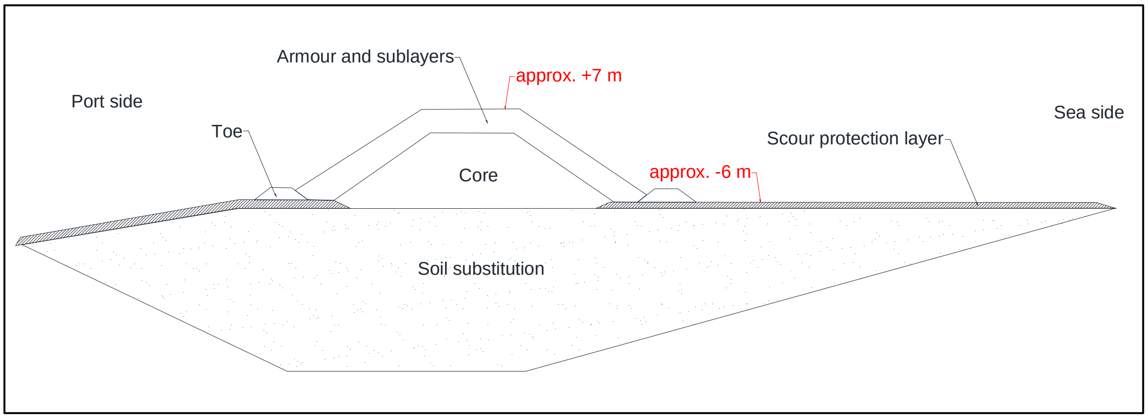


Figure 8: Outline of a typical cross-section of the breakwater

Structurally speaking, the projected breakwater is a conventional rubble mound with an important support function for the toe berm and rock underlayers separating the armour layer from the core. As a main protection against wave action at the sea (river) side, a single armour layer consisting of concrete Accropode units has been design for. The leeside slope of the breakwater is to be protected with quarry units.

Due to the effect of vessel propellers, the back armour and the bottom protection layers have been analyzed thoroughly, as well.

Geotechnical design. Overall stability

The verification of the overall stability was used as a design tool for the dimensioning of the sand replacement. Therefore, the modelling of the global sliding failure modes was carefully assessed.

Overall stability has been verified with the use of different software, such as finite element PLAXIS and GSTABL (rigid body sliding). The impact of using one method over the other has been addressed with parallel calculations for similar design conditions. From a dredging point of view, and due to the permanent sedimentation in the project location, the presence of soft mud remaining in the bottom of the trench, underneath the sand replacement, has been taken into account. It was perceived that the presence of such layers (difficult to exclude completely from the construction process) lead to a significant reduction of the safety factors.

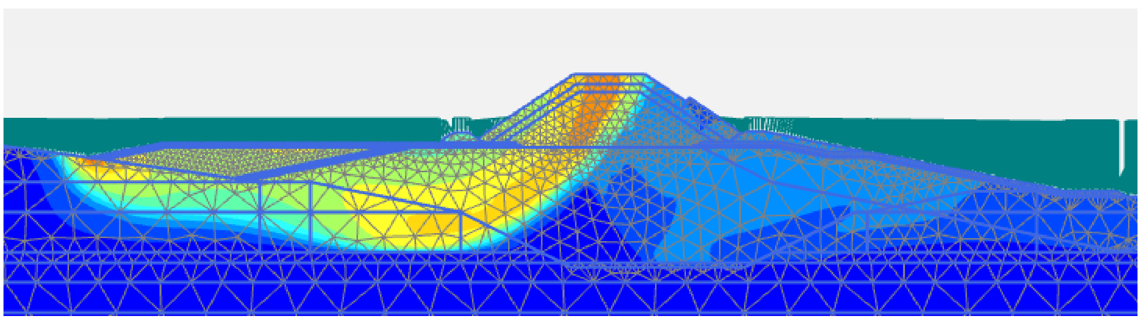


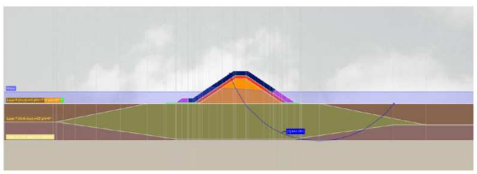
Figure 9: Overall stability Failure Mechanism

As shown in figure 8, the typical failure cross the main body of the breakwater, goes along the sand replacement and the deepest soft clay layers, ending at around 100 m far and more from the axis of the breakwater. This extremely large failure could affect critical structures of the terminal (piping, service platform, main jetty, etc).

Additionally, settlement analyses was carried out with the use of PLAXIS. Settlements in the natural soil and the sand layers occur during the construction and post-construction period, with typical values above 1.5 m.

In the early stages of the project, a semi-probabilistic design was performed for the overall stability (general sliding). As indicated in the 'site conditions' paragraph, attempts were undertaken to reduce the coefficient of variation of the mechanical soil properties. On top of that, the spatial variability of the material properties in the principal clays has been positively affected by the application of normal consolidation laws with depth (physical laws). Finally, the model uncertainty was reduced by testing it to both rigid block sliding algorithms, as well as numerical modelling. Validation of the models was performed for geotechnical problems with analytically known solutions (as described in the Spanish ROM).

Based on all of the above, the geotechnical stability verification was reduced to achieving a standard partial factor on the shear properties of the clays and the sands. Because it was hard to believe that the use of this unique coded factor would result in the target failure probability for the particular problem studied, a variation analysis of the soil properties was performed in accordance with the principle explained in the Spanish ROM. An example of such a variation analysis for a breakwater founded over a sand replacement (with primarily superficial failures) can be found below. Such approach allows to conclude on the importance of the different parameters in the physical problem, hence the importance of the accuracy in their determination.



Parameter	Basic value (a)	Coefficient of Variation (σ)	a - σ	a + σ
γsat,core [kN/m³]	21	0,05	22,05	19,95
γsat,sand [kN/m³]	19	0,05	18,05	19,95
γsat,clay [kN/m³]	14	0,05	13,3	14,7
ϕ'core [°] (+c')	40	0,07 °	37,97	41,92
ϕ'sand [°]	27	0,07 °	25,35	28,60
cu,clay [°]	25	0,15	21,25	28,75
hw (m)*	2	0,1	1,9	2,1

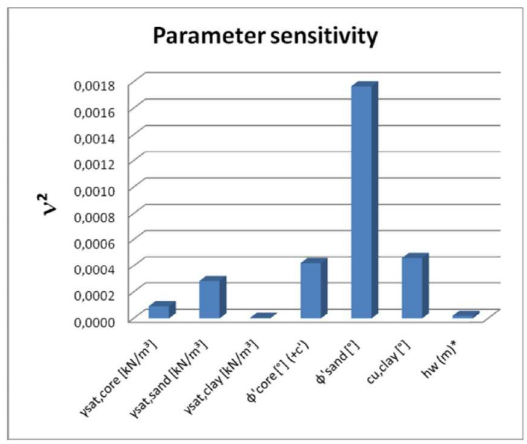


Figure 10: Example sensitivity analysis
(ROM05.05 – 3.3.10 Reliability in Geotechnical Engineering)

Hydraulic design

A Preliminary Hydraulic Design of the breakwater was performed in a Level I approach, with the use of standard formulations (Hudson, Van der Meer, etc.) in their deterministic version.

The final hydraulic design has been performed according to the Advanced Design methodology envisaged by the Spanish Recommendations for this kind of structure: 1st) Level I design, 2nd) Level III verification; 3rd) Possible optimization in light of Level I and III results.

Level III verification entails a Monte-Carlo technique to some representative Preliminary Hydraulic Design cross-sections along the breakwater. The verification procedure follows the Spanish Standards (Recommendations for Maritime Works, ROM 1.0-09). In each cross-section, the considered hydraulic failure modes are studied.

The verification equations for each failure mode (the same as used in Level I approach), are implemented now in the probabilistic version (including the random uncertainties of the fit) and the exceedance of the thresholds or the number of failures is evaluated. In view of the low failure probabilities allowed for, special care was taken with the use of the verification or design equations within their limits of application.

Here we have to say that the hydraulic design (Level I plus Level III) involved the design of all elements, including the scour protection, in order to deal with the failure probability assigned to these failure modes. But we will focus on this design in the following sections.

THE SCOUR PROTECTION DESIGN

In order to warrant the lasting presence of the sand replacement in the stability of the breakwater, taking into account the large currents, an extensive scour protection around the breakwater was designed for to exclude that the overall stability of the breakwater would be compromised. The designed strategy ensures that the sand volumes taken into account in the geotechnical stability remain present during the lifetime of the structure. Hence a coupled hydraulic and geotechnical design was proposed as a design solution of the breakwater.

As expressed before, in the context of the design methodology proposed in the Recommendations for Maritime Works (ROM, Puertos del Estado, Spain) the design of the scour protection in front of a breakwater can be performed assuming a non-principal hydraulic failure mode, as it is usually possible to achieve negligible failure probabilities for this element under moderate costs.

Nevertheless, the special conditions of the mid Rio de la Plata estuary make that scour protection of the projected breakwater off the coast of Montevideo, should be designed assuming a principal hydraulic failure mode that in turn affects geotechnical failure modes.

This is a rare situation for which there are few references in the accumulated experience of breakwater design. Perhaps the most relevant precedent is the design and construction of Zeebrugge breakwaters (De Rouck et al. 2008). Moreover, local knowledge on the actual port of Montevideo is of little use given that current breakwaters were built more than 100 years ago (Nieto, 2012).

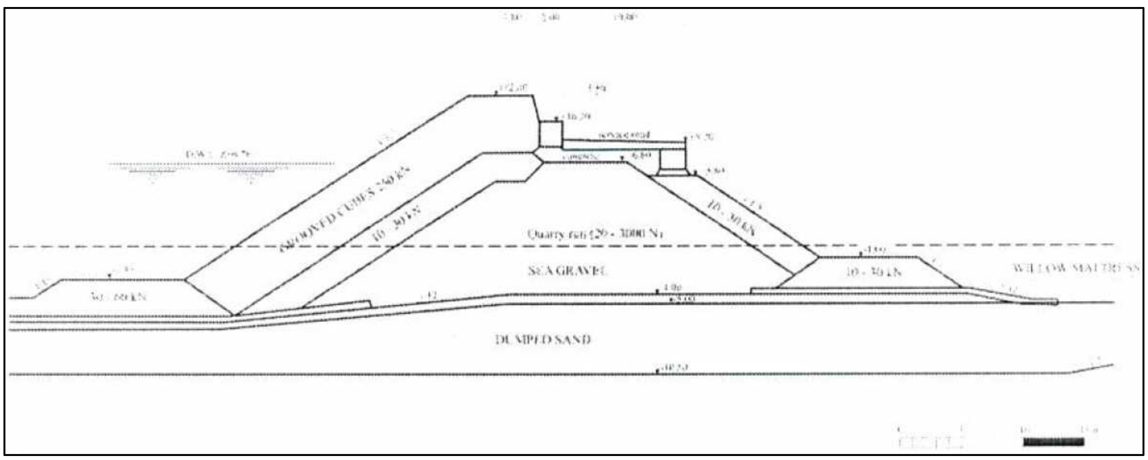


Figure 11. Cross section of the basic design of the breakwater. Zeebrugge Harbour. Source: Julien De Rouck et al. 2008

Conceptual approach to the design

In the design of the scour protection, the geotechnical and hydraulic challenges come together.

A loss of hydraulic stability of the scour protection layer, due to the combined action of currents and waves, can eventually lead to erosion of the sand replacement in front of the structure. This erosion means a loss of material in a place where is needed to ensure the global geotechnical stability of the breakwater against global sliding failures. Therefore, a clear interaction between the hydraulic failure mode "loss of hydraulic stability of the scour protection layer" and the geotechnical failure mode “global stability of the breakwater”, is presented, which cannot be performed decoupled.

The way to deal with this coupled situation is not simple. The methodologies used for hydraulic and geotechnical verifications are fundamentally different, because the temporal scales in which the failures develop, the uncertainties involved in their approach and the analytical and numerical models used in each case are different: on one hand, probabilistic verification techniques are usually an appropriate tool for approaching complex problems of great economic impact and are quite well developed for the verification of hydraulic failure modes; on the other hand, they are not so well developed for the verification of geotechnical failure modes (see e.g. Phoon et al., 2016).

The need of allowing for an unneglectable failure probability of the scour protection (in order to optimize the cost of this element) obliges to make some decisions to lower the failure probability of a geotechnical failure mode (as total failure probability is limited). Therefore, a possible approach to the design is to deal with the hydraulic design calculations according to a certain failure probability, but limiting at the same time the geotechnical failure induced by the scour by introducing physical limits to the geometry in the geotechnical calculations. This idea will be further explained in the 'Approach to the final design' subsection below.

Calculation and verification of the hydraulic failure modes

From the point of view of calculation and verification of the hydraulic failure modes there are at least three challenges: (1) Multivariate characterization of all the random variables involved in the verification; (2) Determination of a verification equation that relates all these variables at the initiation of damage; and (3) Physical modelling of the problem. These three challenges are described below.

First, the multivariate characterization of all the random variables involved in the verification, namely: wave (incident wave height, direction and period), currents (depth averaged intensity and direction), and sea level. Although usually assumed deterministically, two additional coefficients must be added to this list of random variables: the depth limited wave breaking coefficient and the breakwater reflection coefficient. The intensity of the bottom stresses in front of the structure, responsible for triggering the "loss of hydraulic stability of the scour protection layer", will depend on all these variables. To properly describe statistical dependencies among all these variables, as well as their time evolution during storm events, requires fitting and validating a large set of ad hoc statistical models (see e.g. Li et al. 2014).

Several equations have been derived for the design of protection under wave loads (Hudson, Van del Meer, Pilarczyk). These equations come from the Marine Engineering environment and are quite well developed for rock stability in slopes, toe protection of breakwaters, etc, taking into account some relevant effects related to the interaction water-structure, as braking wave action, reflection or energy dissipation. Other kind of equations are derived for the design of protection against current loads (Pilarczyk et al., 1998), or for sediment transport calculations in a River Engineering environment (du Boys, Van Rijn, Bagnold, Einstein, Parker). As a consequence, these formulations have been developed mainly for the design of revetment and protections in rivers, therefore adapted to the particularities of those conditions (soil material, characteristics of the flow, etc).

Currently there are no accepted equations for the design verification of scour protection layers subjected to the combined action of currents and waves (incident and reflected, possibly depth limited, i.e. highly non-linear). The most similar situation for which there are accepted design equations is the start of the movement of the sand under combined wave-current flow (e.g. Soulsby, 1997). This approach is limited, because it only quantifies whether grains are stable or not. The principle of this method is the calculation of the combined bed shear-stress (τ_m; τ_{max}) from the values obtained for the bed sear-stresses which would occur due to the current alone (τ_c) and to the wave alone (τ_w), respectively.

$$\tau_{max} = [(\tau_m + \tau_w|\cos\emptyset|)^2 + (\tau_m|\sin\emptyset|)^2]^{1/2} \tag{1}$$

$$\tau_m = \tau_c\left[1 + 1{,}2\left(\frac{\tau_w}{\tau_c + \tau_w}\right)^{3{,}2}\right] \tag{2}$$

This bed shear-stress is then compared to the critical bed shear-stress for start of movement, estimated based on the Shield's parameter and the particle size of the present material.

$$\theta_{cr} = \frac{\tau_{cr}}{g(\rho_s - \rho)d} \tag{3}$$

But all this calculations process are strongly governed by different parameters, as the so-called bed roughness length, which has been derived for sediments as sand or gravel, outside the domain of the size of the usual scour protection in shallow waters.

Obviously the use of these equations in the design of the scour protection involves great uncertainties (on top of the uncertainty already inherent to sand and gravel transport formulations), which must necessarily be taken into account in the final design.

Third, given the uncertainties inherent to the design process, it is common practice to perform reduced scale model test of a breakwater in a hydraulic laboratory prior to its construction. Physically modeling the interaction of waves, currents and a partially reflecting structure, and its effect on the scour protection layer, presents great challenges for both its implementation in the laboratory and the interpretation of the obtained results, in the latter case mainly due to scale effects.

The implementation of wave and current conditions in a physical model implies the availability of special tank installations, not only to reproduce both agents separately (and accurately), but also to avoid undesirable effects due to their interaction. This interaction leads to difficulties in the determination of the real conditions that are being simulated (when compared with the combined theoretical conditions). Although many tanks exist where water waves can be simulated in 3D conditions, not so many tanks are prepared to combine 3D waves and 3D currents, with the minimum required scale (necessary for the simulation of bed mobility problems), and with the capacity to produce the minimum required current speed.

For the case of the scour protection in front of a breakwater, we are faced with the complexity of simulating (almost) perpendicular water waves and currents, parallel to the breakwater. In this situation, special attention must be given to the generation of the currents, in order to: obtain the right current speed profile at the location of the structure (target zone); avoid reflections in the adjacent structure; and avoid perturbation in the 3D water wave pattern (due to losses of energy in the surrounding area of the target zone). The design of the inlet and outlet structures as well as the splitter walls (both for the current flow and the water wave generation) are certainly crucial for this purpose.

Approach to the final design

In dealing with the above challenges the following approaches were thoroughly discussed during the preparation of the project:

To reduce the probability of failure of both failure modes independently to very low levels, in order to move them away from the failure tree of the principal modes. It was proven that for the circumstances of this project this approach was not feasible, due to: incompatibilities with construction planning, and the high costs involved in the protection. The use of higher caliber material in the scour protection easily creates construction difficulties in view of the respect of the filter rules towards the soil material. In order to accommodate construction constraints, for the design of the filters, the application of open filter layers as well as the installation of geotextiles (e.g. fixed to willow matrasses, a common practice in the Netherlands and Belgium cfr. Verhaeghe et al. 2010) and classical filter layers have been discussed.

To artificially separate geotechnical and hydraulic failure analysis by introducing the concept of a minimum geometry in the geotechnical calculations, based on alarm lines, as was the case for the breakwater in Zeebrugge. Such approach needs to be made consistent with the inspection and maintenance strategy applicable for the project. A first alarm line defines a level at which the operator can start the mobilization process of the equipment that is required to safeguard the situation before the ultimate limit line is exceeded. This second alarm line should correspond with the level of safety that has been required for this geotechnical failure mode. From the operation point of view it is essential that clear alarm line drawings are developed to support the inspection and maintenance strategy for a project.

Even though the second approach requires systematic inspections (and possible also maintenance) during the useful life of the structure, it allows for the economical optimization of the solution. In particular, it allows for setting higher failure probability for the hydraulic failure mode. In return, it requires careful estimation of failure rate as well as failure consequences (as required for the estimation of the time available for intervention after hydraulic failure of the scour protection).

Hydraulic design and verification through Level III methods

Here a simplified version of the Level III verification of the hydraulic failure mode "loss of hydraulic stability of the scour protection layer" is introduced. The objective is to show the most relevant difficulties encountered and to highlight some relevant results.

A Level III Monte Carlo simulation method is based on the simulation of several useful lives of the structure and on the verification of the failure modes for each of the simulated useful lives. Then, the expected failure probability for each failure mode is estimated as the number of useful lives that result in a failure of the given mode over the total number of useful lives simulated. Although conceptually simple, this approach requires:

(1) A statistical model for the random simulation of extreme multivariate conditions.

(2) A state equation for the verification of the failure mode.

In both cases uncertainties in models and equations must be taken into account.

In this study case the statistical model developed for the simulation of extreme conditions is as follows:

(1) Available time series of maritime agents are used to generate a time series of shear stresses. This series is used to identify points in time where the shear stress has local peaks, allowing for the identification of a multivariate time series with the combinations of maritime agents leading to extreme shear stresses (see Mazas et al. 2017 for an approach of identification of events based on one variable that measures the combined effect of several agents).

(2) A marginal mixture probability distribution is fit to every variable, composed of the empirical distribution function up to a conveniently selected threshold, chosen following Solari et al. (2017), and a generalized Pareto distribution over the threshold. These marginal distributions are used for transforming the original variables to standard normal variables (see e.g. Solari and Van Gelder 2010).

(3) A mixture of three Multivariate Gaussian distributions is fitted to the standard normal variables.

(4) A number of extreme events is simulated assuming it follows a Poisson distribution.

(5) For every extreme event the mixture of Multivariate Gaussians is used for the simulation of a new set of standard normal values.

(6) Standard normal values are transformed to original variables using the marginal mixture distributions.

By using bootstrapping techniques the uncertainties arising from the statistical model are properly accounted for; i.e. resampling with replacement from the original data for every useful live before performing steps (1) to (6).

Then, for every simulated extreme event, the occurrence of the failure mode is verified. To this end, the maximum wave and current shear stress was estimated following Soulsby (1997). However, how to include uncertainties in this case is not straightforward as no recommendation is given in the references (possibly due to the fact that these formulation were not developed for structural design, possibly due to the great uncertainties involved when dealing with sediment transport formulation, as is the case here). Thus, three approaches are implemented and compared:

a. To perform a verification using only expected values (i.e. no uncertainty is taken into account in the verification equation).

b. To use a constant Coefficient of Variation (CV) for the estimated mean diameter of the protection layer required to withstand each storm event.

c. To use a constant CV for the shear stress estimated at each storm.

In order to implement approaches (b) and (c), CVs are required. Based on Soulsby (1997), where it is stated that typical uncertainty in the estimation of mean grain size is 20% and that differences between results obtained from different methods available for estimation of shear stresses are less than 50%, we assumed that CV is in the range 0.1 to 0.2 in the case of mean grain size, and in the range 0.2 to 0.5 in the case of shear stresses.

Results obtained for one section of the breakwater (other than the change in alignment) are shown below. First, it was verified that the simulation methodology was capable of properly reproducing the observed climate. Figure 12 shows a comparison of the empirical bivariate distributions of significant wave height and total sea level (left) and significant wave height and current speed (right) obtained with the original data (in colors) and with the data simulated for one useful life (black lines). It is noted that in both cases the simulated data reproduces fairly well the behavior of the observed data. Then, mean weight (W50) required for achieving different failure probabilities were calculated, under the three hypotheses listed above. Results are summarized in Table 1. It is noted that the effects of including the uncertainty of the verification equation are significant, almost tripling required rock weight in this particular case.

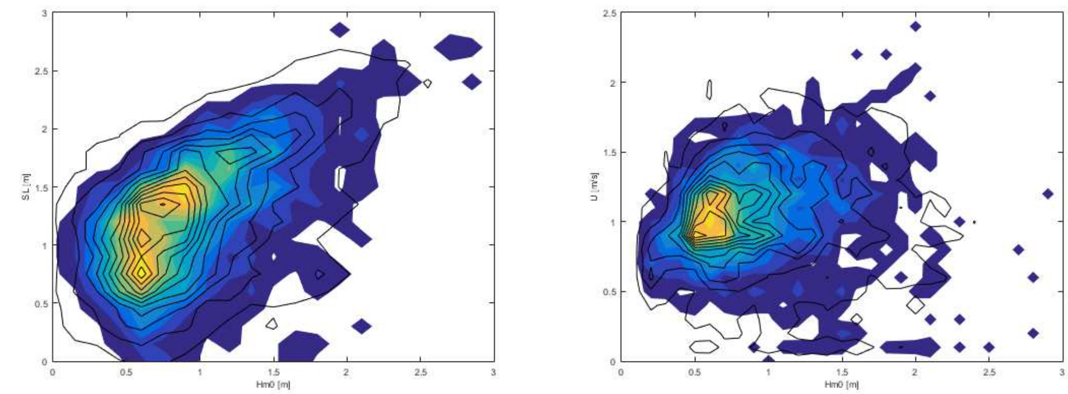


Figure 12 – Empirical bivariate distribution of significant wave height and total sea level (left) and significant wave height and current speed (right). Original data in color and simulated data (one useful life) in black continuous lines.

Table 1 – Mean weight required for assuring different failure probabilities under the different working hypothesis.

	W50 [kg]		
	Pf = 0.5%	Pf = 0.2%	Pf = 0.1%
No Uncertainty	13	20	29
CV = 0.2 in D50	19	31	46
CV = 0.2 in shear stress	31	53	79

CONCLUSIONS

At geotechnical level, the preparation of the project has shown some of the possibilities to reduce uncertainty in the design processes and to increase reliability through reliability based design. Deep understanding of the physical failure processes and the geotechnical tests (lab as well as in situ) remain necessary in order to achieve that.

Regarding the verification of the hydraulic failure modes, the lack of a widely accepted verification equation, with properly quantified uncertainty, poses a significant challenge when it comes to the estimation of the failure probability. The different methodologies considered (ad hoc) for the incorporation of some degrees of uncertainty in the verification result in such a spread of results that physical modelling of the work is practically unavoidable. This, however, is not a simple task and can only be performed properly in a limited number of experimental facilities around the world.

From a global point of view, the introduction of the concept of “minimum geometry”, that ensures geotechnical stability while allowing for larger failure probabilities of the scour protection, proved to be useful for tackling the problem. However, this approach open new questions that remain to addressed; in particular, under this approach it becomes more relevant to know the expected evolution of the scour once the scour protection layer start to mobilize, that is, it becomes particular relevant to know the scour damage evolution, as this evolution conditions the time available for performing surveys and repairing works. Again, to address this issue the available analytical and numerical tools have such big uncertainties that the only feasible solution would be to resort to scale models or take a conservative approach when defining the “minimum geometry”.

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