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Case Study: Design of Palm Island No. 1 Dubai

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Abstract

Palm Island Developers asked Royal Haskoning in the autumn of 2000 to design the crescent breakwater, surrounding their planned man-made Palm Island at the coast of Dubai, UAE. The main function of the breakwater is to protect the beaches and small marinas of the island against incoming waves from the Arabian Gulf.

A number of interesting aspects, for example the influence of incoming waves on a curved breakwater, had to be dealt with, both during preliminary and final design of the breakwater. To design a breakwater profile, which fulfilled all the client's requirements on overtopping and stability, several breakwater cross sections were examined. By studying the effect of different slopes, an underwater berm and different crest widths and the effect of a retaining wall, making use of a 2D wave flume, it was possible to design the most economic breakwater profile. This profile consists of an armour layer of 3-6 ton rock, at a slope 1:2. Overtopping water is collected on a wide crest and then transported through the core of the breakwater back to the sea. Only in extreme situations will the overtopping water pass the landward boundary of the sea defence. Special attention must therefore be paid to proper discharge of overtopping water.

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Introduction

Palm Island is a man-made island, shaped like a date palm, with a 1.5 km long trunk and 17 fronds.
The island is intended to be a recreational island, fully equipped with parks, shopping malls, marinas and luxurious housing. The construction is practically completed at the coast of Dubai by now (Figure 1).

This case study describes the design and constructional aspects of the elliptical breakwater around Palm Island, which will protect the fronds against waves. The paper focuses on the practical use of the design theory for stability and overtopping under oblique wave attack. The results of the 3D model tests, which were carried out for the Palm Island case, demonstrated that the theory of Galland (1994) is applicable for rubble mound curved breakwaters. Special attention is paid to the influence of the breakwater cross section (slope, underwater berm, wide crest, retaining wall) on overtopping quantities, based on 2D model test results.

Short Project Description

The Palm Island Project
Since Dubai will be running out of oil in the near future, new sources of income needed to be found. Dubai chose to invest in the tourist industry. In 1989 about 600,000 tourists brought a visit to Dubai and only ten years later, this number increased to 3 million. Sheikh's own words are that Dubai should be one of the most prominent tourists' destinations in the world. By building Palm Island, a prestigious project will be realised, offering thousands of tourists the opportunity to hire or buy a house in Dubai.

Site description
Palm Island No.1 is foreseen 15 km North of Jebel Ali Port at the Dubai coast in the Gulf, as can be seen in Figure 2. The palm will extend 5 km out of Dubai's coast and has a width of approximately 4.5 km.
A protecting breakwater surrounds the fronds of the palm. The trunk will be connected with the mainland by a 300 m long bridge, the crescent breakwater will only be accessible by boat.

The island points with its trunk Northwest, towards Iran. As for this position, all fronds will be attacked by waves from all directions, except South. Most severe waves, however, are coming from the Northwest.

The crescent breakwater protects the beaches on the fronds against these waves and provides relatively quiet water around the palm for all kinds of water sports, throughout all seasons.

Breakwater layout
The crescent breakwater is approximately 11 km long and 200 m wide and will be used as a recreational island itself. Two gaps are provided at the breakwater island in order to improve the exchange of water around the fronds of the palm with the open sea. Recreational sailing vessels will also use these gaps to enter or leave the Palm area.

In order to experience the Gulf in full from each position of the Palm Island, it was the client's desire to keep the breakwater crest at a level of MSL +3.0 m. The seaside of the breakwater is protected by rubble mound armour; the inside will be used as a beach and therefore consists of unprotected sand.

Figure 1. Satellite photograph on 9 February 2003.

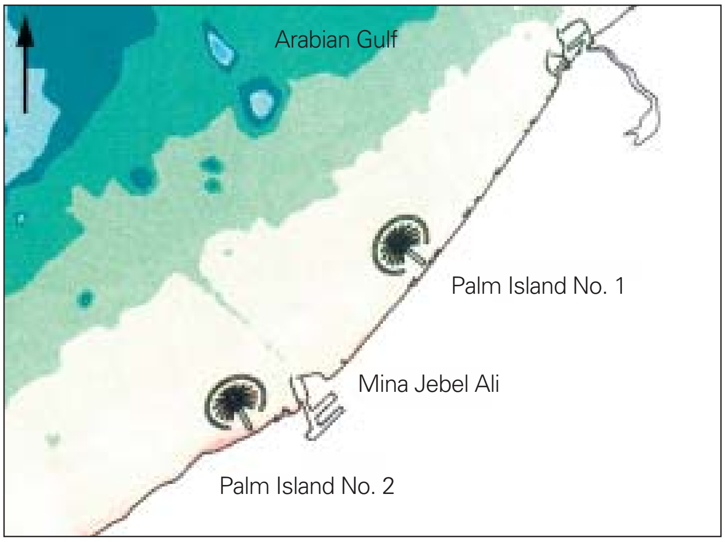


Figure 2. Location of Palm Island at Dubai coast.

Breakwater Hydraulic Design Aspects

Water levels
The extreme water levels for Palm Island occur as a result of tide and storm surges. The astronomical tide along the Dubai coast is semi-diurnal, with a strong diurnal component, leading to a difference of about 0.3 m between the two daily high waters. The tidal range shows little variation along the coast and for the tidal levels at Palm Island the values for Jebel Ali Port can be adopted, as given in the Admiralty Tide Tables.

This spot of the Dubai coast is attacked by the Shamal, which is a yearly recurrent storm event, developing over the Northwestern part of the Gulf and moving then Southeast in the course of a few days.

Table I. Water levels at Jebel Ali Port with respect to MSL.

Return period (years)	1	10	25	50	100	250	1000
Water level (m MSL)	0.85	1.12	1.25	1.34	1.43	1.55	1.75

The surge levels for Shamal storms were determined by WL|Delft Hydraulics (2001) based on three historic events. One of these storms reached wind speeds and wave heights that were far above those of all other storms, which occurred in the period 1963-1993. The return period of this storm was assumed in the order of 1/1000 year.

The design water levels (in m above MSL) for combined surge and tide are given in the table above (Table I).

Depth influenced wave height distribution

The 1/100 year significant wave height due to Shamal waves at deep water is approximately 6.0 m. Because of shoaling effects, the wave height decreases to approximately 4.0 m at the outer point of the breakwater.

The foreshore water depths should also be taken into account when determining the wave height along the breakwater. For Shamal waves, the significant off-shore wave height divided by the water depth is larger than 1/20, resulting in a transitional-water effect. This will lead to a levelling of the Rayleigh Distribution for these waves. From the computations by WL|Delft Hydraulics (2001) it is concluded that for the Shamal design conditions 'depth limited waves' will occur.

For depth limited waves, the following relation is given by Van der Meer (1994):

$$R_{2\%} = 1.4[1\text{-}0.03\cdot(1\text{-}1/\gamma)]^2 \quad (1)$$

$$R_{2\%} = H_{2\%}/H_s \quad (2)$$

For Shamal waves, the breaking ratio is $\gamma = 0.3$ to 0.35 (where $\gamma = H_s/h$), which leads to a relation of $H_{2\%}/H_s = 1.35$, instead of a "normal" deep water relation of $H_{2\%}/H_s = 1.4$.

The locally generated wind waves (other directions than Shamal) have a lower significant design wave height and are therefore more or less deep water waves and thus $H_{2\%}/H_s = 1.4$ holds.

Design wave climate

Owing to its crescent shape, each part is attacked by several waves, all with different heights (and corresponding distribution) and different angles of incidence. The wave climate is thus not uniform along the crescent breakwater. In Figure 3 for several locations along the breakwater the design wave directions and wave heights are indicated.

As can be seen in Figure 3, waves during the Shamal will not be of any influence for the breakwater sides facing the coast (points X, Y, G and H). For these parts, the lower local waves are governing. For the intermediate parts, the breakwater is attacked by both almost parallel Shamal wave fronts of approximately 4.0 m height and various more or less perpendicular local wave fronts of approximately 2.5 m height.

It is known that obliquity of the incoming waves decreases the impact on the stability of the armour blocks and overtopping waves, depending on the degree of obliquity. For each part of the breakwater it must therefore be specified which of the higher oblique waves or lower perpendicular waves is governing.

This governing wave is used for the determination of the required dimensions of the armour layer blocks and maximum expected overtopping quantities.

Oblique Wave Attack

Theoretical design method oblique wave attack

The effects of oblique wave attack for toe and main armour stability and for overtopping are described by C.-J. Galland (1994).
Galland derived several formulae for defining the equivalent perpendicular wave height for oblique incoming waves. Galland carried out six series of tests, each one being defined by its angle of wave incidence (0°, 15°, 30°, 45°, 60° and 75°) for a breakwater, which was divided into four trunks and armoured with Accropodes®, tetrapods, antifer blocks and quarry stone.

For quarry stone, which is the design block for Palm Island, the tests resulted in three equations, with H_e being the equivalent wave height:

Overtopping $H_e = H_s \cdot (\cos ß)^{1/3}$ for $15° < ß < 75°$ (3)

Armour stability $H_e = H_s \cdot (\cos ß)^{0.25}$ for $ß \geq 30°$ (4)

Toe stability $H_e = H_s \cdot (\cos ß)^{0.6}$ for $ß \geq 30°$ (5)

In his discussion, Galland proposes that this approach is not fully satisfactory. First because, in some of his test results, the adjustment is not very good and second because his test results can not represent the influence of a different breakwater slope.

Application theoretical design method for Palm Island
Despite Galland's remarks on the application of his equations, it was decided to use his equation (3), (4) and (5) for quarry stone as a useful method for the preliminary design of Palm Island crescent breakwater.

As remarked in the previous chapter, the wave climate varies along the breakwater and many parts of the breakwater are attacked by waves from various directions. It was therefore necessary to define the ruling equivalent wave height for each part of the breakwater. This was done for the ten locations with known waves from several directions, as given in Figure 3.

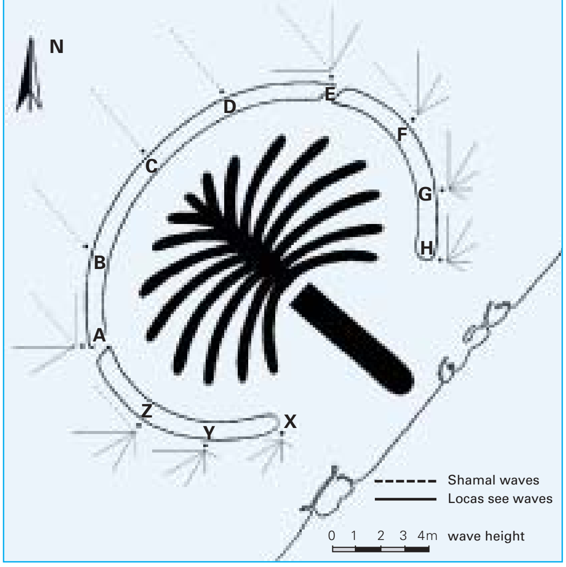


Figure 3. Wave heights at several points along the breakwater.

The required armour diameter was calculated by using Van der Meer (1993) for shallow water conditions and applying a maximum allowed damage number S=6:

Plunging
$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7 \cdot P^{0.18} \cdot \left(\frac{S}{\sqrt{N}}\right)^{0.2} \cdot \xi_m^{-0.5} \cdot \qquad (6)$$

Surging
$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4 \cdot P^{-0.13} \cdot \left(\frac{S}{\sqrt{N}}\right)^{-0.2} \cdot \sqrt{\cot\alpha} \cdot \xi_m^{P} \qquad (7)$$

Transition
$$\xi_{mc} = \left\{6.2P^{0.31} . \sqrt{\tan\alpha}\right\}^{\frac{1}{P+05}} \qquad (8)$$

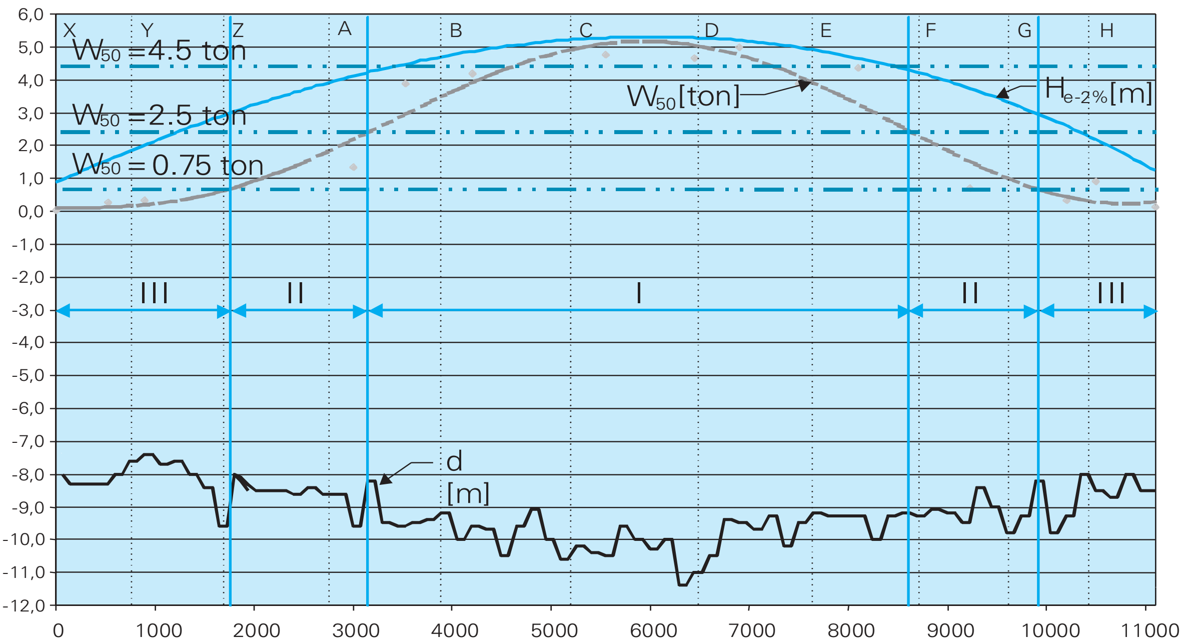
Instead of the incoming $H_{2\%}$ the theoretical "equivalent wave height" $H_{e\text{-}2\%}$ was used. This $H_{e\text{-}2\%}$ as a function of the unfolded breakwater is given in Figure 4.
Also shown in this figure is the corresponding theoretical required average stone weight $W_{50} = p_s \cdot D_{n50}^3$.
The intersection between the theoretically required average stone weight and the mean weight of a selected stone class (i.e. 0.75 ton, 2.5 ton and 4.5 ton) marks the transition to a heavier grade for the armour layer elements on the breakwater.

In this way a preliminary armour design for the crescent breakwater was made, dividing the breakwater into five longitudinal sections with different armour elements.

The most severely attacked section (Section I: 5900 m) is covered with 3-6 ton rock elements (W_{50} = 4.2 - 5.4 ton). The East and West end-parts (Section III: in total 2975 m) are both covered with 0.5-1 ton rock elements (W_{50} = 0.64 - 0.80 ton). The intermediate parts at the Eastside and Westside (Section II: 2225 m) are covered with 1-4 ton rock elements (W_{50} = 2.2 - 2.7 ton).
Both secondary armour layer material and toe material are adapted adequately to these primary layers.

A typical cross section for the most severely attacked part of the breakwater is given in Figure 5.

Figure 4. Equivalent wave height ($H_{e\text{-}2\%}$) and corresponding mean rock weight (W_{50}) as a function of the unfolded breakwater.



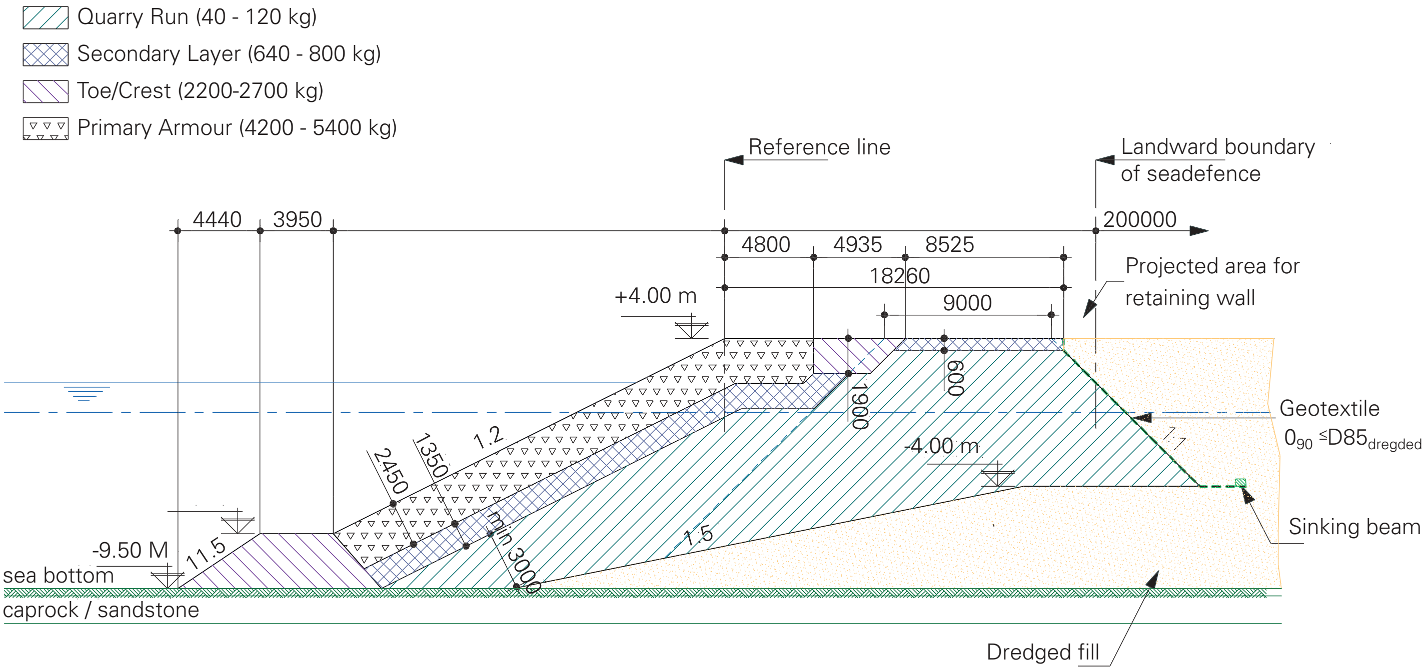


Figure 5. Typical cross section for Section I (4,750 m – 7,700 m).

3D Model tests and results

The method of Galland was used as an instrument to prepare the preliminary design of the breakwater. In order to investigate the limitations of this method and to check stability and restricted overtopping quantities, several 3D model tests were carried out.

These tests were performed in a 32 x 20 x 1.2 m wave basin in Grenoble. This basin is equipped with a wave generator capable of producing regular or random waves. Wave absorbing areas were placed on the edges of the basin to neutralise the wave reflections. The dimensions of the wave basin limited the scale of the model sections. With help of Froude's law the scale of the sections was set at 1:40.

Two model sections (model A and model B) were defined, in order to test overtopping and stability for both Shamal waves and locally generated wind waves. For model A, waves from 310, 320 and 330 degrees North were generated. For model B, waves from 320 and 360 degrees North were generated. Owing to the curved layout of both sections and their different position towards the wave panels, the generated waves had an angle to the normal of the breakwater between 10 and 80 degrees. In model B a transition of 3-6 ton armour layer elements to 1-4 ton armour layer elements was included.

The duration of each test was 5 hours prototype time. Damage to the breakwater was given as a damage number and as a percentage. The damage number, as a function of the measured number of displaced rocks, was given by:

$$S = 1.855 \cdot N_{displaced} \cdot D_{n50} \qquad (9)$$

In order to compare the test results to the theoretical expectations, the expected damage number S, according to Van der Meer, was determined for the equivalent wave height H_e, according to Galland's theory, for each tested angle to the breakwater normal. For that purpose, (6) and (7) were rewritten to:

Plunging $$S = \left(\frac{H_e}{\Delta D_{n50}} \cdot \frac{1}{\xi_m^{-0.5} \cdot 6.2 \cdot P^{0.18}}\right)^5 \cdot \sqrt{N} \qquad (10)$$

Surging $$S = \left(\frac{H_e}{\Delta D_{n50}} \cdot \frac{1}{\xi_m^{P} \cdot \sqrt{\cot\alpha} \cdot P^{-0.13}}\right)^5 \cdot \sqrt{N} \qquad (11)$$

The comparison was made for all sections with armour layer material 3 to 6 ton (Model Section A and part of Model Section B). For a large number of comparable tests the occurred damage number S is plotted in Figure 6, together with the theoretical damage number S for the equivalent wave height H_e, given by (10) and (11). The H_e for rock stability is characterised by Galland in (4). In Figure 6, also the 90% confidence levels are drawn, defined as $2 \cdot \sigma$ for all parameters in (10) and (11) and σ according to Van der Meer (1993). Approximately 65% of the measured damage numbers fall inside these 90% confidence levels of the theoretical damage number S.

It should be noted that the results for low damage numbers and short wave periods (approximately 6.3 seconds) fall outside the given 90% confidence levels. This could probably be the effect of the factor x in (3, 4 and 5). For Palm Island breakwater, these relative low damage numbers are of less importance, since the design must fulfil for the larger maximum accepted damage number S = 6.

The 3D model test results as carried out for Palm Island breakwater show that the same conclusion as Galland draws in his publication can be drawn: for increasing angles of incidence, some trend is noticeable that indicates an increasing stability, especially for higher damage levels.

Although not shown in this publication, the same conclusions can be drawn for overtopping quantities.

For higher damage numbers and not too short wave periods (Tp ≥ 6.3 s), Galland's formulae are very well applicable. However, if conditions permit, 3D model tests are recommended to check stability and overtopping quantities of a new curved breakwater design problem.

Overtopping

Design criteria overtopping

The client's requirement was a breakwater with a crest height at the same level as Palm Island, i.e., MSL +3.0 m. The 1/100 year water level, owing to Shamal surge at MHHW, is MSL +1.43 m. The 1/100 year significant wave height for Shamal waves is H_s = 4.0 m, giving considerable overtopping during these design conditions. A basic approach for defining overtopping is given by Owen (1995), using a non-dimensional freeboard parameter and a non-dimensional average specific discharge:

$$Q^*_p = a \cdot \exp(-b \cdot R^*_p / \nu) \tag{12}$$

$$R^*_p = R_c / H_s \cdot \sqrt{(s_p / 2\pi)} \tag{13}$$

With ν a correction factor for various influences and *a* and *b* coefficients depending on the breakwater slope and R_c the difference between crest level and water level.

For Palm Island crescent breakwater, this results under design conditions (t_r = 100 year) in an overtopping quantity at the outside, seaward crest in the order of 1500 l/s/m^1. Without additional measures this will result in severe damage on crest, near buildings, vehicles and large risks for pedestrians, according to Van der Meer (1993).

Apart from the not allowed rising of the crest, a number of methods are known to reduce the overtopping quantities, such as reduction of the slope, application of an underwater berm and application of a retaining wall at the transition of the breakwater to the backfill. Another possibility is to apply a wide crest on which overtopping is drained. Owing to the need to reduce the overtopping to acceptable quantities at the landward boundary, it was decided to carry out several 2D model tests, in order to define the effects of each of these solutions. In consultation with the client, it was decided to accept 20 l/s/m^1 at the landward boundary of the sea defence for 1/100 year wave conditions.

2D model tests and test results

In total nine test series were carried out for nine different cross sections, each test series consisting of a number of different combinations for water level and significant wave height, modelling as such the prototype 1/1, 1/25, 1/50 and 1/100 year wave height condition. The 2D model was built for the most severely attacked section of the breakwater, attacked by (almost) perpendicular Shamal waves.

Although armour layer stability and toe stability as well as overtopping were studied during these tests, most attention was paid to the overtopping aspects, being one of the most important design parameters.
By comparing the results of several tests with specific cross section characteristics, it was possible to investigate the effect of certain modifications on overtopping.

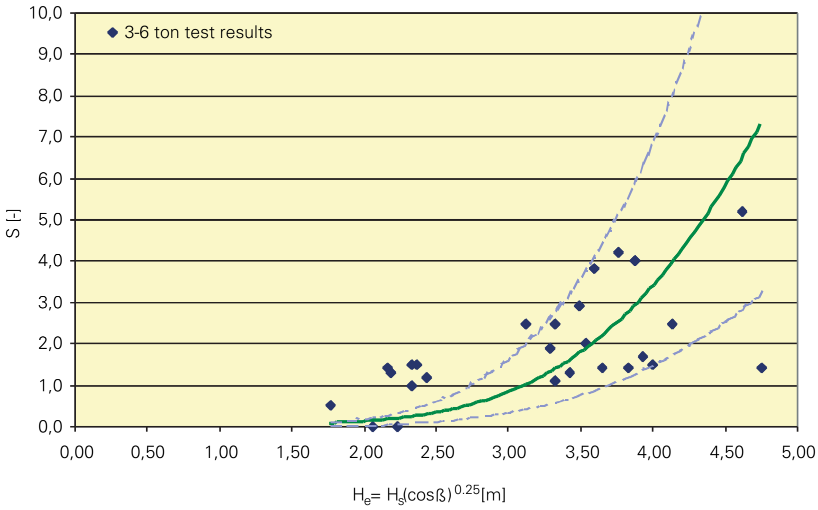


Figure 6. Comparison of 3D model test results with Theory of Galland.

Table II. Effect of cross section characteristics on overtopping quantities at landward boundary.

Characteristics	$Q_{1/10\ year}$ [l/s/m^1]	$Q_{1/100\ year}$ [l/s/m^1]
Slope		
1:2.5	-	5
1:2.0	-	8
Crest level		
MSL +3.75 m	0	2
MSL +3.00 m	2	23
Retaining wall at landward bnd		
wall of 0.75 m	-	13
no wall	-	26
Crest width		
30 m	1	8
20 m	0	26
Berm		
no berm, but wide crest	3	12
10 m berm at water level	4	44

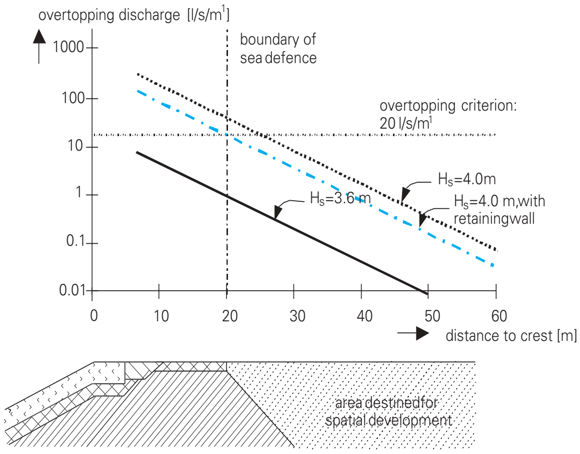


Figure 7. Overtopping discharge as function of the crest width, with and without retaining wall.

The effect of these modifications is summarised for both 1/10 year prototype condition and 1/100 year prototype condition (see Table II). The overtopping quantities in this table are given for the landward boundary of the sea defence. Modifications, other than described (such as sea level rise, or a different material choice), have been discounted in the results presented in this Table.

Most of the conclusions, which can be drawn from the results in Table II are not new or unexpected:

- a steeper slope will result in more overtopping;
- a higher crest level will result in less overtopping;
- a retaining wall will reduce the overtopping with nearly 50%, depending on its location on the crest;
- a longer crest will reduce the overtopping at a certain "landward" boundary of the sea defence.

The model tests proved also that, for this special case, a breakwater with a berm at water level is not so effective as a breakwater with a wider crest, when overtopping is examined. Next to that, it was concluded that the cross section of a breakwater with a wider crest uses less material than a breakwater with a berm at water level for the same order of overtopping quantities.

By applying a wide crest, however, special attention must be paid to the rock elements on the crest.
The model tests showed that these rocks must be of sufficient weight over a relatively long stretch of the crest. Otherwise these rocks will be transported over the crest by the strong currents of the overtopping waves, causing increasing damage.

Conclusions

Based upon the results of the 2D model tests, it was concluded that a breakwater with a wide crest is the most economical solution for the crescent breakwater.

Since the armour layer of 3-6 ton armour rocks showed stability results conform expectations for a slope 1:2.0, it was decided to select this slope for the breakwater. The crest level was, in close consultation with the client, chosen at MSL + 3.0 m. The crest width, now, had to be determined for each section of the breakwater, taking into account the maximum accepted overtopping quantity of 20 $l/s/m^1$. By comparing the results of overtopping tests for the same breakwater cross section and the same incoming wave climate, but several positions at the crest, it was possible to compose Figure 7.

For each incoming wave height (Shamal or local wave direction), the overtopping at the seaward boundary for the equivalent wave height, determined by (3), was estimated with help of (12) and (13), verified by the results of the 2D and 3D model tests. The required crest width, limiting the overtopping to maximally 20 $l/s/m^1$ for each section, was determined by assuming the same logarithmic reduction in overtopping quantities as found in Figure 7.

The maximum crest width in this way determined was found at the part of the breakwater, which is attacked by nearly perpendicular Shamal waves. The maximum crest width is 20 m, when a retaining wall of 0.75 m is present. The minimum crest width, at the East and West outer-ends, is not determined by overtopping criteria, since hardly any overtopping will occur for the low local waves at these spots. The minimum crest width is 4.5 m, which is determined by constructional criteria.

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