Design of Revetments in the Öresund Link.
Reclaimed Artificial Island and Peninsula

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Abstract

The basic and detailed design of revetments and breakwaters in the Öresund Link were carried out by the Consultant, Carl Bro a/s[4] for the Contractor, Öresund Marine Joint Venture, ÖMJV[5], based on design requirements and additional background information given by the Owner, Öresundskonsortiet, ÖSK. Final design was to be approved by the Owner, but with the liability of the Consultant/Contractor and with due consideration to the contractors method of construction. As part of the design process and in order to fulfil given geometrical restrictions the design requirements where further elaborated based on results of physical model tests and desk studies of critical assumptions. Some of the aspects and considerations in that process are discussed below.

Introduction

The Öresund Link creates a fixed link from Denmark to Sweden. As part of the Link, a 4 km long artificial island, named Peberholm, south of the island Saltholm and a Peninsula east of Amager have been constructed, see the overall layout in Fig. 1. The Link carries both rail and road traffic, from the Peninsula to the new island in a 3.5 km immersed tunnel, and from the island to Sweden on a 7.8 km double deck bridge with cable-stayed central spans. The Peninsula includes approx. 3.5 km permanent revetments. The artificial island includes approx. 8.5 km of permanent revetments and 1 km of semi-submerged breakwaters at the eastern and western ends. Further, approximately 2 km of temporary revetments and breakwaters were designed and constructed along with 3 temporary, sheltered and 3 unprotected work harbours.

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A total of approximately 8.000.000 m^3 of dredged material has been placed in the reclaimed areas behind permanent revetments and breakwaters consisting of approximately 650.000 m^3 coarse pebbles, 130.000 m^3 filter stones and 250.000 m^3 armour stones.

A brief description of the design requirements and calculations for design of typical cross sections of the revetments in the 14 km coastal structures is given below. The discussion is mainly on wave heights and the correlation with high water level used in the design of armour layer and in calculations of overtopping. Also the formal proof of the stability of the toe and scour protection is discussed in further detail.

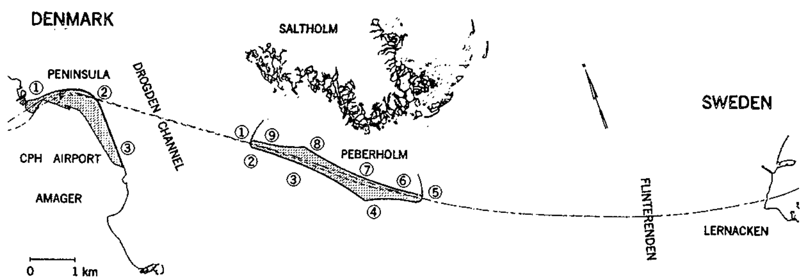

Figure 1. Layout of the Öresund Link - Reclamation Works

As Consultant for design Carl Bro a/s carried out the tender, basic and detailed design of revetments and breakwaters for the Contractor Öresund Marine Joint Venture, (ÖMJV) based on design requirements given by the Owner, Öresundskonsortiet, (ÖSK). Desk studies and physical model tests revealed that the design requirements had to be re-evaluated to make the final structure fulfil geometrical requirements also given by the Owner.

Design Requirements

Significant wave heights and water levels with a 10, 100 and 10,000 year return period where given for specific locations in the Öresund area as design requirements. The wave data were given for two positions on the alignment of the Link (one in the Drogden Channel and one in Flinterenden). Also a set of data for Drogden lighthouse situated approx. 8 km south of the link was given. The requirements were based on field data and numerical wave and water level modelling (MIKE 21 NSW and MIKE 21 HD) performed by Danish Hydraulic Institute, DHI. The numerical modelling was carried out with no island or peninsula present.

Further results from the study done by DHI were available as Background Information, and wave climate data reduced due to friction and wave breaking over the shallow areas was given along the entire length of the Link alignment as well as for 6 points south of the future artificial island. From this a wave climate depending on water depth was developed and used in the Tender Design, see Table 1 below.

The above described approach for determining the design wave heights was not accepted by the Owner and for the basic and detailed design of the armour layer more conservative wave heights were specified in order to reduce the risk of damages. Data from "deep" water were to be applied unreduced on the revetments and breakwaters according to the criteria given below. The results from the numerical modelling were only accepted by the Owner for design of the northern perimeter of the reclaimed island. Also the criteria $H_s < 0.6h$, where H_s is the significant wave height and h is the local water depth at design water level, was given by the Owner, but due to large water depth not applied.

The basis for design requirements for wave heights applied in the final design can be summarised by the following :

Peninsula (P1-P3)	:	Wave data from Drogden Channel
Western part of the Island (I1)	:	Wave data from Drogden Channel
Southern perimeter (I2-I4)	:	Interpolation between Drogden and Flinterenden
Eastern part of the Island (I5)	:	Wave data from Flinterenden
Northern perimeter (I6-I9)	:	Results from numerical model increased by 1.15

The positions at the island, I1-I9 and peninsula, P1-P3 are indicated in Figure 1 and the wave heights used for designing armour stones etc. are listed in Table 1.

Location		Tender Design		Final Design (Armour layer)		Final Design (Overtopping)	
		H_s (m)	HWL (m)	H_s (m)	HWL (m)	H_s (m)	HWL (m)
Peninsula	1	1.5	+1.00	1.6	+1.50	1.4	+1.35
	2	1.5	+1.50	1.6	+1.50	1.4	+1.35
	3	1.5	+0.50	1.6	+1.50	1.3	+1.15
Island	1	2.0	+1.50	1.7	+1.50	1.5	+1.35
	2	2.0	+0.50	1.8	+1.50	1.5	+1.10
	3	2.0	+0.50	2.0	+1.50	2.0	+1.00
	4	2.0	+0.50	2.5	+1.50	2.2	+0.90
	5	2.0	+1.50	2.5	+1.50	2.2	+1.30
	6	1.2	+1.00	1.1	+1.50	1.1	+1.35
	7	1.2	+1.00	1.3	+1.50	1.2	+1.35
	8	1.2	+1.00	1.1	+1.50	1.0	+1.35
	9	1.5	+1.00	1.1	+1.50	1.0	+1.35

Table 1. Wave heights and High Water Levels (100 year return period). Water levels include expected level increase of 0.15 m over 100 years.

The water level in the Öresund region is governed by wind and barometric pressure, and tide is only limited. The Öresund link is constructed on a sill, that creates a dividing line between the salty waters in Kattegat to the north and the brackish water of the Baltic Sea to the south and east. A large difference in water level can occur across the sill, and the exact water level at the Link is difficult to determine.

The design requirements stated that if no other studies were performed the conservative wave heights should be used fully correlated with maximum high water level in the design and especially for calculations of overtopping. The correlated high water levels used in the Tender Design were based on data as presented in Background Information. Figure 2 below indicates a correlation between high water levels and waves from northerly directions, while this does not seem to be the case for waves from southerly directions. Due consideration was also given to the high risk sections around the tunnel portals and the abutment area (point P2, I1 and I5 in Figure 1) by using full correlation between maximum wave heights and water levels.

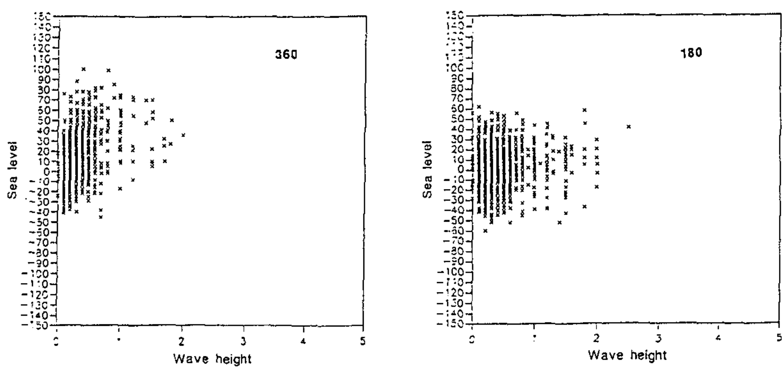

Figure 2. Wave heights and correlated water levels. Data from 1962-1984 (Waves from Northerly (360) and southerly (180) directions)

This however resulted in an unacceptable large volume of predicted overtopping that theoretically would endanger the overall stability of the structure. Hence a detailed statistical investigation was carried out for data on observed water levels and wave heights at Drogden Lighthouse 1962-1984 in order to establish a more accurate description of the correlation. The data was sorted according to wave height and high water level and curves of correlation could be determined, cf. Figure 3 and 4 which include selected results. The limited period of observation meant that a full set of design curves could not be evaluated, and an engineering approach in determining the curves for large wave heights was accepted by the Owner. The less conservative wave heights found in the numerical modelling were specified by the Owner to be used for the calculation of the overtopping, and with a given wave height the correlated high water level was determined. This resulted in the design wave heights and water levels also listed in Table 1. It must be noted that the high water levels listed in Table 1 includes a general increase of 0.15 m of the water levels in the Öresund region over 100 years.

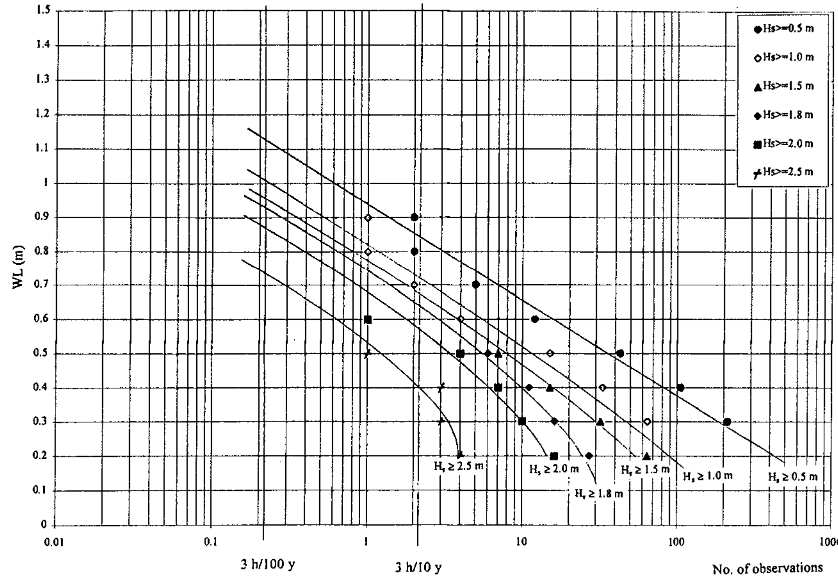

Figure 3. Correlation between water level and wave height
(Waves from southerly directions, based on data from 1962-1984)

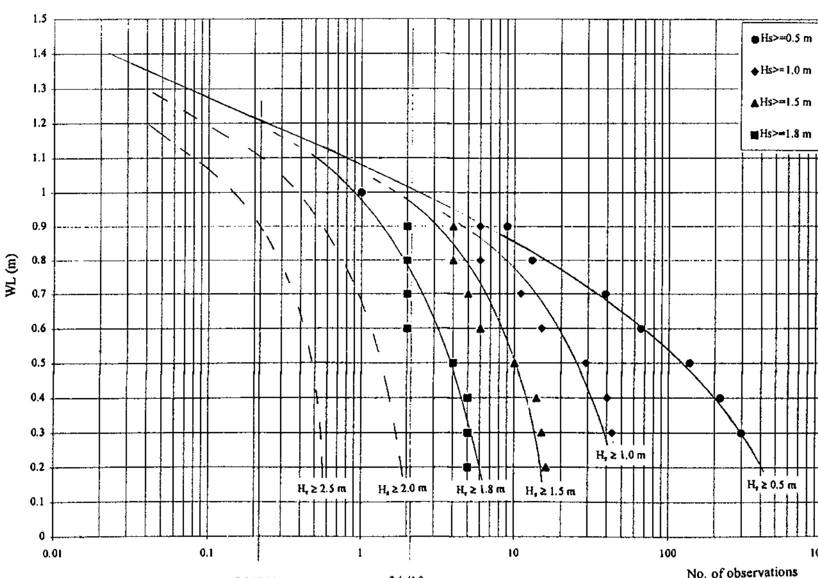

Figure 4. Correlation between water level and wave height
(Waves from northerly directions)

Typical cross sections

In Figure 5 and Figure 6 two typical cross sections of revetments in the Öresund Link are given. Figure 5 shows a normal cross section with the filter stone layer extended behind the crest as a protection against overtopping. Figure 6 illustrates the design adopted at the tunnel portal and abutment area. The vertical wall was part of the geometrical requirements given by the Owner as part of the design basis.

The core of the revetment consists of coarse pebbles ($W_{50\%}$ = 1.0-3.5 kg) which were specified in the design requirements. The filter stones ($W_{50\%}$ = 16-80 kg) follow the filter criteria towards core and armour layer. As already noted above the Owner had specified the interface between backfill and core to be multiple layers of sand, which are difficult and costly to place. In order to greatly simplify and reduce the number of operations the Owner agreed to substitute the layers of sand with a geotextile, which could be proven to have the same effect.

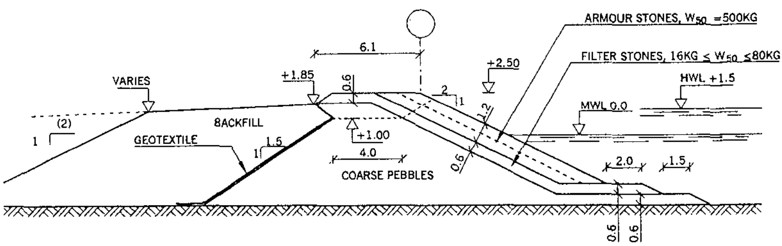

Figure 5. Typical cross section. Low crest

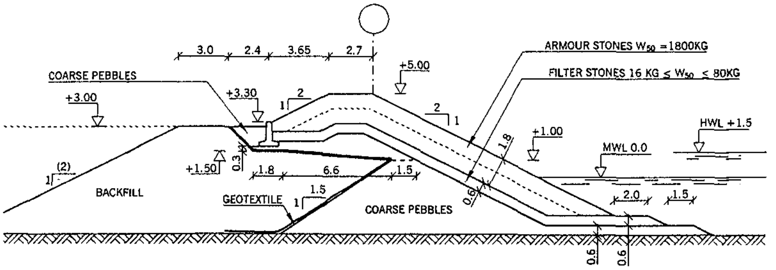

Figure 6. Typical cross section. High crest.

As an important part of the design basis for armour layer, crest, toe and scour protection a series of physical model test were performed for the revetments and breakwaters. From the results simple design criteria could be found.

Armour stones

The formulae given in (Meer, 1987) were approved by the Owner in the design requirements for calculating the armour stone size. Further the damage level was defined as the damaged cross sectional area relative to the total cross sectional area of armour stones. This gives a non-logical dependency between armour stone size and water depth / crest level. Theoretically a higher crest or deeper water required smaller armour stones for identical wave climates. In stead the results of the physical model tests were used to establish design criteria for the stability number, N. Δ is the relative density and $D_{n,50\%}$ is the nominal diameter of the armour stone.

$$N = H_s/\Delta D_{n,50\%} = 1.45 \text{ for damage level of } 2\ \% \tag{1}$$
$$N = H_s/\Delta D_{n,50\%} = 1.85 \text{ for damage level of } 5\ \% \tag{2}$$

The design requirements specified that the armour stone size for the sections at the tunnel portal and at the abutment were designed for 2% damage in a wave climate with 100 year return period and for 5% damage in a 10,000 year return period. The remaining permanent revetments and breakwaters are designed for 5% damage for a 100 year return period.

The armour stones were grouped in a limited number of stone classes in agreement with the Contractor, but due to repeated changes in design requirements throughout the construction phase this number had to be increased. A list of the armour stones used is given in Table 2, where the locations refer to Figure 1.

Location		H_s (m)	$W_{50\%}$ (kg)
Peninsula	1	1.6	500
	2	1.6	1,000
	3	1.6	500
Island	1	1.7	1,800
	2	1.8	1,000
	3	2.0	1,000
	4	2.5	1,800
	5	2.5	3,700
	6	1.1	1,000
	7	1.3	500
	8	1.1	300 / 200
	9	1.1	1,000

Table 2. Armour stone size with corresponding 100 year wave height.

The armour stones on the northern perimeter have been designed to withstand a temporary situation (5% damage for a 10 year return period) without the low crested breakwaters, the Whiskers, at the eastern and western tip of the island. Further it can be noted that the temporary revetments and breakwaters protecting the compounds used as a construction site have been constructed from materials that can be reused in the construction of the whiskers ($W_{50\%}$ = 1,800 kg armour stones).

Toe and scour protection

Initially the toe and scour protection was designed based on experience and a large redundancy was built into the structure. The toe structure was questioned by the Owner, and a detailed investigation was carried out based on the results of the physical model tests. Design curves are shown in Figure 7, where H_s is the significant wave height, N is the stability number and h_t is the water depth over the toe or scour protection.

The physical model tests showed a general smoothing of the edge of the coarse pebbles and minor damage (less than 5%) to the layer of filter stones. Both layers were at least dynamically stable and the overall stability was not endangered in any of the tests. It was accepted by the Owner, that the curve obtained for the filter stones could be used as a design curve for both toe and scour protection. Based on results presented in (CIRIA/CUR, 1991) and other previous studies the design curve was then extended to be valid for $h_t/H_s > 0.6$. Also H_s could be limited by $H_s = 0.5h$ in stead of the criteria $H_s = 0.6h$ used for the design of the armour layer.

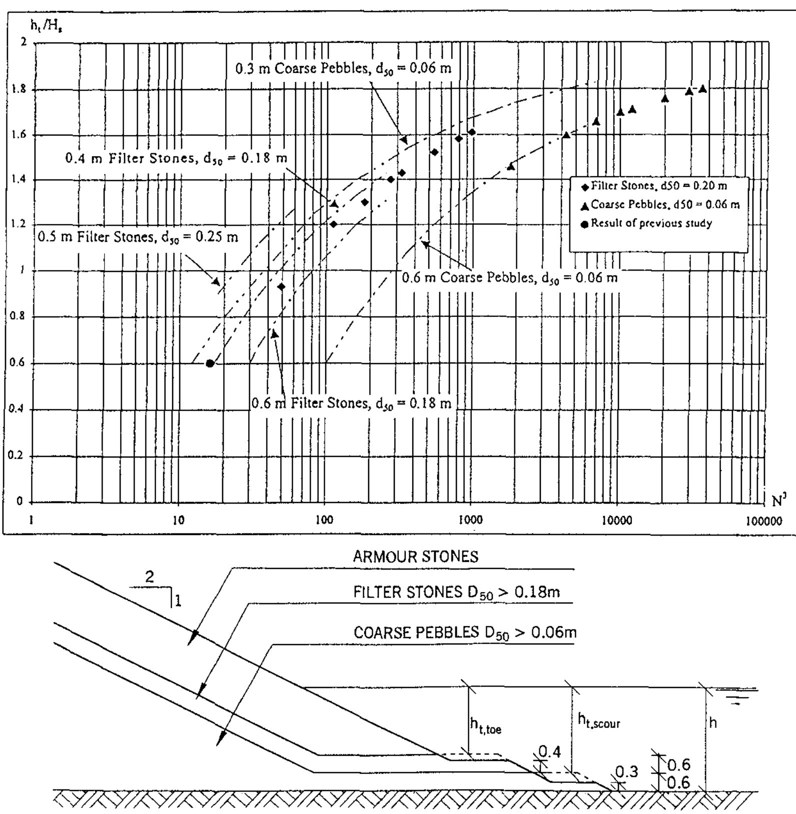

Figure 7. Design curves for toe and scour protection

As can be seen from Figure 7 a toe with 0.6 m layers of filter stones on coarse pebbles including a large redundancy could not be proven theoretically stable due to the shallow water over the toe. By including a smoothing, where the layer thickness was allowed reduced (0.6 m to 0.4 m for filter stones in the toe and 0.6 m to 0.3 m for the coarse pebbles in the scour protection), the structure could be proven theoretically stable for the following water depths, h :

0.4 m Filter stones in toe stable for	$h > 1.45$ m
0.3 m Coarse pebbles in scour protection stable for	$h > 0.45$ m

The Owner then accepted a smoothing of the toe in general to a slope of 1:6, and by using (Meer, 1987) the toe and scour protection could be proven theoretically static stable for the following water depths, h :

Filter stones in toe stable with slope 1:6 for	$h < 1.20$ m
Coarse pebbles in scour protection stable with slope 1:6 for	$h < 0.35$ m

No formal proof of the static stability of the toe and scour protection could be obtained for the intermediate water depths, but the Owner agreed that there is no reason why the structure should be especially unstable for these water depths.

A study of the sediments in the seabed in the area showed mainly coarse materials with limited chances of extensive scour in spite the increase in current speed due to concentration around the island and peninsula. On sections with a large ratio of sand in the seabed the toe and scour protection was extended by 1 m to a total of 3.0 m width of the toe.

Hence the initial design of the toe and scour protection was finally accepted with only minor increase of the toe width on exposed sections, but as a part of the accept, the Owner specified a monitoring programme commencing after handover.

Overtopping

The initial theoretical calculations of overtopping in the Tender Design showed an unacceptable amount that would endanger the overall stability of most sections of the revetments. Based on the physical model tests a formula for calculating the mean volume rate of overtopping, Q_m without correction for the influence of wind was derived by using the structure of a formula by Bradbury et. al. as given in (CIRIA/CUR, 1991).

$$Q_m = gH_sT_z \cdot 2.8 \cdot 10^{-5} \cdot (\Delta h/H_s)^{-7} \qquad (3)$$

Using this formula the overtopping volumes were recalculated using maximum wave heights correlated with maximum water levels. This also showed an unacceptable amount of overtopping, and a the crest levels were recommended raised. The physical model tests indicated a recommended freeboard of not less than $\Delta h = 1.35H_s$ for an overall stable structure. The crest levels were fixed by the Owner, and hence the Owner initiated discussions on the correlation between waves and high water levels. Finally agreement was reached on design criteria with less conservative wave heights and correlated water levels as given in Table 1. This reduced the overtopping significantly but additional protection of the crest was still necessary. Especially on the southern perimeter of the island, where the crest is down to +3.0 m, a layer of filter stones behind the crest of armour stones is placed.

The Owner further granted that the Contractor would not be responsible for overtopping causing flooding of building sites or erosion of the unbound service road / backslope of clay till bunds.

As an indication of the theoretically calculated amounts of overtopping for different sections of the revetments a list is given in Table 3 for a wave climate and high water level situation with a 100 return period. The volume is measured behind the crest of armour stones.

Location		H_s (m)	HWL (m)	Crest level (m)	Q_m (m^3/h/m)
Peninsula	1	1.4	+1.35	+2.50	54.0
	2	1.4	+1.35	+3.25	2.0
	3	1.3	+1.15	+2.50	19.0
Island	1	1.5	+1.35	+5.00	0.06
	2	1.5	+1.10	+5.00	0.04
	3	2.0	+1.00	+3.00	23.0
	4	2.2	+0.90	+3.00	35.0
	5	2.2	+1.30	+5.00	0.9
	6	1.1	+1.35	+3.00	0.9
	7	1.2	+1.35	+3.00	1.6
	8	1.0	+1.35	+3.00	0.4
	9	1.0	+1.35	+3.00	0.4

Figure 3. Calculated overtopping for situation with 100 year return period

The permanent structures should also withstand a situation with a 10,000 year return period, where the volumes of overtopping are 10-50 times as large. Again the overall stability could be endangered if the backfill is eroded, but the Owner accepted liability for this failure mode and for erosion of the service road behind the revetment.

Conclusion

The revetments and breakwaters in the Öresund Link were described in design requirements given by the Owner, ÖSK, including certain requirements for loads, layout, geometrical dimensions and levels as well as specifications for materials used. The method of construction used by the Contractor, ÖMJV also gave rise to certain requirements for the detailed design carried out by the Consultant, Carl Bro a/s.

Mainly the design is based on general design criteria for armour stone size, design of toe and scour protection as well as a critical crest level, all evaluated from a series of physical model tests. The design of some elements in the revetments and breakwaters were more or less given through requirements in the contract, but it is noted that multiple layers of sand as a filter towards the back fill were substituted by a geotextile.

The input data as regards wave characteristics and water levels were specified by the Owner based on a conservative evaluation of the numerical modelling of the Öresund region for design of armour stones. Hence an additional factor of safety was included in the design of the armour layer.

Full correlation between waves and high water levels combined with requirements to geometry, lay out etc. given by the Owner, theoretically resulted in an unacceptable large volume of predicted overtopping endangering the overall stability of the structure. Correlation curves were derived based on a detailed statistical investigation of observed wave heights and correlated water levels. The Owner then specified less conservative wave heights for the evaluation of overtopping, but still additional protection of the crest was found necessary. The Owner then agreed that the Contractor would not be responsible for damages to areas behind the revetments due to overtopping.

The initial design of the toe and scour protection was questioned by the Owner, but formal proof of static stability based on physical model tests, could be made for most water depths by including a smoothing of the large redundancy built into the structure. Interestingly the large redundancy theoretically made the toe and scour protection less stable due to a reduced distance to the surface. An engineering approach was accepted for intermediate water depths but a monitoring programme was required by the Owner.

References

CIRIA/CUR, 1991 : Manual on the use of rock in Coastal and Shoreline engineering, CIRIA special publication 83 cue report 154.

Meer, J.W. van der 1987 : Stability of Breakwater Armour Layers - Design Formulae