COST COMPARISON OF BREAKWATER TYPES

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Abstract

A cost comparison have been made between various types of breakwaters for a fictitious situation in waterdepths to a local maximum of -15 m and an adopted sea climate. The comparison is based on optimal total project costs, being the sum of costs of construction and capitalized damage during the service period. For these selected conditions the caisson breakwater and the rubble mound type provided with a single concrete armour layer appear most attractive. Composite type of breakwaters seem advantageous for water depths approximately below -20 m.

Introduction

It is remarkable that caisson breakwaters are widely found in coastal areas of Japan and Italy and far less elsewhere. Conventional rubble mound breakwaters, provided occasionally with concrete armour top layers, are far more numerous in western industrial countries. It may be questioned whether the selected type may also result from a rational design approach.

The aim of this paper is to provide a brief review of various types of breakwater which are most fit for a location on the basis of economical considerations with regard to the type of harbour, the site conditions and the service time of the construction.

Approach

For a fixed layout the costs of following breakwaters are compared:

* Conventional rubble mound breakwater
* Bermbreakwater
* Rubble mound b.w. + toplayer of concrete units: Cubes, Tetrapods and Accropods
* Caissons breakwater
* Composite breakwater

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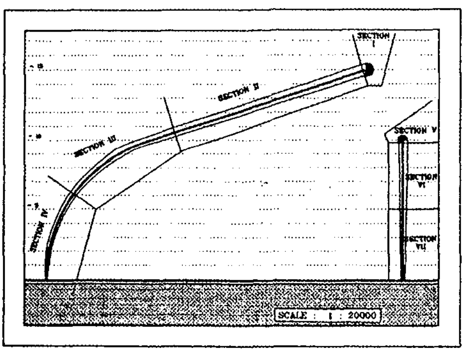
The conventional rubble mound breakwater and the bermbreakwater had previously been compared for a fictitious layout by Hauer (et al,1995). The same conditions have been used for caisson and composite breakwaters (Schols, 1997) and for conventional breakwaters provided with concrete armour units (Schepers, 1998). In all cases similar site conditions are applied. Costs taken into account are construction costs and capitalized maintenance during the lifetime of the construction.

Layout and site conditions

There are two breakwaters in the adopted layout: a northern mole of 1 km and a southern of 3 km length. Both are subdivided in sections with a specific average depth (figure 1). In all solutions the shallow sections in the breaker zone (approx. -6.0 m) are designed as conventional breakwaters. Therefore cost comparisons are due to differences in the deeper sections. The depth at the northern head is at -10 m, at the southern head at -15 m. Depth contours of the sandy seabed, D_{50} = 200 micron, are parallel to the coast, sloping 1:100 down to deep water.



SECTION	length	av. depth
I	-	15.0
II	1500	12.9
III	950	8.95
IV	550	2.95
V	-	10.0
VI	500	8.10
VII	500	2.80

Figure 1 Layout of breakwaters and water depth

Wave climate at deepwater (H_{so}) and setup of waterlevels (h) above MSL are expressed in terms of probability of exceedance of these values (figure 2). The storm duration is set at 6 hours. For the deep water conditions the mean wave period T_z is related to H_{so} according (Allersma and Massie, 1973):

$$T_z = 3.94 * H_{so}^{0.376} \quad (1)$$

The wave direction is pependicular to the coastline. In view of the shallow area near the coastline the maximum significant waveheight H_s is limited by the local waterdepth h according to H_s/h = 0.55. The tidal amplitude is 1.0 m.

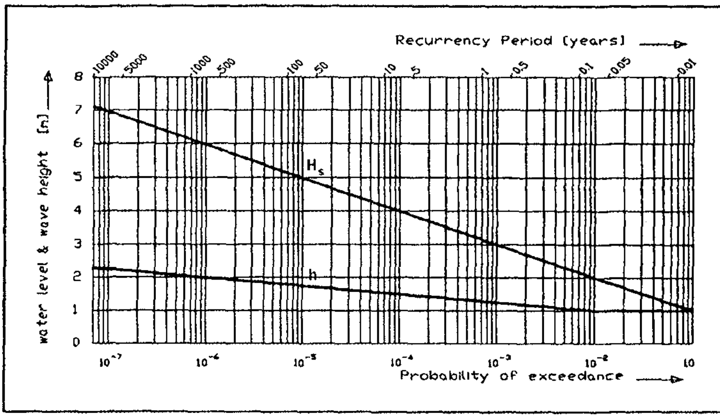


Figure 2 Wave and sea climate

Two yield curves A and B of the quarry are adopted (figure 3), which are similar to ordinary existing curves. The more gentle curve A is applied to the conventional breakwater (without concrete armour units) and to the caisson types, curve B to the breakwater types with concrete armour toplayers.

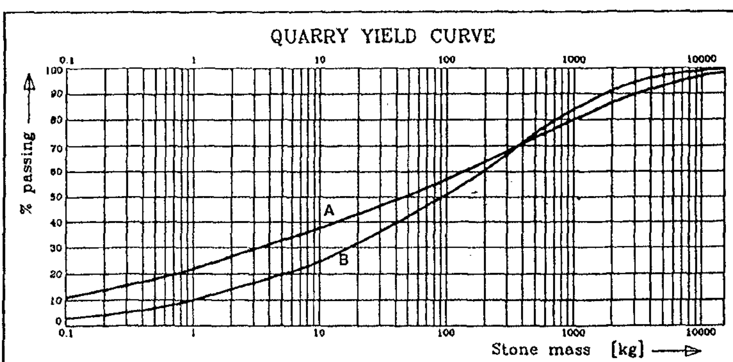


Figure 3 Quarry yield curves

Stability requirements

Design calculations for the rubble mound breakwater, including the stability of the concrete armour units, are based on methods developed by Van der Meer (1993), applied in the program Breakwat. The design of the caisson type of breakwaters are mainly based on the method of Goda (1985). In all cases crest levels are high enough so as to achieve a transmitted wave height $H_T < 0.5$ m. Cross sections of the various types of breakwaters from the main layout section (II) are given in figure 4 to figure 8.

Rubble mound breakwaters

*** Conventional breakwater**

For each section of the layout the slopes have been varied so as to achieve optimum total costs given the quarry yield curve A as boundary condition. With the resulting slope 1:3.5 costs of repair are rather low but construction costs increased. The cross section is drawn in figure 4. Heaviest armour gradation is 8-15 tons. The efficiency of quarry production was just 24% .

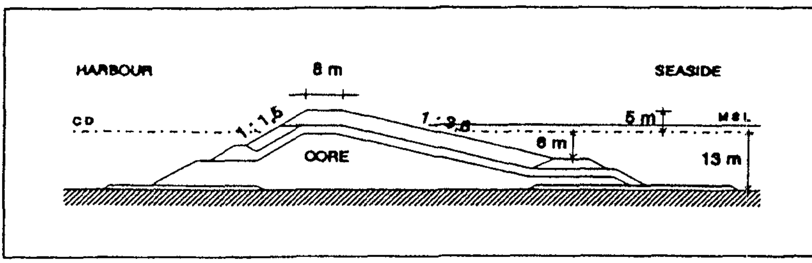


Figure 4 Conventional breakwater, optimal design section II

*** Bermbreakwater**

In order to limit costs of damage the bermbreakwater had been designed rather conservatively using the arbitrary selected return period of 500 years for H_s to size the rocks of the berm. Optimization of the waveheight in view of berm stability and the littoral transport by wave action had not been carried out. The cross section is drawn on figure 5. The quarry efficiency was 78% .

*** Conventional breakwater + armour units: Cubes, Tetrapods or Accropods**

The design method is similar to the conventional type. By replacing the heavy armour rock (8-15 tons, curve B) by concrete armour units (tetrapods, cubes and accropods), the design may be subject to potential cost savings in rock supply. Moreover much effort was given to optimize the quarry efficiency (64%); part of the heavy rocks were crushed for use as armour in the shallow parts of the layout (bedlevels -3 to -6 m). Though the applied steeper slopes (1:1.5 to 1: 4/3) will require higher crest levels to match tranquility in the port basin, rock volumes needed were remarkebly reduced. On figure 6 the cross section of the accropod is drawn.

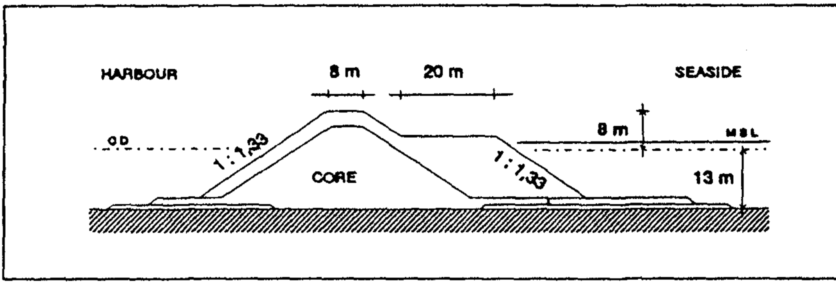


Figure 5 Bermbreakwater, section II

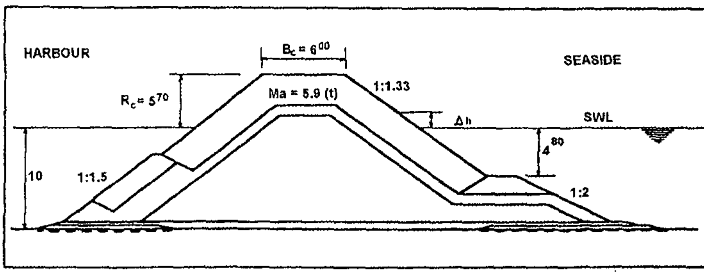


Figure 6 Section II rubble mound + Accropod armour units

Caisson breakwater and composite breakwater

Again the same layout and climate conditions (figure 2) have been used to analyse an optimum design of the two types vertical breakwaters. In both cases the concrete caissons are filled with sand. Minimum freeboard is set by transmission requirements. By variation of the freeboard total costs of material could be reduced. Increasing freeboards may contribute to more structural mass from the caisson above design waterlevel and increase stability to sliding and reduction of the required width of the structure so as to avoid risks of tilting. By limited sailing distances for rock transport (< 150 km) construction costs of the composite breakwater appear lower than of the caisson type. Fragmentation curve A is used for the substructures of rubble mound. Overproduction could be reduced by crushing production of too large rocks down to required grades. Rock production efficiencies are for caisson type 64% and composite type 68% .

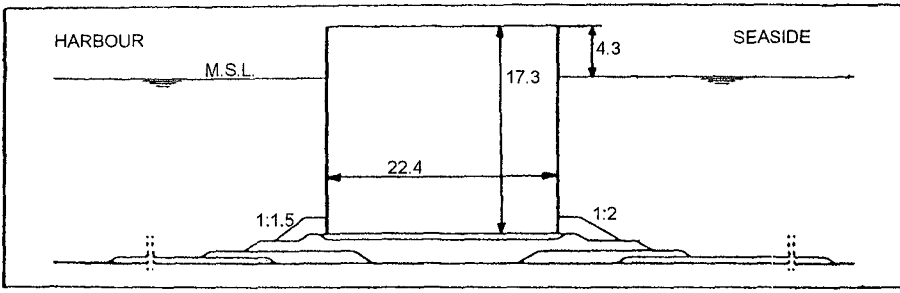


Figure 7 Cross section caisson breakwater

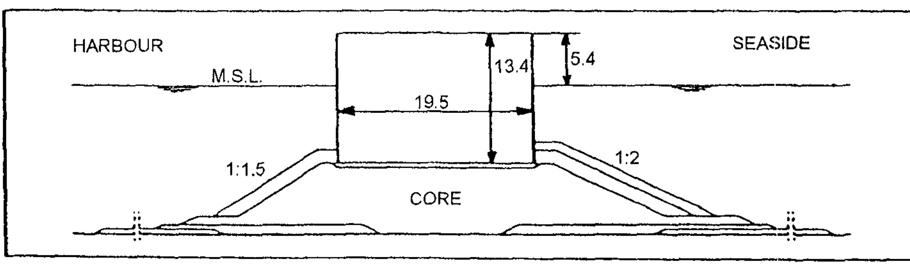


Figure 8 Cross section composite breakwater

Cost calculations

The aim is to base the design on minimum total project costs (TPC), i.e. the lowest sum of costs of construction (CC) and the capitalized costs of damage during the lifetime of the construction. The costs of damage are the sum of direct costs of repair (DC) and indirect costs (IC) due to consequential losses (e.g loss of port operations) multiplied with the present worth factor (pwf). The consequential losses are related to the amount of industrial investments in the port area. The rate of interest used is 4% and the currency is the Dutch guilder (DGL, 1997).

$$TPC = CC + pwf*(DC+IC) \tag{2}$$

Direct costs of repair are related to the yearly risk of damage due to wave action. The method is illustrated for concrete armour units in figure 9 following the damage ranges of N_{od} of Van der Meer (1993).

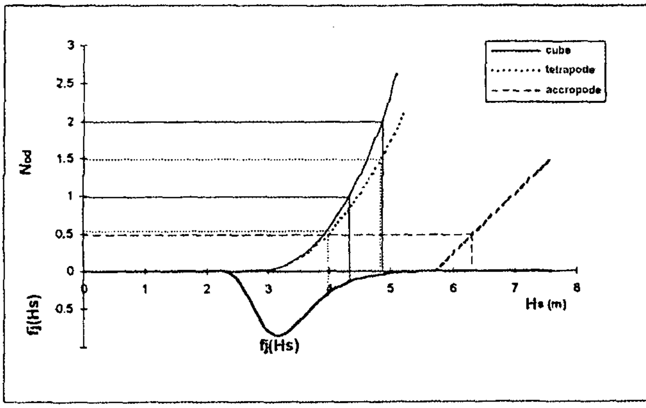


Figure 9 Risk of damage to concrete armour units

Characteristic volumes per m' breakwater

To illustrate characteristics differences in volumes of material applied in the various types, typical volumes for the main layout section II are mentioned in table 1. The remarkable low volume of concrete in the Accropod solution is due to the applied single unit armour layer, as recommended by the supplier. Also clear are the relative high amount of rock of the bermbreakwater and the reduced rock volumes in the types with concrete armour units, when compared with the conventional breakwater, as the result of steeper slopes applied.

TYPE OF BREAKWATER	ROCK (t)	CONCRETE (m^3)	SAND (m^3)	GEOTXT (m^2)
Conventional rubble mound	1700	-	-	35
Bermbreakwater	2200	-	-	-
Rubble mound + cubes	1400	80	-	35
Rubble mound + tetrapods	1400	65	-	35
Rubble mound + accropods	1400	30	-	35
Caisson breakwater	500	90	305	130
Composite breakwater	700	70	245	120

Table 1 Typical volumes/m' section II layout

Conditions for comparison

The applied optimum conditions as bases for the cost comparison between the various types of breakwater are summarized below:

* Varying frequency of hydraulic loads (25 to > 100 years return period)
* Transmitted wave height inside port basin: $H_T > 0.5$ m less than 10 times per year
* Distance of quarry 75 km

* Basic costs according Table 2
* Repair costs acceptable damage = 1.5 * basic costs
* Repair costs of failure = 2 * basic costs

Basic costs of production, transport and construction

Activities	Unit costs
Quarry production (all gradings)	DGL 15.00 / ton
Transport of rocks (over land)	
< 300 kg	DGL 0.25 / tonkm
> 300 kg	DGL 0.40 / tonkm
Construction costs	
Bedprotection	DGL 15.00 / ton
Core	DGL 10.00 / ton
Rock armour and filter layers	DGL 15.00 / ton
Concrete armour units (all in)	
- cubes	DGL 300.00 / m^3
- tetrapods	DGL 325.00 / m^3
- accropods	DGL 400.00 / m^3
Concrete caissons (all in)	DGL 500.00 / m^3
Mobilization & demobilization	DGL 2.0 million

Table 2 List of basic costs

Results

Total project costs have been calculated with increasing return periods of design wave heights (25, 50, etc. years). Clearly these costs decrease with increasing level of design wave heights, due to the rapidly decreasing sum of capitalized direct and indirect damage costs and the relatively small increase of construction costs. Table 3 illustrates the results for the concrete cubes solution. In this case the optimum is reached at a design H_s = 5.4 m, return period 500 years.

ITEM	Costs			
Return period	25 years	50 years	100 years	500 years
Design H_s	4.2 m	4.6 m	4.8 m	5.4
Construction costs Capit.dam. costs	225 230	230 105	237 48	250 10
Total project costs	455	335	285	260

Table 3 Costs for breakwater with cube toplayer (mill.DGL)

The indirect damage costs resulting from the level of investments and the economical value of harbour operations appear to have a great influence on the optimum return period of H_s. Reduction thereof will allow for smaller design values. Similar results have been attained with other designs.

Moreover, in the points of optimum return periods for lowest total project costs, the capitalized damage costs are minimal and a clear distinction can be made between the various designs on the bases of the construction costs in those points. The results thereof are summarized in table 4.

TYPE OF BREAKWATER	CONSTRUCTION COSTS (million DGL)
Conventional rubble mound Bermbreakwater	480 270
Rubble mound + Cube units Rubble mound + Tetrapods Rubble mound + Accropods	250 245 195
Caisson breakwater Composite breakwater	205 215

Table 4 Construction costs of breakwater types

The high costlevel of the conventional type is mainly due to inevitable overproduction of lighter quarry material (quarry efficiency 24%). Costs can be reduced drastically if these volumes can be utilized elsewhere. Although higher volumes are involved, the bermbreakwater yields lower costlevels, mainly due to a higher efficiency (78%). However with increasing transport distances, the savings due to the bermbreakwater decrease (Hauer et al.1995).

The use of concrete units so as to replace heavy armour rocks over e.g. 10 tons have limited advantages to the costs when unit weights and volumes involved are close to the yield curve of the quarry. However, remarkable savings are reached with a single unit layer of Accropods; this might be the result of a single heavy rock layer as well, provided properly placed. The influence of transport costs of rock will decrease in that case.

The caisson breakwater appears more favourable than the composite type. For deeper bedlevels (> 15 m) this may be the opposite due to higher rock volumes required for the base structure.

Most favourable solutions for the ficitious layout and conditions are the accropode and the caisson breakwater. However transport costs and unit concrete costs may change the results. Larger transport distances may reduce the difference and may even result in an advantage for the caisson solution.

Increased waterdepths

For average coastal conditions some interesting results are found with varying water depths. Though expected, it is remarkable that as from water depths larger than 8 to 10 m the caisson types of breakwaters already seem to have cost savings compared to the rubble mound breakwater types, obviously due to increasing volumes of rocks at the base. However, with depths increasing to more than 20 m the composite breakwater appears to be a more cost saving solution, as the increasing heights of the caissons alone require additional widths. The study has been extended to foundation depths of 30 m (Schols, 1997), indicative values are mentioned in table 5.

Conventional type	Caisson type	Composite type
0 - 8 m	8 - 20 m	20 - 30 m
Waterdepth		

Table 5 Indicative range of favourable breakwater type

However, for real situations specific site conditions may alter the results drastically. Construction costs depend strongly on the rate of down time due to wave climate and tidal height conditions, if offshore located breakwaters are considered. Production of accurate placing of heavy concrete armour units from floating crane barges, may vary quite differently from the speed of placing caissons in full height or as composite type, in the presence of ocean swell and local wind waves. Moreover, the feasibility of a caisson solution depends largely on the stability of the foundation, and in particular the sensivity of the subsoil to liquefaction

Conclusions

For conditions similar to the case following can be concluded:
- The rubblemound breakwater + Accropod toplayer is most attractive, due to less rock volumes (steeper slopes) and the application of a single unit toplayer.
- The caisson breakwater appears to be a good alternative and may even become favourable if costs of required rock transport for a conventional type are increased.
- The high costlevel of the conventional type is mainly due to inevitable overproduction of lighter quarry material. In cases lower design wave heights are applicable, lighter quarry material and higher level of quarry efficiency will reduce related construction costs.
- Although large volumes of rocks are required, the bermbreakwater yields lower costlevels, mainly due to higher efficiency of quarry production.
- In case high wave heights require heavy armour units, a conventional rubble mound breakwater provided with a toplayer of concrete elements will soon become favourable. A stable single unit toplayer (armour) may increase this advantage.
- Caisson types of breakwater seem to become advantageous with waterdepths exceeding approx. 10 m. For composite breakwaters the advantage may start at waterdepths exceeding 20 m.

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