Comparison construction costs conventiona rubblemound breakwaters/bermbreakwater

by

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Abstract

Given a harbour location and a wave-climate, two different types of rubble-mound breakwaters were designed. Using realistic quarry yield curves and prices for production, transport and construction, the total costs of the conventional statically stable and dynamically stable bermbreakwater designs were compared to each other. This comparison led to the conclusion that differences in building costs strongly depend on the way the quarry yield is subdivided into different stone classes for both types of breakwaters. In order to satisfy the demand of the heaviest armour stone classes, overproduction for the lighter stone classes is inevitable. The extent of this overproduction had decisive influences in the comparison of total costs. Calculations for different transport distances between quarry and construction site showed only minor differences in the comparison of total costs of both types. The concept of dynamically stable bermbreakwaters appeared to provide great possibilities for considerable reductions of the overproduction in the quarry and the total costs. Realisation of maximum reductions yet requires advanced models for the treatment of the problem of longshore erosion with bermbreakwaters. More extensive research on this topic is recommended.

1.0 Site conditions

A fictitious harbour location was adopted (figure 1):



Figure 1: Harbour location

The breakwaters for protection of the harbour were divided in seven sections. Designs for all sections were based on averaged local depth and averaged local wave conditions.

1.1 Wave climate

For the wave climate at deep water a North Sea wave climate was adopted (fig 2).



Figure 2: Wave climate

These wave and waterlevel statistics were modelled by two exponential distributions:

$$F(H_s) = 1 - e^{-\frac{H_s}{0.434}}$$
 \wedge $F(h) = 1 - e^{-\frac{(h-0.5)}{0.109}}$

The deep water wave statistics were transformed using shoaling, refraction and breaking laws. The resulting local wave statistics for each section were used in all design calculations. For the wave steepness a deterministic value of 0.02 was adopted.

1.2 Quarry yield curves

Two quarry yield curves were adopted: a wide curve A and a steep curve B:



Figure 3: Quarry yield curves

At first, all designs were based on a quarry production described by the wide quarry yield curve A. Later on, the influence of a different quarry yield was investigated by adopting the steep quarry yield curve B.

1.3 Production, transport and construction prices

Activities	Unit Cost	
dredging	US \$ 5 / m ³	
purchase and placing of geotextiles	US \$ 10 / m ²	
placing of gravel filters	US \$ 7,5 / m ³	
placing of core material	US \$ 3 / ton	
placing secondary armour / rolling equipment placing secondary armour / floating equipment	US \$ 5 / ton US \$ 7,5 / ton	
<pre>placing primary, lee & toe armour: stones < 5 tons / rolling equipment stones < 5 tons / floating equipment stones > 5 tons / rolling equipment stones > 5 tons / floating equipment</pre>	US \$ 5 / ton US \$ 7,5 / ton US \$ 7,5 / ton US \$ 10 / ton	

Table 1: Average costs construction activities

Based on an analysis of costs for the opening of a quarry (ref (1)), equipment, blasting, sorting, loading and overhead costs, the total average production costs were approximated at US \$ 7,5 per ton irrespective of the number and size of the different stone classes.

The total transport costs (hire of equipment, fuel, unloading at the project site) were approximated at 12,5 dollarcents per ton per kilometer. For stone sizes heavier then 700 kg. the transport costs were estimated at 20 dollarcents per ton per kilometer.

Average costs for different construction activities are given in table 1.

2.0 Conventional rubblemound breakwater design

To reduce the number of alternatives only non overtopped breakwaters were considered. A typical design for the cross sections of the non-overtopped conventional rubble-mound breakwater is shown in figure 4. This cross section concerns the optimum design of section II of the conventional breakwater.



Figure 4: Optimum design section II conventional breakwater

In order to ensure a stable toe in shallow water without the need of a heavier stone class for the toe armour, the toe was designed in an excavated trench for the sections IV and VII. Based on a criterium of no serious overtopping for the 0.10 year storm the crest level for the conventional breakwater was set at 5.00 m. CD. For accessibility reasons the crest width was set at 8 m.

2.1 Quarry divisions for the conventional breakwater

In order to find the optimum design with maximum quarry use and minimum costs, two subdivisions of the quarry yield A in stone classes have been evaluated. The chosen subdivision in six stone classes is shown in figure 5.

The total quarry production needed for construction of the conventional breakwater was determined by the amount of stones needed to satisfy the demand of the

heaviest armour stone class (8-15 ton). According to the subdivision of the wide quarry yield in figure 6 only 3.8 per cent of the total quarry yield was suited to serve as armour stones. For the final conventional breakwater design 16 per cent of the total quarry demand for all stone classes concerned the heaviest armour stones. So, for the favourable quarry yield described by the wide quarry yield curve A, only 100*(3.8/16) = 24% of the total production could be used to build the breakwater. For the steep quarry yield curve B this percentage was even less.



Figure 5: Quarry yield subdivision conventional breakwater

2.2 Optimum design criteria for the conventional breakwater

After choosing the nominal diameter D_{n50} of the armour stone class for both quarry yield subdivisions, the only remaining design parameter for the armour layer is the seaside front slope. Employing a conservative procedure, the required slope of the armour layer can be set by a criterium 'only minor damage' after an extreme storm with a quite long, arbitrary chosen recurrence period (in most cases 50 to 100 years). On the one hand this conservative design for the breakwater will be a safe design, on the other hand it will also be relatively expensive. A more advanced procedure has been applied in this study.

The required front slope has been related to a deep water design wave height according to a criterium 'only minor damage' to the armour layer after an storm with this wave height. Evaluation of this criterium was done by calculation of the required combination of front slope and design wave height to ensure a maximum damage factor S = 2 (start of damage) in the Van der Meer equations (ref (2)). In conformity with the relation between the design wave height and the recurrence period of this wave height, the required front slope could also be expressed as a function of the recurrence period.

After this, for all sections of the breakwater the total construction costs (production in the quarry, transport, placement) have been formulated as a function of the front

slope and the transport distance between quarry and construction site. For a wide range of front slopes the sum of these total construction costs, the capitalized expectation of the maintenance costs and the capitalized expectation of the economic consequences of failure was calculated and optimized by realistic presumptions. In order to evaluate the economical consequences of failure in addition to the repair costs of the failed breakwater, the total loss of harbour facilities and loss of harbour income were estimated by means of the next model (TC = total construction costs):

Total investment for the breakwater:	2
Total investment facilities:	С
Yearly income from harbour activities: 0.1 * 25 * Te	С
In case of failure of the breakwater:	
Repair costs failed breakwater 1.25 * To	С
Loss of assets facilities $\dots \dots \dots$	С
Total loss of income (3.5 years)	C
>> Total damage = 20 * Total damage =	С

For each section of the breakwater optimum front slopes with minimum total costs were chosen. By way of the relation between front slope and design wave height, these optimum front slopes could be translated in optimum design wave heights. Figure 6 shows the results of calculations for section III in case of a transport distance of 75 km between quarry and construction site.

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Figure 6: Relation design wave height and costs section III

In this model, the high economical losses in case of failure had a major influence in the optimization. The optimum designs for most sections of the breakwater were found for design conditions corresponding to front slope designs which excluded the possibility of failure. This exclusion of the possibility of failure is due to the limits to the maximum local wave height ($H_{max} \approx 0.55 * h_{loc}$, where h_{loc} corresponds to the sum of local water depth and wind set up during the most extreme storm period). As a result, the optimum design wave heights for most sections mainly depended

on the local water depth. Optimum designs for the sections in deep water demanded the heaviest design conditions. In figure 7 a comparison of total costs between section II (local depth = 12.9 m.) and section III (local depth = 8.95 m.) is shown. The increased maximum local wave height for section II resulted in a considerable shift of the optimum design wave height: $H_{design} = 3.9$ m. for section III and H_{design} = 5.1 m. for section II. All these values refer to a transport distance of 75 km between quarry and construction site.



Figure 7: Comparison total costs section II and III

After calculation of optimum design wave heights for all sections, the total costs of the designs for the two quarry yield subdivisions were compared with each other and the most favourable stone division was chosen. The influence of transport distance between quarry and site was investigated by calculation of total costs for two designs with two different transport distances 25 and 75 km. Since the transport costs are increasing linearly with distance, total construction costs for all possible distances can be calculated from these two designs. Table 2 shows the design variables and total construction costs per section of the design for a transport distance 75 km.

Sections I and V relate to the two roundheads in the breakwater design. The construction of these sections required comparatively larger percentages armour stones than the construction of the cross sections. This resulted in comparatively less heavy design conditions for both roundheads. Sections IV and VII relate to the sections in shallow water, which connect the breakwater to the shore. These sections had optimum recurrence periods less than 1 year.

Section no.	Section Length [m]	Local Depth [m]	Front Slope	H _{optimum} [m]	T _{return} optimum [Yrs]	Total Costs [10 ⁶ \$]
I	-	15	1:3.2	4.9	55	5.2
II	1500	13	1:3.8	5.1	90	145.0
III	950	9	1:2.7	3.9	≥ 6	75.7
IV	550	3	1:1.5	2.6	≤ 1	4.5
V	-	10	1:2.8	4.2	≥ 12	4.0
VI	500	8	1:2.7	3.8	≥ 5	40.5
VII	500	3	1:1.5	2.6	≤ 1	4.1
total construction costs				279.0		

Table 2: Design variables and total costs per section

3.0 Berm breakwater design

Although much research on behaviour of bermbreakwaters has been performed in the past decade, designing bermbreakwaters still has to be done on the basis of experimental results with a wide range of scatter. Most knowledge concerns the dynamical behaviour of cross sections under design conditions. However, very little is known about the dynamical behaviour of cross sections under failure conditions. Also the knowledge of longshore erosion with bermbreakwaters still contains considerable uncertainties. Hence the designs for the dynamically stable bermbreakwater concept had to be based on conservative assumptions.



Figure 8: Optimum design section II bermbreakwater

A typical cross section of all bermbreakwater designs is shown in figure 8. This cross section concerns the optimum design of section II of the bermbreakwater.

The crest height of the bermbreakwater was set at the high value of 8.00 m. CD, a value chosen to virtually eliminate the possibility of failure due to overtopping of the bermbreakwater. This was judged to be necessary in view of the uncertainties in the knowledge of behaviour of bermbreakwaters under failure conditions. For accessibility reasons the crest width was set at the same 8 m. as for the conventional breakwater.

3.1 Quarry divisions for the bermbreakwater

At first, the wide quarry yield curve A was adopted. Five different subdivisions of this quarry yield with increasing M_{50} of the armour stone class have been evaluated. Later, the influence of a different quarry yield was investigated by repeating the same procedure with the steep quarry yield curve B. Each subdivision consists of three stone classes (figure 9): armour, core and a part of the quarry yield with stone masses below 0.01 ton. This last part is used in construction of the gravel filters between core and geotextiles.



Figure 9: Steep quarry yield division bermbreakwater

In all designs, the demand of gravel for the filter layers of the bermbreakwater was on average only 4 per cent of the total quarry demand. The percentages of gravel in the quarry yield (24.8 per cent for the steep quarry yield B in figure 10) always exceed this demand. For this steep quarry yield, overproduction of about ca. 21 per cent is inevitable. For the wide quarry yield A this inevitable overproduction will increase to ca. 33.5 per cent. With an optimally designed bermbreakwater, overproduction in the quarry can theoretically be limited to only this overproduction of gravel.

3.2 Design cross sections of the bermbreakwater

Calculations on the dynamical behaviour of cross sections have been performed by means of the program BREAKWAT of Delft Hydraulics Netherlands (ref (8)). An example of the output of the BREAKWAT program is shown in figure 10.



Figure 10: Example output Breakwat program

Profile 1 is the constructed initial profile, profile 2 the dynamically stable profile after a storm with a recurrence period equal to the planning period of the breakwater (50 years), profile 3 the profile after an extreme storm with return period of 10 times the planning period. After some introductory calculations it was concluded that minimal use of material could be realized by setting the berm height at 3 m. CD. All slopes were set at 1:4/3. The berm width was set according to a criterium 'no erosion upper seaside slope' for the storm with return period equal to the planning period. The position of the core was fixed by a criterium 'no exposure core' for the extreme storm with return period of 10 times the planning period.

Profile 3 in figure 10 must be interpreted as a graph of the *possible* behaviour of the bermbreakwater in failure conditions, as these conditions exceed the range of model tests conditions which underlie the BREAKWAT program.

3.3 Longshore transport at the bermbreakwater

In order to evaluate the problem of longshore transport a probabilistic approach according to Vrijling et al. (ref (7)) was used. Estimation of the required additional armour was based on the calculation of the total number of stones which can be expected to move during the planning period of the breakwater. Twice this amount of stones (for both roundheads, sections I and V) has been added to the total need of armour stones. The amount of this additional nourishment to cope with longshore transport was very small (maximum 3 per cent of the total need of armour stones) and had minor influence on the total construction costs. Total production, transport and placement costs for armour, core, gravel, textiles and additional nourishment are shown in table 3, which concerns the optimum bermbreakwater design for the flat quarry yield curve A and a transport distance 75 km between quarry and construction site.

Component	Production costs [10 ⁶ US \$]	Transport costs [10 ⁶ US \$]	Costs placement [10 ⁶ US \$]	Total costs [10 ⁶ US \$]
armour	15.3	30.7	15.3	61.3
core	18.5	23.0	5.6	47.1
gravel	20.4	2.8	2.2	25.4
textiles	2.1	-	2.1	4.2
nourishment	0.3	0.6	0.3	1.2
total construction costs				139.0

Table 3: Costs per component optimum bermbreakwater design

4.0 Comparison

Apart from the provision of nourishment for longshore transport, no realistic estimates could be made for the extent and costs of maintenance at the bermbreakwater design. Due to the lack of knowledge of the behaviour of bermbreakwaters in failure conditions, also no realistic estimates could be made for the probability of failure for this type of breakwater. Therefore the design of the bermbreakwater was based upon the conservative approach that all sections should be able to survive design conditions with a recurrence period of 500 years.

The design of the conventional breakwater was based upon a more advanced method of minimizing the sum of construction costs, maintenance costs and economical consequences of failure. This resulted in design conditions with recurrence periods of maximum 90 years. The optimum designs for most sections are based on relatively modest, depth limited design conditions. Notwithstanding the more refined approach of the conventional design the bermbreakwater concept proved to offer considerable savings in construction costs.

As no maintenance costs and no economic consequences of failure have been calculated for the bermbreakwater, these costs have been left out of the comparison, that comprises the construction costs alone.

4.1 Influence transport distance between quarry and site

Comparison of total construction costs for the optimum designs of both types of breakwaters and the wide quarry yield curve A yielded table 4:

Distance quarry-site [km]	Total costs conventional [10 ⁶ US \$]	Total costs bermbreakwater [10 ⁶ US \$]	Savings bermbreakwater [10 ⁶ US \$]
0 ·	231	82	149
25	247	101	146
50	263	120	143
75	279	139	140
100	295	158	137
250	396	273	123

Table 4: Comparison costs optimum designs for various distances (quarry yield A)

Building a bermbreakwater instead of a conventional rubble-mound breakwater for this harbour project can result in savings of about 150 million US \$. This is due to the better adaptation of the demand for the various stone classes of the bermbreakwater to the supply of the quarry. This advantage is slightly diminished because the bermbreakwater design requires more material then the conventional breakwater design, and consequently the total transport costs will be slightly higher. The extent of this saving is linearly related to the transport distance between quarry and site. In percentages of the conventional rubble-mound total construction costs the bermbreakwater shows savings of 64 per cent for very short transport distances to 30 per cent for long transport distances between quarry and site.

4.2 Influence quarry yield curve

Due to the deterioration of the adaptation of the quarry supply to the demand of the primary armour stone class, a steep quarry yield curve B will have enormous consequences for the total construction costs of the conventional breakwater. For the wide curve A ca. 24 per cent of the total quarry production can be used to build the breakwater. For the steep curve B this percentage decreases to ca. 9 per cent. As a result the necessary total quarry production increases by 100*(24/9) = 266 per cent. This enormous increase of necessary production causes on an average a 115 per cent growth of total construction costs. On the contrary, the total construction costs of the bermbreakwater are only slightly effected by the more favourable gravel supply for quarry yield curve B. This more favourable gravel supply resulted in on an average an 4 per cent decrease of total construction costs. A comparison of total construction costs for the optimum designs of both types of breakwaters and the steep quarry yield curve B is listed in table 5.

Distance quarry-site [km]	Total costs conventional [10 ⁶ US \$]	Total costs bermbreakwater [10 ⁶ US \$]	Savings bermbreakwater [10 ⁶ US \$]
0	570	76	494
25	586	96	490
50	602	116	486
75	618	136	482
100	634	156	478
250	731	276	455

Table 5: Comparison costs optimum designs for various distances (quarry yield B)

Now savings of 68 to 87 % can be made, when the bermbreakwater design is preferred to the conventional design.

5.0 Remarks

Some subjects neglected in the comparison will be discussed in this chapter.

The calculation of total costs for the conventional breakwater and the bermbreakwater was based upon the same prices for production, transport and construction. In reality, the construction of a conventional design will require more accurate and complicated methods for the placing of the various filter- and armourlayers than the construction of a bermbreakwater design. Therefore the construction costs of a bermbreakwater can be expected to be more favourable than suggested.

The bermbreakwater was designed without any restriction to the expected rates of longshore transport. For the nourishment of these rates of longshore transport was provided by the placing of additional amounts of armour stones. As the knowledge about the phenomenon of longshore erosion with bermbreakwaters still is imbued with uncertainties, one could doubt the soundness of this design and demand a conservative restriction to the minimum weight of the armour stones to eliminate the risks involved with longshore erosion. However, the ratio $H_{sd}/\Delta.D_{n50} = 4.4$ of the optimum bermbreakwater design is amply within the range $3 \le H_{sd}/\Delta.D_{n50} \le 6$ suggested by Van der Meer and Vrijling (ref (2) & (7)).

In this study the overproduced amounts of stones have been treated as losses. In a practical case one could however be able to sell or use at least some parts of these amounts. In that case especially the conventional design can be expected to be more favourable than reported in this study.

Due to the specific harbour lay-out with the harbour facilities directly behind the breakwater, in this study only non-overtopped breakwaters were considered. In some practical cases one can also decide to change the harbour lay out and construct an overtopped conventional breakwater. Building an overtopped conventional breakwater will result in a considerable decrease of the required amount of armour stones and, in consequence of this decrease, a considerable decrease of the economical loss due to overproduction. Therefore building an overtopped conventional breakwater can result in considerable savings. Building an overtopped bermbreakwater on the contrary is not considered justified engineering practice at present.

In view of the high expenses for the production of the required rubble armour stone for the conventional breakwater, another way to lower the construction costs for the conventional alternative may be found by application of some other type of armour, like concrete cubes or tetrapods.

Finally, the severity of the local wave conditions will also effect the results of the comparison of costs. Quite severe wave conditions have been adopted for this study. Less severe wave conditions will require lower armour weights. As a result, an increased part of the total quarry yield can be used for the armour stone classes. This enlarged supply of armour stones will change the mutual differences in production costs for the conventional and the bermbreakwater design. The high production costs for the bermbreakwater. Therefore, in case of less severe wave conditions, the conventional design will be more favourable in proportion to the results of the comparison in this study.

6.0 Conclusions

With regard to the specific harbour lay out and the specific wave conditions in this study, the construction of a bermbreakwater instead of a conventional rubble-mound breakwater resulted in considerable savings. For small transport distances between quarry and construction site (0 to 25 km) up to 64 per cent of the total costs for the conventional breakwater could be saved. For long transport distances (250 km) still 30 per cent of the total costs could be saved. In all considered circumstances the total costs for a conventional design exceeded those for a bermbreakwater design.

Moreover the costs for a conventional design appeared to be far more sensitive to a disappointing quarry yield during the actual production in the quarry. Misfortune during the production can result in a more then 100 per cent growth in total costs for a conventional design. The total costs for the bermbreakwater were only slighly effected by a deviation of the quarry yield.

7.0 References

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