Criteria for the selection breakwater types

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Introduction

In history PIANC has paid quite some attentions to breakwaters, in ancient years results of practical experience have been published in the country presentations at the Conferences, but later some of these papers started to have a more scientific basis. One of the first papers giving a good background on the processes of stability of breakwaters was published on the conference in Rome by IRIBARREN CAVANILLES AND NOGALES [1953]. As usual in those days, the paper does not have a title, but in fact it deals with the stability calculation of armour units.
Later more detailed work was published in separate reports, like the "Final report of the International Commission on the study of Waves" in 1976.

The state of the art

Some years ago PIANC has issued a report on the design of rubble mound breakwaters [PIANC, 1992]. and this year the reports on vertical wall breakwaters and on berm breakwaters are published. For convenience reference to these reports will be the RM-report, the VW-report and the BB-report.
The core of the RM-report, as well as the VW-report is a probabilistic approach. The BB-report does not really present a design guideline; for these breakwaters simply too much is unknown at this moment in order to publish such a guideline. Because of that the BB-report focuses mainly on the experiences with various berm breakwaters.

The RM-report focuses on the main armour of the breakwater. For the main armour a probabilistic approach is followed. For the other design aspects a deterministic calculation is suggested. In the VW-report the overall stability of the breakwater (i.e. the caisson) is considered in a probabilistic way, while the other aspects (e.g. the structural strength of the caisson) are determined in a probabilistic way.

In both reports in fact not a full probabilistic calculation is suggested, but the method of Partial Safety Coefficients is followed. The choice was made for this method instead of a full probabilistic computation, because at that moment a full probabilistic computation was difficult to do on a simple computer. But even now, which the increase power of computers it is still useful to use this approach because computation is more transparent.

In fact two partial coefficients are relevant; one expresses the uncertainty in the boundary conditions (mainly the wave height) and the other one the uncertainty in the strength of the structure. Because the uncertainty in the strength of the materials is very small, this second uncertainty mainly depends on the uncertainty of the used computational formulae.

A number of examples worked out in both reports shows that the most important coefficient is the partial coefficient for the boundary condition.

Damage or failure

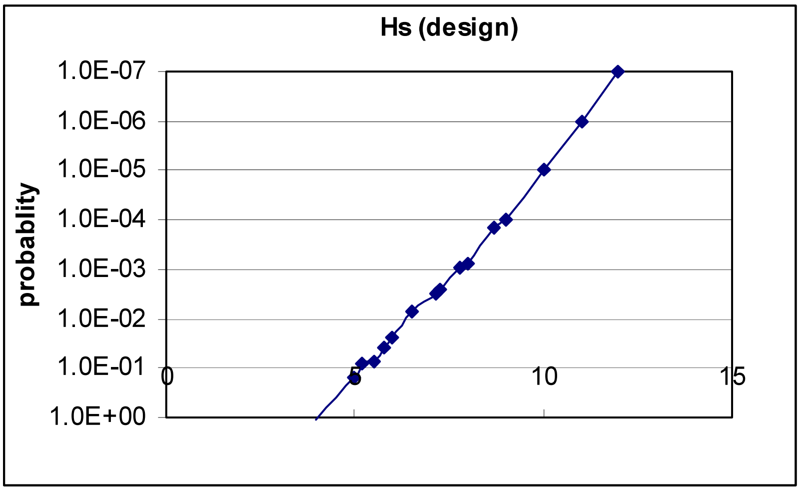
In order to make a good choice between the various types of breakwaters a good and clear understanding of the difference between damage and failure is absolutely necessary. In case of damage the breakwater can continue to fulfil its tasks without any problems, but without repair of the damage, it will probably not survive a next storm event.
In case of failure the breakwater cannot fulfil its basic tasks any more. From an operational point the allowable failure-probability depend on the loss of income of the port due to the fact that the port cannot be used to its full extend because of absence of normal functioning of the breakwater.

For example, a breakwater fails during a storm event. Until reconstruction is finished the loss of income during the time the port is not operational can be computed. The probability of failure can be computed, and the product of this probability and the loss of income plus the reconstruction costs are the risk. The extra costs to make the initial breakwater safer can be compared with the decrease in risk of failure, and from that analysis follows an optimum failure frequency.

For damage a similar approach can be followed. When a breakwater is damaged during a storm, repair is needed. The probability of occurrence of such damage multiplied with the repair cost is the risk. Also here the extra costs to make the initial breakwater stronger can be compared with the risk, and from that analysis follows an optimum damage frequency.

Example

In order to illustrate a few aspects of optimisation and risk, an example is used. This example is partly borrowed from D'ANGREMOND AND VAN ROODE [2001]. As basis a rather standard wave climate with the following exceedance is used:



Economic optimisation of rubble mound breakwaters

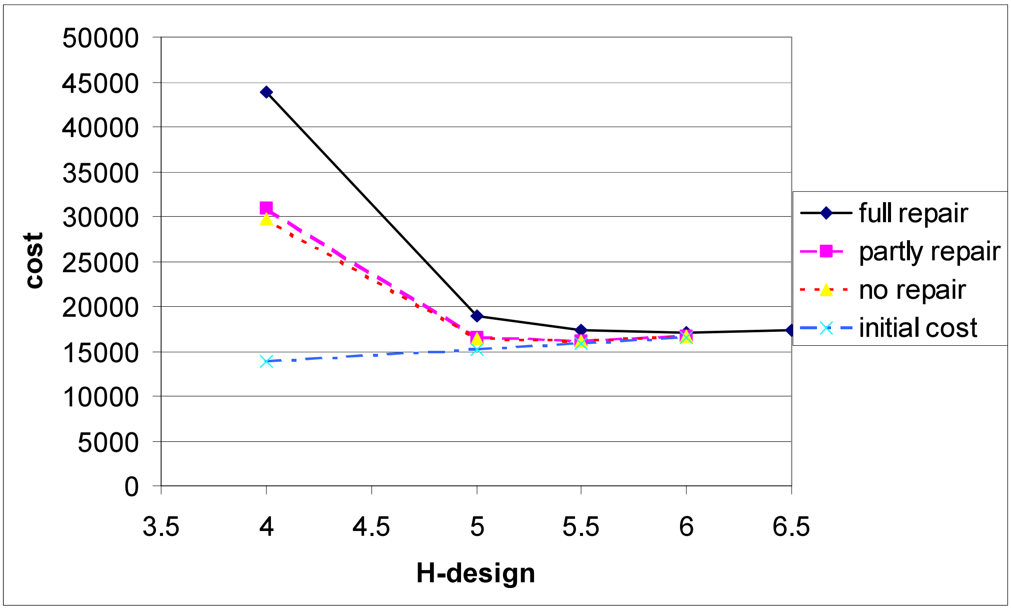
For rubble mound breakwaters failure is usually not considered as a design criterion but only extreme damage. Already in 1994 a method has been developed to calculate the optimum rubble mound breakwater { VAN DER KREEKE AND PAAPE, 1964]. the breakwater is designed for a number of design wave heights, where a higher design wave height requires a heavier and more costly armour layer, whereas the core remains unchanged.

In D'ANGREMOND AND VAN ROODE [2001] this has been worked out in a numerical example. in this example the initial cost of the breakwater is € 8620 for the core and €1320*H_s for the armour units.

H_s	total cost I + S		
	full repair of partial damage	Only repair of serious damage (>8%)	No repair, only reconstruction in case of failure
(m)	(€)	(€)	(€)
4	43900	31000	29800
5	18970	16570	16420
5.5	17400	**16200**	**16200**
6	**17080**	16630	16630
6.5	17300		

Apart from the selected maintenance strategy the optimal design wave height can be determined. In the table these values are printed bold. It means practically that in this example a design wave height of 5.5 m should be used.

Graphically this can be represented as follows:

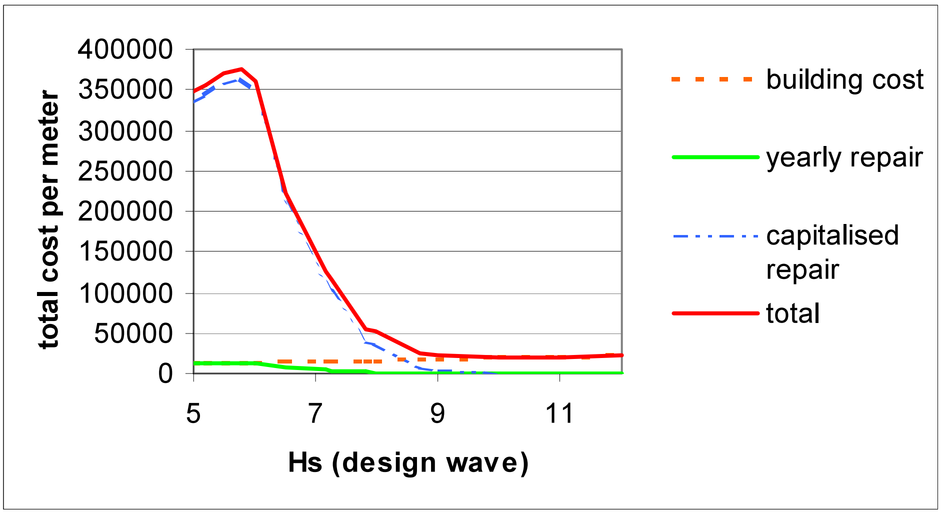


From the data and the graph one can easily find the most optimum design wave height for this breakwater.
However, in practice this method has not very often been applied. In first instance this seems very strange. When there exists a good method to find the optimum design wave height, why not using it ?

The main reason for this is that in the perception of most breakwater owners it is not possible to add up investment costs and maintenance costs. Usually money comes from two different sources, and are often not interchangeable.

The typical vertical wall breakwater

The above mentioned reasoning for optimisation of the design wave height for a rubble mound breakwater is not valid for a typical vertical wall breakwater. In the above it was explicitly assumed that in case of a lower design wave, there will be a higher damage, and consequently higher repair costs. But this was a rather linear relation. In case of a vertical wall breakwater, this relation does not exist. When there is an overload, in general the breakwater will fail completely and will have to be rebuild. This means that repair cost after overloading is usually in the order of the initial costs, sometimes even higher because one has to remove the remains of the failed breakwater.
In such cases, by definition one has to select a relatively high design wave. First, because of



pure economic reasons. When a too low design wave height is adopted, the probability of failure is too high. The line "yearly repair" gives not the real yearly repair costs, but the yearly risk of repairing the breakwater (=in this case rebuild the breakwater).

For the same example as before, now a vertical wall breakwater is used. It is assumed that the cost of the breakwater is € (1500* H_s +4000). per running meter. The lifetime is 100 years and the interest rate is 3.3%.

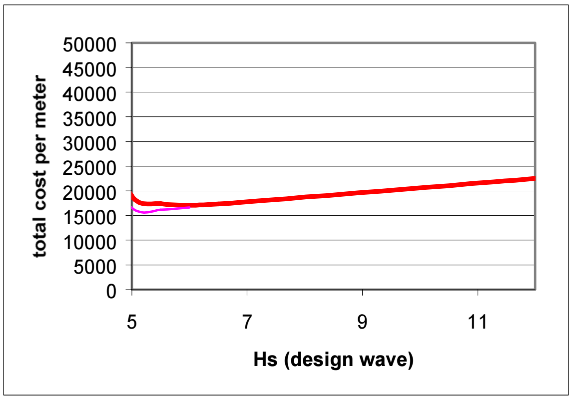
It is obvious that for vertical wall breakwaters a relatively high design wave height has to be selected, otherwise the risk is too high.

Conclusions from the example

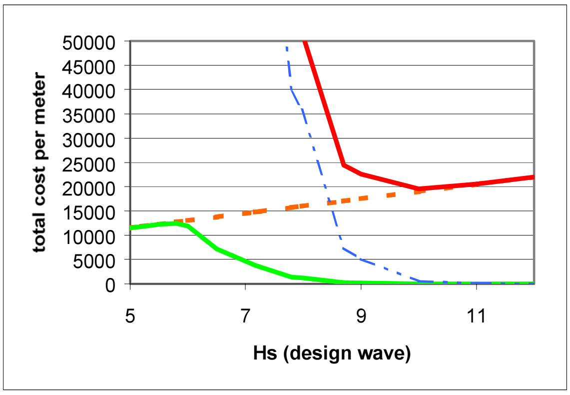
The total cost of a well designed breakwater may not be very much different for both a rubble mound as for a vertical wall breakwater (on purpose in the examples the cost are made more or less identical), but that a vertical wall breakwater requires a much higher design wave, and consequently a much higher initial investment. Repair is not so often needed for a vertical wall breakwater.
To make the comparison easier, the results are plotted in the figures below on the same scale. This example has been selected in such a way that the total cost of the structure are more or less identical.

But because the lines in case of the vertical wall breakwater are much steeper than in case of the rubble mound, its implies that the effect of uncertainties in wave data have a much larger impact in case of a vertical wall breakwater. This means that if the wave data are not very reliable, from a financial point of view the rubble mound breakwater has to be preferred.

Comparable calculations can be made for a dynamic breakwater. In that case one will see that the design is even less depending on the quality of the input data.



Rubble mound breakwater



Vertical wall breakwater

Of course, this example is only valid as the costs of the various types of breakwaters are very much comparable.

Other considerations

There are many other considerations; often they are even more decisive that the financial differences on the long run. In deep water, in case of large tidal differences or in case the entrance of the port is relatively narrow, it is often impossible to construct a rubble mound breakwater in an economic way. On the other hand when no facilities are available to build an launch caissons or there is subsoil with highly variable strength (and consequently highly variable settlement) a caisson structure may be impossible.

Of course, an important point in the decision is the local experience with breakwaters. In case an owner or his consultant has much experience with caisson structures, they will certainly prefer them. The same is valid for the rubble mound. Building time at sea is also a consideration.

References

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IRIBARREN CAVANILLES, R. AND NOGALES Y OLANO, C. [1953] Rapport Espagnol, Proceedings XVIIIe Congrès International de Navigation, section II, Q1, pp45-66

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