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CONSTRUCTION, MAINTENANCE AND REPAIR AS ELEMENTS IN RUBBLE MOUND BREAKWATER DESIGN

by

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1. INTRODUCTION

Very often rubble mound breakwater designs seem to be a result only of stability considerations corresponding to design wave conditions. Designers tend to put too little emphasis on practical problems related to construction, maintenance and repair.

As is discussed in the paper due consideration of these problems leeds to a more economical design in terms of lower total costs during the structural lifetime.

2. CONSTRUCTION

In the design much emphasis is laid on the stability of the armour layer of the breakwater.
However, regarding the construction and the total costs the armour layer is often not so important. The core, toe protections and secondary armour layers of quarry rock are at least as important. This can be demonstrated with an example of the relative costs of the construction materials used in Zeebrugge, Belgium, see Table 1. Fig. 1 shows the typical cross section of the breakwater.

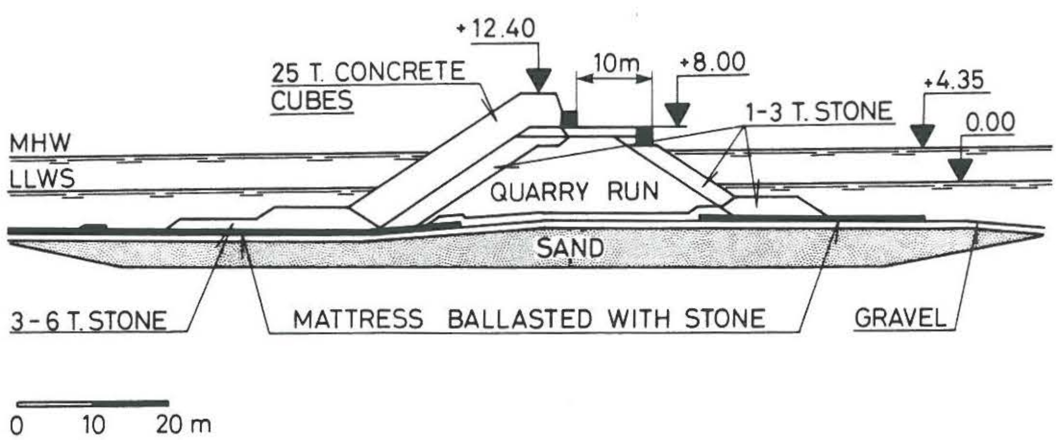

Fig. 1. Typical cross section of outer breakwaters Zeebrugge, Belgium.

Table 1. Relative construction costs, breakwaters Zeebrugge.

Costs of materials per m³ breakwater in % of core material	%
Material	%
Sand	8
Gravel	67
Core 2/300 kg + 1/3 t	100
Secondary layer 1/3 t	95
Armour layer 25 t cubes	72
Berms 1/3 t and 3/6 t	120

	Costs in 0/00 of total/m			Superstructure
	Materials	Execution	Total	Alone in % per m
Soil replacement + compaction	0	92	92	
Bottom protection	139	201	340	
Toe protection (berms)	58	42	100	
Core	204	63	267	57
Secondary layers	65	26	91	20
Armour layer	41	45	86	18
Cap construction	12	12	24	5
	519	481	1000	100

As can be noted the core takes almost 60% of the costs for the superstructure and the armour layer only 18%. (Note that in this case the word superstructure means the total structure except sea bed preparation, bottom and toe protection.)

In Zeebrugge the core material is the cheapest grade of stone, being quarry run of 2-300 kg. This grade of stone, however, limits the workability to a significant wave height of 1.2 m, which value is exceeded during approximately 20% of the time. Therefore, it was worth trying to reduce the down-time using heavier but more costly grades of stone when the waves exceed the limit for quarry run. In this way important savings could be made as the following example may show.

The total costs for the construction of the core can be approximated by

$$C = Q \cdot u + n \cdot C_F + (n - x) \cdot C_o$$

in which

Q = total quantity of stone in t

u = unit rate in BEC/t

n = total number of available working days

C_F = fixed costs of operations in BEC/day

$(n - x)$ = total number of workable days, x = days of delay due to unfavourable weather; $(n - x) = \frac{Q}{p}$, p being the daily production in t

C_o = operational costs in fr/day

BEC = Belgian fr. corresponding to app. 0.026 US $

With the following realistic figures:

C_F = 7 mill. BEC/day
C_o = 3 mill. BEC/day
$Q = 6 \cdot 10^6$ t
p = 6000 t/day

the total construction costs can be calculated. This is done for two types of stone, 2/300 kg and 2/300 kg plus 1000/3000 kg for bad weather conditions. The grade 1000/3000 kg is stable up to wave heights of about 2.5 to 3.0 m. Table 2.

Table 2. Example of influence of grading of core material on costs.

Quarry stone grade	Limiting wave height	% of exceed-ence	$n - x = \frac{Q}{p}$	n	x	u	$\bar{u}$ (mean)	C	C
	m		days	days	days	BEC/t	BEC/t	mill. BEC	%
2/300 kg	1.2	20	1000	1250	250	600	600	15,350	100
2/300 kg	1.2	20				600			
+			1000	1053	53		632	14,163	92
1000/3000 kg	2.5	5				800			
deviations		15		− 197	− 197		+ 32	− 1,187	− 8

As can be seen a cost reduction of 8% or for the actual case over a billion francs can be achieved. Moreover, 16% is saved in time.

A breakwater is in most cases the first part of a harbour development and almost by definition the most critical from a construction point of view. The necessary time for the construction of breakwaters is often critical – especially on exposed locations – and might influence the total costs significantly. The design has great influence on this. From Table 2 it was seen that the possibility of using heavier core material during rougher sea states reduced the construction time by 16%. The following example illustrates the influence of armour unit weight on construction time.

In a conventional design the seaward slope of a rubble mound breakwater is often protected by concrete blocks. As these blocks tend to be heavy and the placing has to be done very carefully in a predescribed way, the progress of the dam front is defined by the time needed to place the blocks. Each block takes about the same time to place independent on the weight (within a fairly wide range) and thus the progress depends on the number of blocks per meter dam to be placed. A steep slope with a small number of heavy blocks permits a quicker progress than a gentler slope with a greater number of smaller blocks.

For instance a slope of 1 : 1.5 with cubic blocks of 30 tons and a slope length of 28 m has 6.1 blocks per m breakwater. A slope of 1: 2 with an equivalent block of 22.5 tons needs 9.1 block per m breakwater. With the same frequency for the placing of the armour units the second concept consumes 50% more time. This leads to a difference of 50% in construction time or would require an additional crane for placing rocks. In some cases it might even be profitable to overdesign the block weight, thus reducing the number of blocks to be placed, to save time.

These considerations tend to give preference to heavy armour units on steep slopes. Of course heavier blocks might require a bigger and more expensive crane, which put restrictions to the argument.

If landbased equipment is used for the placement of the armour the width of the breakwater should be determined with due consideration to the possibility of establishing a construction road wide enough to allow stone dumpers to pass the crane and to turn. In the case of Zeebrugge breakwater, Fig. 1, the 10 m wide crest of the completed structure is not sufficient in this respect. Therefore, either a wider structure must be designed or a construction road established at a lower level. However, a lower level means more down-time due to overtopping. A compromise was found at level + 6.8 m where the total width of the core plus the adjacent filter layers is 13.7 m enabling an American Hoist 11.310 crane to work and dumpers to pass, Fig. 2.

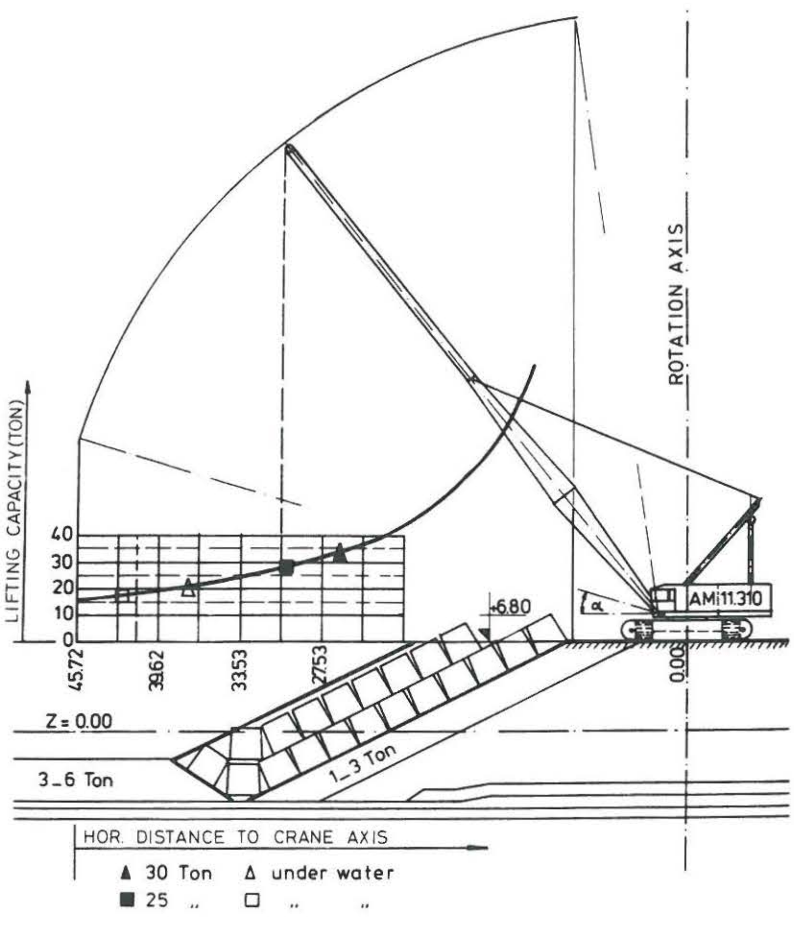

Fig. 2. Construction road, Zeebrugge breakwater.

This lower level also reduced the reach of the crane necessary to place the concrete blocks at the toe of the slope. Note that if the fine material from the surface of a construction road fill to voids in the filter layers it should be removed before completion of the structure.

3. MAINTENANCE AND REPAIR

When damage occurs the methods, the time and the costs of repairs are dictated by the original design. For this reason the design should include also considerations of repair of the structure. Distinction should be made between regular maintenance work and incidental major repairs. The most important elements that play a role to ease the job are given in the table with their relative importance for maintenance and major repairs.

	Maintenance	Major repair
Maintenance prescriptions	x x	–
Accessibility from land or water side	x x	x
Non-specialized equipment needed	x x	x
Materials available	x x	x
Funds	x	x x

Provision of good accessibility, either for land based or floating equipment is essential. The width of access roads and cap structures should cope with the equipment necessary and available for the placing of armour blocks. For floating equipment, the water depth and the exposure are very important factors. For each type of breakwater the sections can be evaluated by drawing the accessibility and tonmeter graphs as exemplified in Fig. 3.

For land based equipment increasing tonmeters ask for bigger cranes and wider roads. Floating equipment tends to be more expensive than land based equipment, especially on the seaward side of the breakwater where workability is more limited.

The ideal case is that the local authorities or contractors have the necessary, non-specialized equipment to maintain and repair the breakwater.

Preferably locally available materials should be used. Pre-cast armour blocks for maintenance can be made in the stock. The client could buy some casings for this purpose.

Although there might be a specific design criterium for the *crest width* it is relevant to evaluate this width as function of the accessibility for construction and future repairs.

The elements to be considered are:

- the costs of placing armour units with a land based crane
- the costs of placing armour units with a floating crane
- the costs of increased or decreased dam width

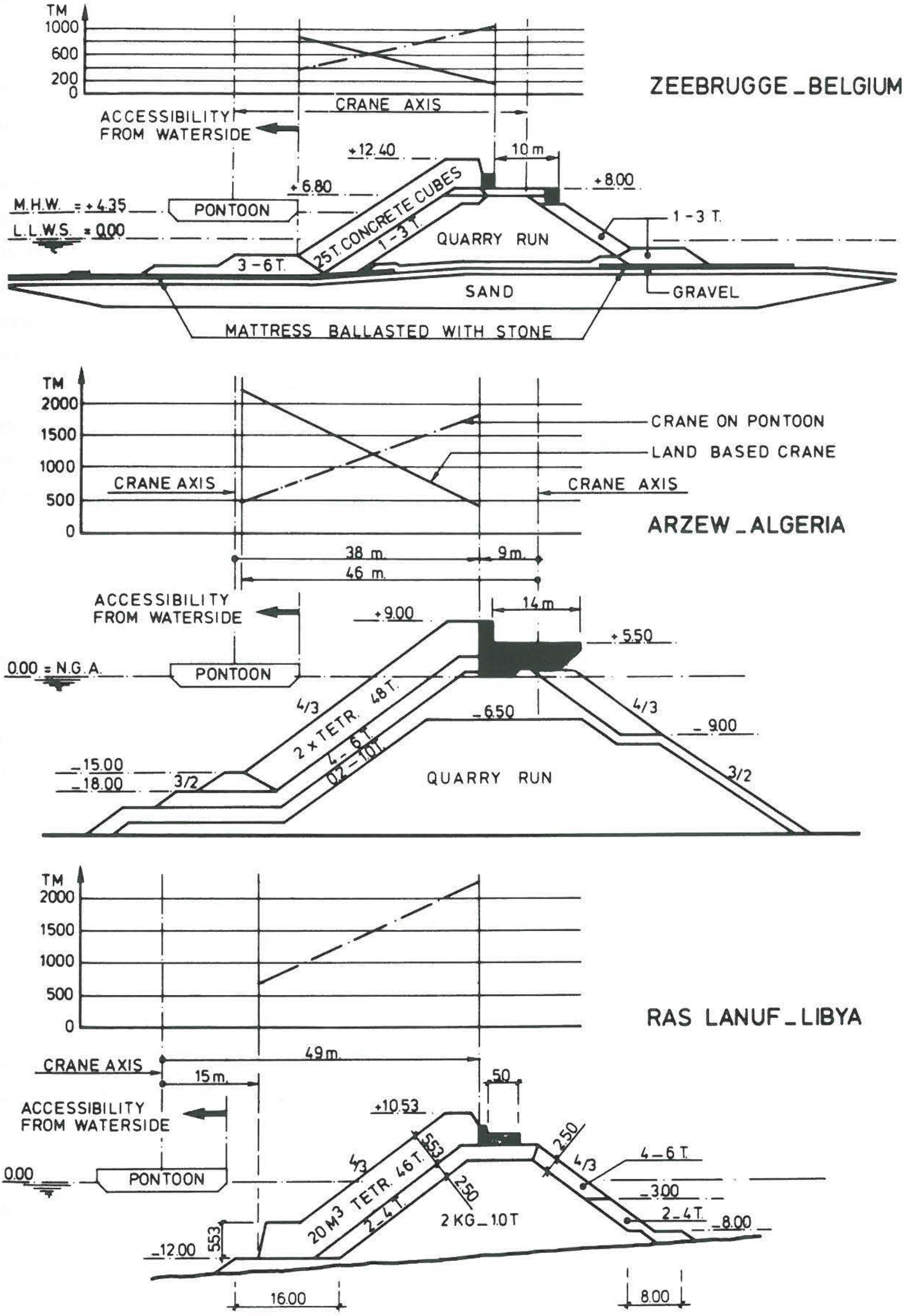

Fig. 3. Example of accessibility evaluation of rubble mound breakwaters.

The costs of placing armour units depend on the equipment, the labour costs and the production. Typical sets of equipment are listed in the following Table 3.

Table 3. Comparison of landbased and floating cranes.

Floating crane		Landbased crane
- crane, placing armour		- crane, placing armour
- crane barge		
- pontoon for transport (1 or 2 units)		
- crane, loading pontoon		
- trucks, short hauling distance		- trucks, long hauling distance
- crane, loading trucks		- crane, loading trucks
operational costs	200 - 300 %	100 %
workability	40 - 70 %	80 %
unit rate	300 - 500 %	100 %

As can be seen the unit rate for placing armour from the exposed seaward side is at least three times more expensive than using a landbased crane. However, working with a land based crane requires a crest width of at least 10-14 m for medium size breakwaters.

The answer to the question of the economic width of the crest depends on each individual case but there is a strong tendency for narrow crests as can be shown with an example.

We assume a breakwater with the following properties

- waterdepth 12 m below datum
- crest at 8 m above datum
- armour units, modified cubes, 7 units/m breakwater
- design criterium 2-5 % damage for design conditions

and an estimated accumulated armour layer damage over the lifetime of approximately 3 times the damage under design conditions, i.e.

$3 \cdot 4\% = 12\%$ or $0.12 \cdot 7 = 0.84$ units/m breakwater.

The repair costs may be the double of the construction costs but as these can be discounted over the lifetime, the unit rate to be considered for repair, will not differ much from the initial construction costs. Thus for construction and repair 7 + 0.84 units have to be placed. When the unit rate floating = 4 x unit land rate land based, the difference equals 3 x unit rate land based placing (3 x B).

This has to be compared with the costs of the width of the breakwater. When the breakwater in its centre consists vertically of 18 m quarry rock covered with 2 m concrete, 1 m of the width costs

18 m³ rock x R + 2 m³ concrete x C, where for example

B = 100 US $ for transport and placing of one armour unit
R = 35 US $ for supply and placing stone per m³
C = 100 US $ for 1 m³ concrete

B, R and C depend of course on local circumstances.

The costs of 1 m extra width is then $18 \cdot 35 + 2 \cdot 100 = 830$ US $

The difference in costs between floating and landbased placing is $7.84 \cdot 3 \cdot 100 = 2352$ US $ per m breakwater.

Thus this example shows that the difference between floating and landbased placing of armour equals roughly 3 m of width. However, for landbased placing the crest must be 10-14 m wide. Thus the conclusion can be drawn that if there is no need for a wide crest (traffic or other) the economics tend to support a narrow crest.

Coming back to the examples of Zeebrugge, Arzew and Ras Lanuf, as shown in Fig. 3, Zeebrugge and Arzew have wide crests both dictated by traffic considerations. Ras Lanuf has a narrow crest of only 5 m which in that case is an economical solution as well.

4. COST OPTIMIZATION

The final design should represent a cost minimum, Fig. 4. Capital investments can be calculated rather accurate whereas maintenance and repair costs are much more difficult to estimate.

If we design for condition A we might find ourselves in B if the maintenance and repair costs are underestimated. A flat curve for maintenance and repair costs will restrict the economic risk.

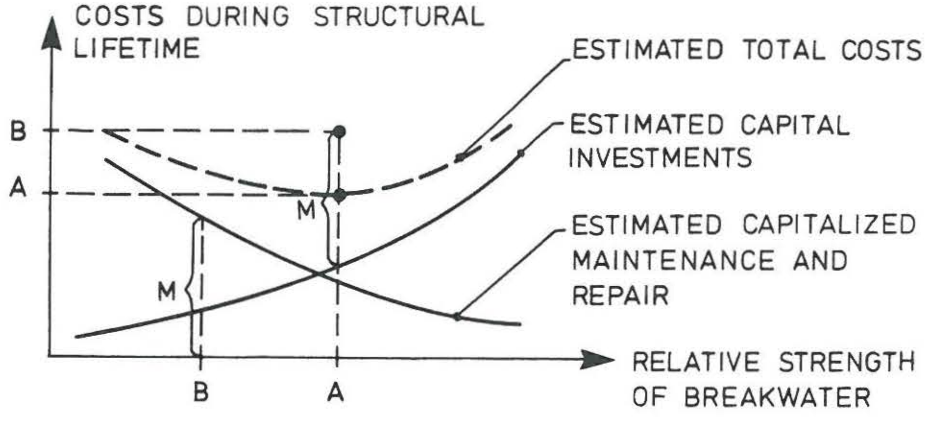

Fig. 4. Cost optimization.

On the other hand, when the curve for capital investment is flat, overdesign might pay in reducing the risk of underestimated repairs.

For a quantification of the principles outlined above a stocahstic model should preferably be used (Nielsen and Burcharth, 1983).

Although the armour layer forms a minor part of the total costs, the choise of its elements has a great influence because it affects not only the construction costs but also the construction period and the maintenance costs.

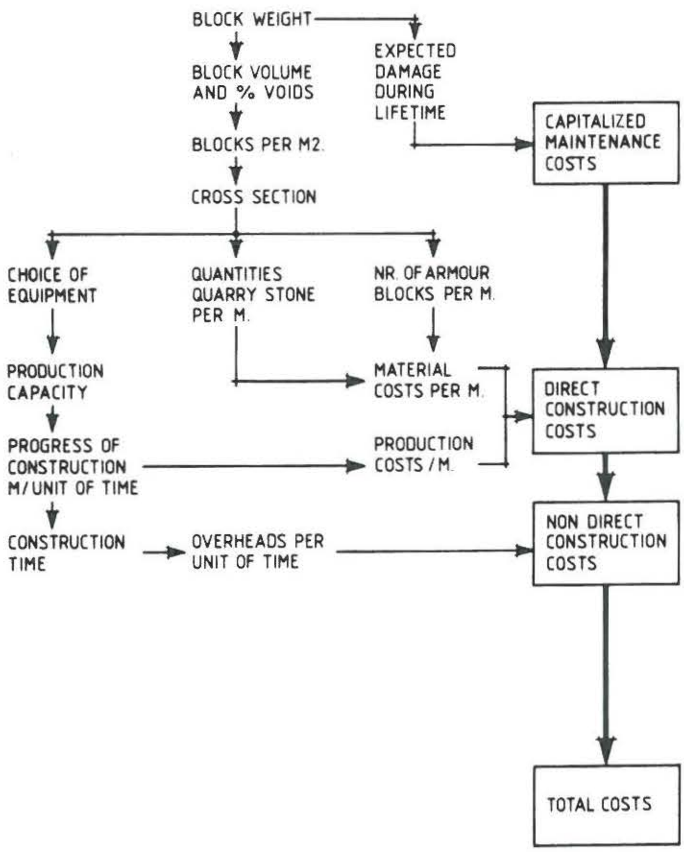

Fig. 5. Influence of choice of armour block on breakwater costs.

If for instance a 25 t cube would suffice for stability reasons an overdesign to 30 t cubes would give the following result.

Unit weight armour	ton	25		30	
Length of breakwater	m	4000		4000	
Expected damage during lifetime in number of armour units		1250		221	
Capitalized maintenance costs	mill. BEC		217		45
Block volume	m^3	10.4		12.5	
% voids armour layer	%	45		45	
Armour units per m^2 of slope		0.254		0.225	
Number of armour units per m breakwater		7.1		6.3	
Quantity of quarry stone per m breakwater	ton	900		900	
Material costs per m breakwater	mill. BEC	0.836		0.854	
Progress of construction in m per month	m/month	80		90	
Production costs per m breakwater	mill. BEC	0.327		0.322	
Direct construction costs per m breakwater	mill. BEC	1.163		1.176	
Direct construction costs (4000 m)	mill. BEC		4652		4704
Construction time for 4000 m	months	50		45	
Overheads per month	mill. BEC/month	14		14	
Overheads for 50 months (4000 m)	mill. BEC		700		630
Total costs for 4000 m breakwater	mill. BEC		5569		5379

Expressed in percentages the result gives	25 t	30 t
capitalized maintenance costs	4%	1%
direct construction costs	84%	85%
total costs after overheads	100%	97%
construction period	100%	90%

In this special case the overdesign would save 3% in money and 10% in time.

The constructional strength/resistance, expressed in terms of significant wave height, will be increased by a factor of $(30/25)^{1/3} = 1.063$ and the capitalized maintenance costs will be reduced to $\frac{45}{217} \cdot 100\% = 21\%$.

5. DAMAGE PREVENTION

Due to limitations of knowledge regarding wave climate and structural response there is always a chance of damage or failure. The client responsible for the maintenance of the structure should have a "user's guide" which inform him about the vital parts of the structure and the maximal allowable deterioration of these parts. Also he should be told at what external conditions certain effects might be expected. Regular surveying and special surveys after extreme conditions should be prescribed (survey to survive).

The basis for such a user's guide is the knowledge about the residual strength (resistance) of the breakwater under various levels of deterioration such as displacements and breakage of armour units, sea bed scour, deterioration of rock and concrete materials, displacements of superstructures etc.

This knowledge can be provided by much more extensive model test programs than generally used today because such programs involve many stages of partial failures and also involve material aspects. In principle diagrams as sketched in Fig. 6 should be produced.

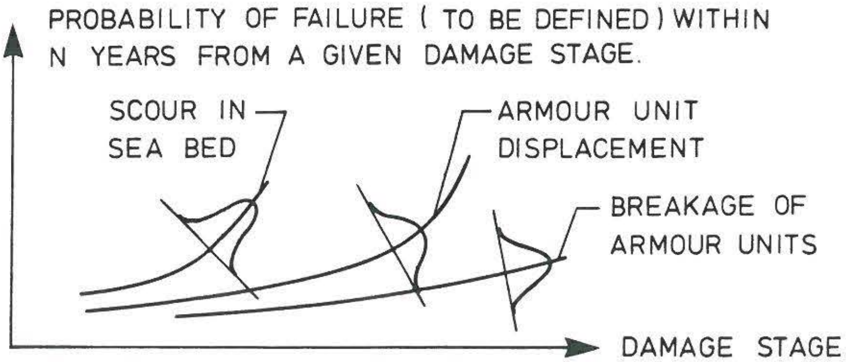

Fig. 6. Qualitative example of diagrams for evaluation of different single modes of partial damage.

The obvious possibility of co-existence of several damage modes makes it necessary also to produce at least some characteristic diagrams and tables for the evaluation of joint partial damages, Fig. 7.

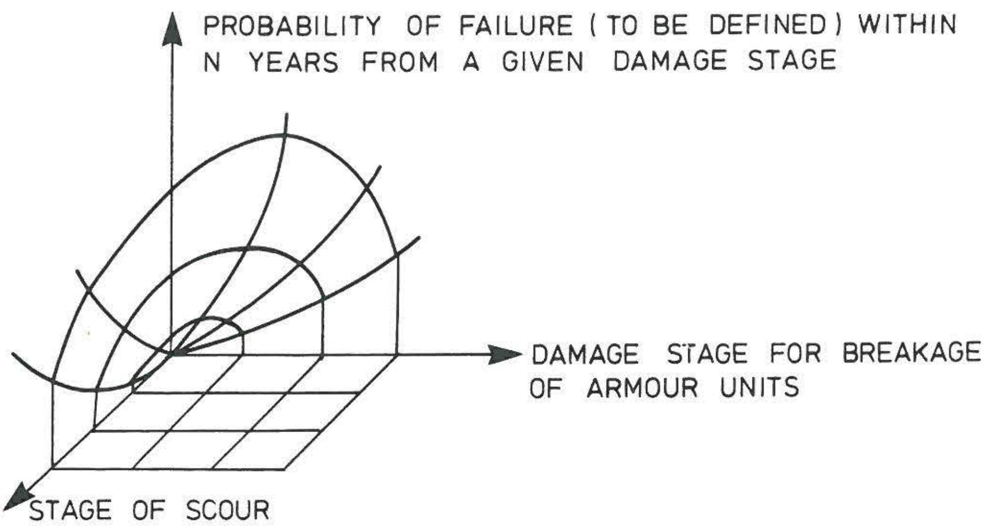

Fig. 7. Qualitative example of diagram for evaluation of two co-existent partial damage modes.

The damage evaluations are of course very dependent on the type of breakwater. At least three types of structures can be identified due to significant differences in sensitivity to exceedence loads, Fig. 8.

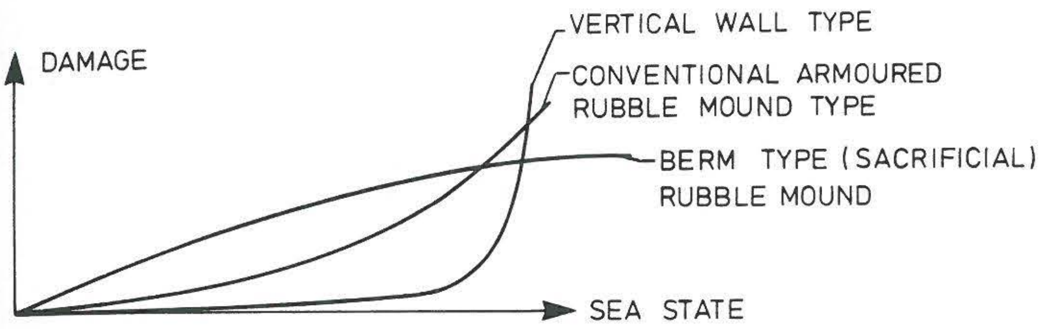

Fig. 8. Illustration of comparative damage sensitivity of various types of breakwaters.

6. REFERENCES

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