On Optimum Design of Breakwaters

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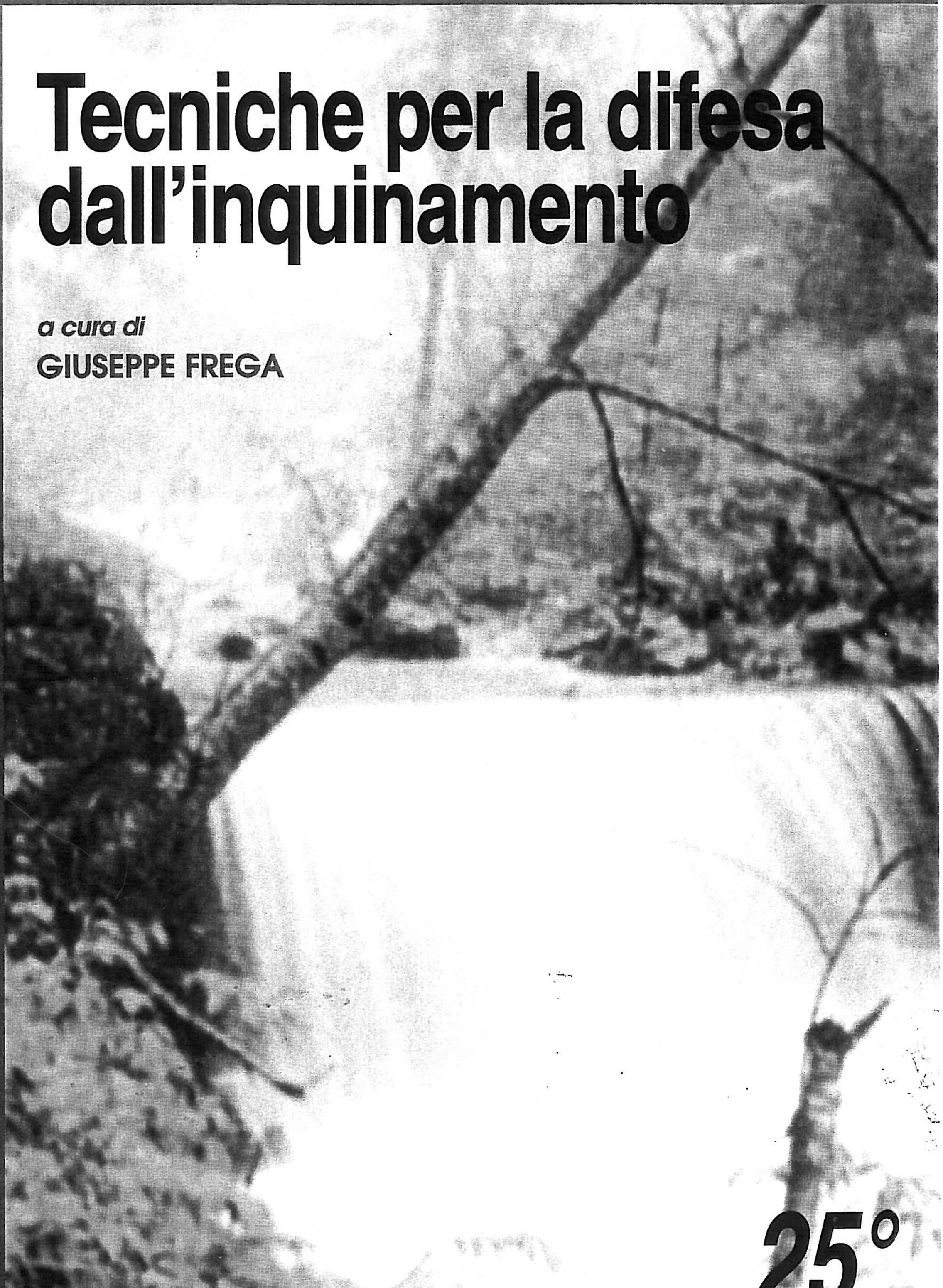
a cura di

GIUSEPPE FREGA



25°

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MAGGIO 2004

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DIPARTIMENTO DI DIFESA DEL SUOLO

ASSOCIAZIONE NAZIONALE DI INGEGNERIA SANITARIA
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a cura di
Giuseppe Frega

EDITORIALE BIOS

SI RINGRAZIANO

- Il P.I. Giuseppe Cammarano, l'Ing. Giorgio Bernabò, il Geom. Francesco Leone, il Tecnico Francesco Bonanno e Antonio Pantusa per l'organizzazione;
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PREFAZIONE

Il corso di aggiornamento del 25° anniversario ha compreso molti argomenti venuti all'attenzione dell'area scientifica che si occupa delle problematiche tecniche legate all'ambiente. Il ruolo di tali problematiche ormai non si esaurisce più solo nelle analisi, ma investe ogni fase delle elaborazioni progettuali partendo dagli studi preliminari che devono individuare limiti, opportunità, alternative correttamente basate su approfondita e collaudata conoscenza specifica tecnica.

Né va dimenticato che allo stato attuale siamo ancora in presenza di un quadro normativo estremamente vario e la possibilità di puntualizzare le nuove prospettive offerte dalla rivisitazione legislativa può aiutare ad evitare confusioni sulle prime applicazioni.

Al miglioramento della capacità professionale di tutti coloro che si interessano delle tematiche tecniche volte alla difesa dell'inquinamento si rivolge quindi questa 25ª fatica editoriale, per la quale ringrazio l'Editore e tutti i collaboratori coinvolti

Università della Calabria, giugno 2005

Il Direttore del Corso

Prof. Ing. Giuseppe Frega

ON OPTIMUM DESIGN OF BREAKWATERS

H.C. Hans F. Burcharth

Aalborg University, Denmark

SOMMARIO

La memoria propone una procedura per la corretta progettazione di una diga frangiflutti di difesa portuale. La scelta del tipo di frangiflutti da adottare viene guidata oltre che dai desiderati aspetti funzionali, anche da altri fattori quali il livello di sicurezza auspicato ed i costi di manutenzione e di "downtime". Alla luce delle norme ISO 2394, viene proposto un metodo di ottimizzazione dei costi e se ne illustra l'applicazione per alcuni casi di studio (Follonica e Sines).

Introduction

Because breakwaters belong to the most important and costly coastal structures it is of importance to optimise the design with respect both to function and lifetime costs.

The paper presents a short discussion on choice of type of breakwater and functional aspects. The main content of the paper is an analysis of the optimum safety level for rubble mound breakwaters.

Selection of type of breakwater

The main types are shown in figure 1.

The optimum choice of type is very complicated, as wide ranges of both functional and environmental aspects must be considered.

The main aspects to consider are listed below.

- Function (access road, installations, berths, wave reflection, and wave transmission);
- Water depth;
- Wave climate;
- Material availability;

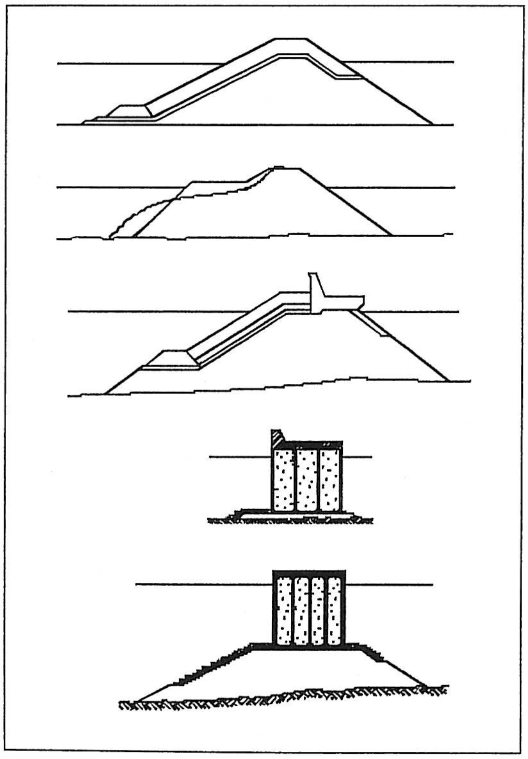


Fig. 1 - *Main types of breakwaters*

- Foundation conditions (geotechnics);
- Conditions for construction (wave exposure, space on land/sea, transport of materials, equipment, skill);
- Urgency in construction;
- Ecological impact;
- Visual impact;
- Tectonic activity.

The traditional advantages and disadvantages between rubble mound structures and monolithic structures like caissons are listed below.

Advantages of rubble mound structures:
- Construction simple in principle;
- Ductile damage development;
- Not sensitive to differential settlements.

Constraints and disadvantages of rubble mound structures:
- Availability of adequate quarry needed;
- Large quantity of material required in deeper water;
- Transport of large amounts of materials;
- Large space required for storing of materials (rock and concrete units);
- Separate structure needed to establish moorings.

Advantages of caisson structures:
- Construction time short on location;
- Relatively small amounts of materials for structures in deeper water;
- Moorings are easily established along the caissons.

Disadvantages of caisson structures:
- Brittle failures;
- Sensitive to poor subsoil foundation conditions in terms of weak soils prone to large settlements by consolidation;
- High reflection of waves.

However, because of the large number of aspects there are no simple rules for the choice of type of breakwater. The actual combination of the functional and environmental conditions vary considerably from location to location and determines in practice the final choice.

Layout considerations

Morphological impacts depend mainly on the layout of the breakwater, but also wave reflection characteristica of the structures are of importance. Figure 2 illustrates some common impacts.

Note that downdrift erosion is determined almost solely by the lay-out and is hardly influenced by the type of structure.

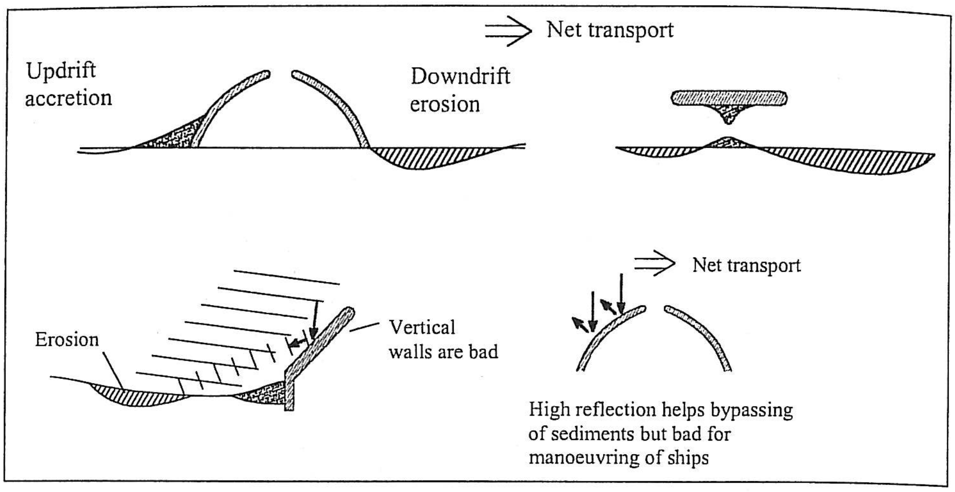


Fig. 2 - *Illustration of potential morphological impacts of breakwaters*

Functional criteria related to overtopping

Overtopping criteria in terms of allowable amounts of overtopping water are very important for the design of the breakwater cross section as both the crest level and the seaward profile influence the overtopping.

The allowable overtopping rates depend on the function of the breakwater and on the design limit states. As discussed in a later paragraph at least a serviceability limit state (SLS) and an ultimate limit state (ULS) must be considered.

Figure 3 shows a proposal for a functional classification of breakwaters and the related limit state overtopping criteria. For classes I and II the primary criteria are given as a maximum transmitted significant wave height $H_{s,t}$ caused by overtopping. Typical values of the related time averages overtopping rates are given. More detailed figures could be given, dependent on the local conditions. The Coastal Engineering Manual (2002) provides figures for allowable overtopping rates.

It is not only the average overtopping rate which is of importance. Actually the volume of water in the single overtopping wave is more important for the impact on traffic, installations and structures. However, only very approximate values of single wave overtopping and critical values exist.

Of significant importance for the use of the hinterland is also the spatial distribution of the overtopping water behind the breakwater. Fortunately most of the water splashes down within a relatively short distance from the seaward shoulder of the structure signifying an exponential decay with distance. However, the decay depends very much on the cross sectional geometry of the breakwater as well as on the wavelength.

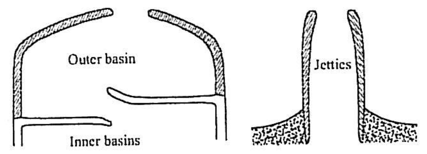
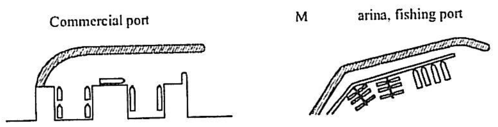
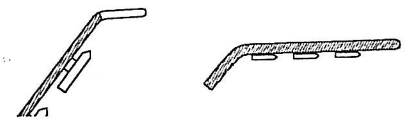
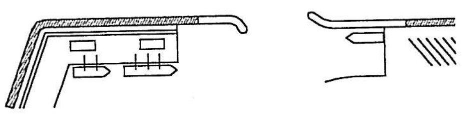
I		Wave transmission SLS: $H_{s,T} = 0.5 - 1.8$ m Damage to main armour SLS: D = 5 %, RLS: D = 15 % ULS: D = 30 % Sliding distance of caissons SLS: 0.2 m, ULS: 2 m
II		Wave transmission SLS: $H_{s,T} = 0.3 - 1.0$ m Damage to main armour SLS: D = 5 %, RLS: D = 15 % ULS: D = 30 % Sliding distance of caissons SLS: 0.2 m, ULS: 1 m
III		Wave overtopping SLS: $q = 10^{-5} - 10^{-4}$ m³/ms ULS: $q = 10^{-3} - 10^{-2}$ m³/ms Damage to main armour SLS: D = 5 %, RLS: D = 15 % ULS: D = 30 % Sliding distance of caissons SLS: 0.0 m, ULS: 0.5 m
IV		Wave overtopping SLS: $q = 10^{-5}$ m³/ms ULS: $q = 10^{-2}$ m³/ms Damage to main armour SLS: D = 5 %, RLS: D = 15 % ULS: D = 30 % Sliding distance of caissons SLS: 0.0 m, ULS: 0.5 m

Fig. 3 - *Proposal for functional classification of breakwaters and related limit state design criteria for overtopping*

Figures 4 and 5 illustrate this in showing examples of spatial distribution of overtopping based on physical model tests (without wind) with different cross sections at The Hydraulics and Coastal Engineering Laboratory, Aalborg University. Figure 4 illustrates optimization of cross sections with different water depths to obtain equal overtopping characteristica.

Design overtopping return period	Overtopping design criteria		
	q (litre/ms)	H_s0 (m)	Tp (s)
1 year 25 year	0.10 1.00	4.50 8.50	13.0 14.4

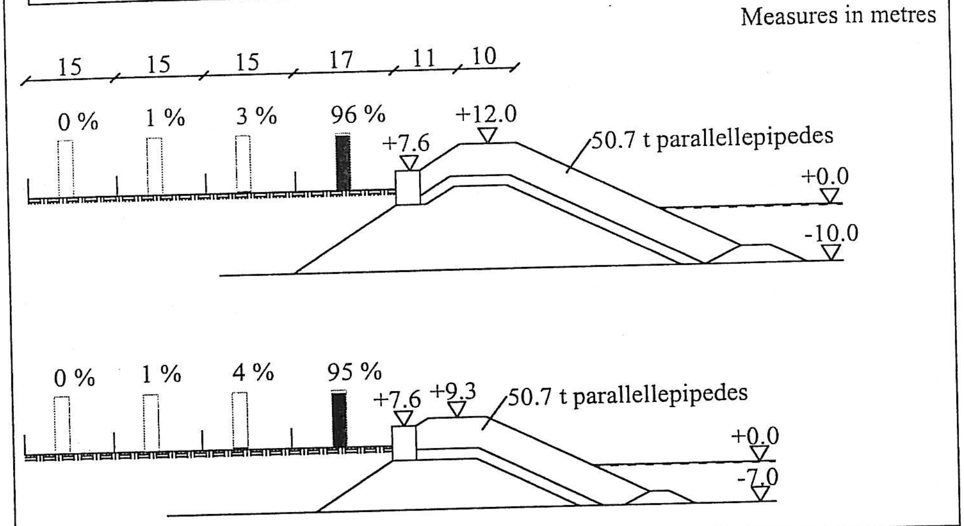


Fig. 4 - *Spatial distribution of overtopping behind conventional rubble mound breakwaters for the Genova Voltri Port coastal protection. Hydraulics & Coastal Engineering Laboratory, Aalborg University, Denmark*

Figure 5 shows the influence on the spatial distribution of overtopping reservoirs in front of a vertical wall.

Overtopping formulae are rather uncertain. For this reason physical model tests are recommended. It should be noted that scale effects cause underestimation of small overtopping rates in case of rubble mound slopes. The flatter (less steep), the larger the scale effect. For this reason it is recommended to correct all model test values below 1 l / ms by using the trend in an appropriate overtopping formulae from app. 1 l / ms and downwards.

Design for construction

The potential method(s) of construction should be reflected in the design in order to reduce construction costs. Basically, the work should proceed unhindered under normal sea state conditions and the risk of major damage when severe sea states occur should be minimized.

Design overtopping return period	Overtopping design criteria		
	q (litre/ms)	H_{s0} (m)	T_p (s)
1 month	0.01	4.50	15.0
1 year	0.10	5.06	16.0
25 year	1.00	6.71	18.0

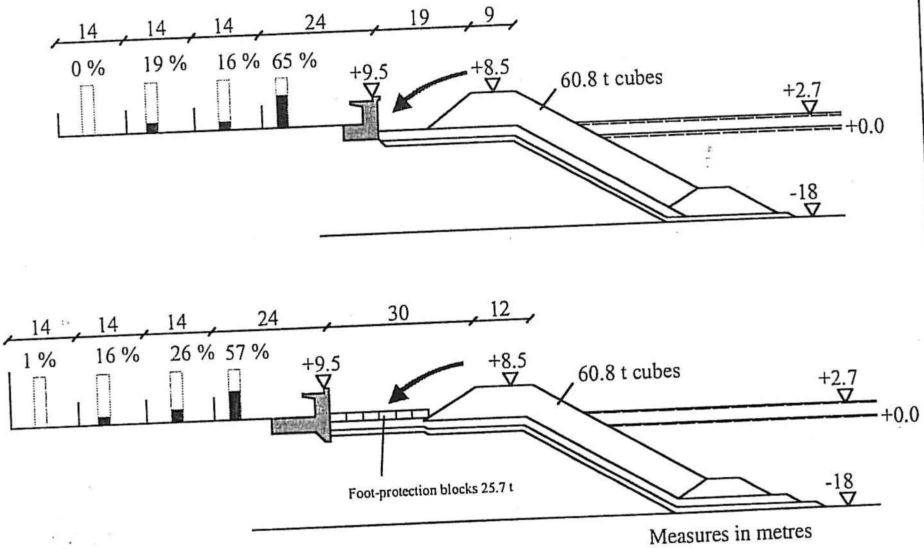


Fig. 5 - *Spatial distribution of overtopping behind breakwaters with front reservoirs for the extension of Port of Agaeta, Grand Canaria. Hydraulics & Coastal Engineering Laboratory, Aalborg University, Denmark*

For rubble mound breakwaters the unprotected part is always at risk. For this part a flexibility to use coarser core material and coarser under layer material in case of approaching bad weather can save a lot of money, basically by reducing downtime, which more than compensates the extra material costs (Rietveld and Burcharth, 1987).

Another example is the top level of the core material which should preferably be at a level high enough to be used as construction road if land based construction equipment is going to be used.

Basically one must consider the various construction phases and relate them to the local wave-water level condition, and if necessary adjust the design to ease construction.

Safety levels based on economical optimization

Background

When the type of breakwater has been selected for further investigations one has to select a safety level for the design. Most national standards and recommendations introduce overall safety factors on resistance to a specific return period sea state. No safety levels are given in terms of acceptable probability of certain damage within service life of the structure. The exception is the Spanish Recommendations for Maritime Structures, in which the given safety levels depend on the functional and economic importance of the breakwater. However, these safety levels must be regarded as tentative as they are not based on more systematic investigations.

Objective of present study

The objective is to identify the safety levels related to minimum total costs over the service life. This includes capital costs, maintenance and repair costs, and downtime costs. Figure 6 illustrates the principle.

The present analysis deals with rubble mound breakwaters without superstructures.

ISO prescription

The ISO-Standard 2394 on Reliability of Structures demands a safety-classification based on the importance of the structure and the consequences in case of malfunction. Also, for design both serviceability limit state (SLS) and an ultimate limit state (ULS) must be considered, and damage criteria assigned to these limit states. Moreover, uncertainties on all parameters and models must be taken into account.

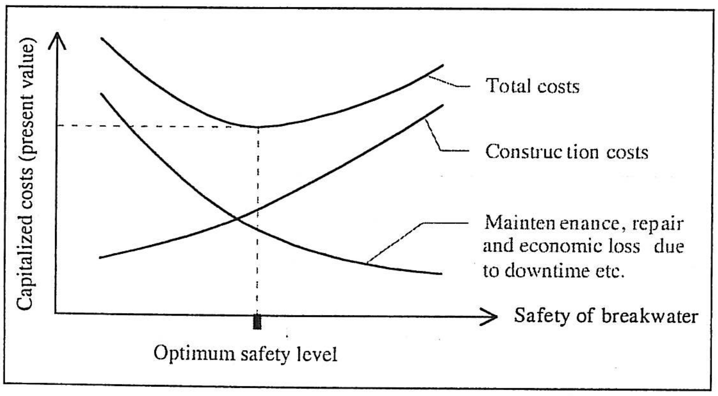


Fig. 6 - *Illustration of principle in cost optimization*

In the present study are analysed the influences on optimum safety levels of:

- Real interest rate, inflation included;
- Service lifetime of the breakwater;
- Downtime costs due to malfunction;
- Damage accumulation.

Safety classes for coastal structures

In general there is very little risk of human life due to damage of a breakwater. This is not the case for many other civil engineering structures. For this reason it is not convenient to use existing classifications.

A suitable classification could be the one given in Table 1.

Normal breakwaters would belong to very low and low safety classes. A sea dike protecting low land with populations would belong to high safety class.

Procedure in numerical simulation for identification of minimum cost safety levels

The analysis is performed by the use of simulation technique. The procedure used is as follows:

- Select type of breakwater, water depth and long-term wave statistics.
- Extract design values of significant wave height H^{sT} and wave steepness corresponding to a number of return periods, T = 5, 10, 25, 50, 100, 200 and 400 years.
- Select service lifetime for the structure, e.g. T^L = 25, 50 and 100 years.
- Design by conventional deterministic methods the structure geometries corresponding to the chosen H^{sT} -values.
- Calculate construction costs for each structure.
- Define repair policy and related cost of repair.
- Specify downtime costs related to damage levels.
- Define a model for damage accumulation.
- For each structure geometry use stochastic models for wave climate

Safety class	Consequences of failure
Very low	*No* risk of human injury. *Small* environmental and economic consequences
Low	*No* risk of human injury. *Some* environmental and/or economic consequences
Normal	*Risk* of human injury and/or *significant* environmental pollution or *high* economic or political consequences
High	*Risk* of human injury and/or *significant* environmental pollution or *very high* economic or political consequences

Table 1 - *Example of safety classes for coastal structures (Burcharth, 2000)*

and structure response (damage) in Monte Carlo simulation of occurrence of damage within service lifetime.

- Calculate for each structure geometry the total capitalized lifetime costs for each simulation. Calculate the mean value and the related safety levels corresponding to defined design limit states.
- Identify the structure safety level corresponding to the minimum total costs.

Cross sections

The analysed cross sections are shown in Figura 7.

Only rock and concrete cube armour considered.

Crest levels are determined from a criteria of max. transmitted Hs = 0.50 m by overtopping in a sea state with return period equal to service life, and a criteria of having the core crest level at minimum 1.5 m above the water level for the purpose of a construction road.

Repair policy and cost of repair and downtime

Table 2 shows the applied damage levels and repair policy related to the design limit states. Besides SLS and ULS, as demanded by ISO, a repairable limit state RLS is introduced as well. RLS is here defined as the maximum damage level, which allows foreseen maintenance and repair methods to be used.

In costs of repair is taken into account higher unit costs of repairs and mobilization costs.

Repairs are initiated shortly after exceedence of the defined damage limits.

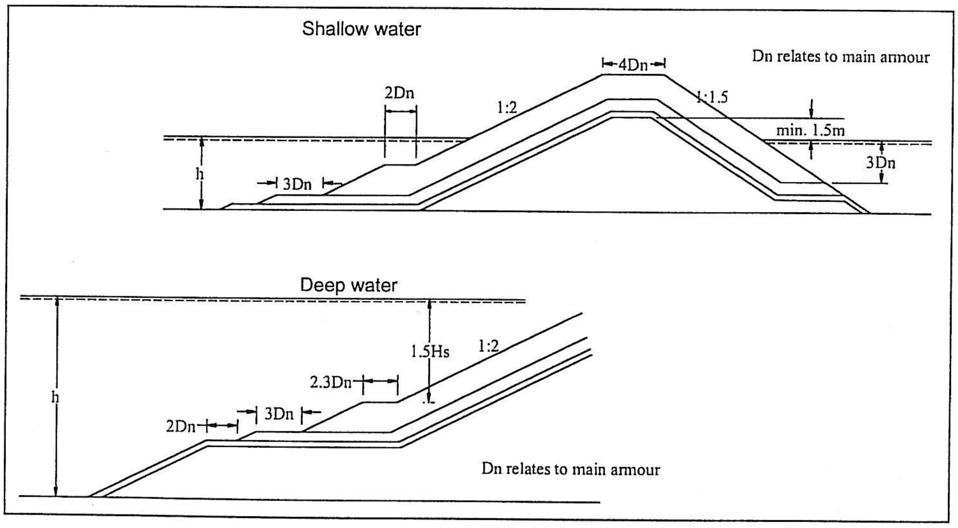


Fig. 7 - *Shallow and deep water cross sections used in the analyses*

Damage levels	S (rock)	N^{od} (cubes)	Estimated D	Repair policy
Initial	2	0	2%	no repair
Serviceability (minor damage, only to armour)	5	0.8	5 %	repair of armour
Repairable (major damage, armour + filter 1)	8	2.0	15 %	repair of armour + filter 1
Ultimate (failure)	13	3.0	30 %	repair of armour + filter 1 and 2

Table 2 - *Repair policy*

Downtime costs related to 1 km of breakwater is set to 18,000,000 Euro when the damage to the armour exceeds 15% displaced blocks.

Damage accumulation model

The simulations are performed with and without damage accumulation. The damage accumulation model is based on the following principles:

- Storms with Hs causing damage less than a set limit do not contribute.
- Accumulation takes place only when the next storm has a higher Hs ñ value than the preceding value.
- The relative decrease in damage with number of waves (inherent in the Van der Meer formulae for rock armour) is included.

Formulation of cost functions

All costs are discounted back to the time when the breakwater is built. The optimization problem is formulated as follows:

$$\min_{T} \quad C(T) = C_I(T) + \sum_{t=1}^{T_L} \left\{ C_{R_1}(T) P_{R_1}(t) + C_{R_2}(T) P_{R_2}(t) + C_F(T) P_F(t) \right\} \frac{1}{(1+r)^t}$$

where

T	return period used for deterministic design
T^L	design life time
$C^I(T)$	initial costs (building costs)
$C^{R1}(T)$	cost of repair for minor damage
$P^{R1}(t)$	probability of minor damage in year *t*
$C^{R2}(T)$	cost of repair for major damage
$P^{R2}(t)$	probability of major damage in year *t*
$C^F(T)$	cost of failure including downtime costs
$P^F(t)$	probability of failure *t*
r	real rate of interest

Case studies

Three cases are presented in the present paper. The basic data are given in Table 3.

Case	Water Depth	Armour density	Waves		Stability formula	Built-in unit prices core/filter 1/ filter 2 /armour [EURO/m3]
			Orgin $H_{S,o}^{100\,y}$	Distribution $H_{S,o}^{400\,y}$		
1.1	8 m	Rock 2.65 t/m3	Follonica 5.64 ml	Weibull 6.20 m	van der Meer (1988)	10 / 16 / 20 / 40
1.3	15 m	Cube 2.40 t/m3	Follonica 5.64 ml	Weibull 6.20 m	van der Meer (1988) modified to slope 1:2	10 / 16 / 20 / 40
2.3	30 m	Cube 2.40 t/m3	Sines 13.2 m	Weibull 14.20 m	van der Meer (1988) modified to slope 1:2	5 / 10 / 25 / 35

Table 3 - *Case study data*

Results

The results of the three case studies are given in the following Tables 4 - 6 and Figures 8 - 10.

Conclusions related to cost optimization of rubble mound breakwaters

The main conclusions of the analyses can be summarized as follows:

Real Interest Rate	Downtime costs	Optimum design data				Optimum limit state average number of events within service lifetime			Construction costs for 1 km length	Total lifetime costs for 1 km length
		Optimized armour unit mass W_{50}	Optimum design return period T	H_s^T	Free-board Rc	SLS	RLS	ULS		
(%)		(t)	(years)	(m)	(m)				(1,000 EURO)	(1,000 EURO)
2	0	20	> 6	4.40*	6.9	0.00	0.000	0.000	12,529	12,529
5	0	16	> 6	4.40*	6.5	1.44	0.012	0.000	11,419	12,388
8	0	14	> 6	4.40*	6.3	2.94	0.038	0.004	10,699	11,995
2	200,000 EURO per day in 3 months	20	> 6	4.40*	6.9	0.00	0.000	0.000	12,529	12,529
5		18	> 6	4.40*	6.7	0.64	0.004	0.000	11,988	12,452
8		16	> 6	4.40*	6.5	1.44	0.008	0.007	11,419	12,066
* Depth limited waves. H_s = 4.40 m corresponds to 6 years return period.										

Table 4 - *Case 1.1. Optimum safety levels for rock armoured outer breakwater. 50 years service lifetime. Shallow water depth 8 m. Damage accumulation included*

Real Interest Rate (%)	Downtime costs	Optimum design data				Optimum limit state average number of events within service lifetime			Construction costs for 1 km length	Total lifetime costs for 1 km length
		Optimized design return period, T (years)	H_s^T (m)	Optimum armour unit mass W (t)	Free-board Rc (m)	SLS	RLS	ULS	(1,000 EURO)	(1,000 EURO)
2	None	400	6.20	12.5	6.3	1.11	0.008	0.001	17,494	19,268
5		200	5.92	10.9	6.0	1.84	0.015	0.003	16,763	18,318
8		100	5.64	9.5	5.8	2.98	0.031	0.008	16,038	17,625
2	200,000 EURO per day in 3 months	400	6.20	12.5	6.3	1.11	0.008	0.002	17,494	19,391
5		200	5.92	10.9	6.0	1.82	0.015	0.004	16,763	18,453
8		100	5.64	9.5	5.8	2.98	0.031	0.008	16,038	17,821

Table 5 - *Case 1.3. Optimum safety levels for concrete cube armoured breakwater. 50 years service lifetime. 15 m water depth. Damage accumulation included*

Lifetime (years)	Real Interest Rate (%)	Optimum design data				Optimum limit state average number of events within structure lifetime			Construction costs for 1 km length	Total lifetime costs for 1 km length
		Optimized design return period, T (years)	H_s^T (m)	Optimum armour unit mass W (t)	Free-board Rc (m)	SLS	RLS	ULS	(1,000 EURO)	(1,000 EURO)
50	2	1000	14.7	168	14.8	1.21	0.008	0.001	76,907	86,971
	5	400	14.2	150	14.8	1.84	0.016	0.003	73,722	81,875
	8	100	13.2	122	14.8	3.39	0.052	0.012	68,635	78,095
100	2	1000	14.7	168	15.4	2.68	0.013	0.002	78,423	93,440
	5	400	14.2	150	15.4	3.90	0.029	0.005	75,201	84,253
	8	200	13.7	136	15.4	5.28	0.056	0.011	72,675	79,955

Table 5 - *Case 2.3. Optimum safety levels for concrete cube armoured breakwater. 30 m water depth. 50 years and 100 years lifetime. Damage accumulation included. Downtime costs of 200,000 EURO per day in 3 month for damage D > 15%*

- The results show that optimum safety levels are higher than the safety levels inherent in conventional deterministic designs, especially in the case of low interest rates.
- Further, the results show that for the investigated type of breakwater the critical design limit state corresponds to Seviceability Limit State (SLS) defined by moderate damage to the armour layer. Designing for SLS and performing repair when the SLS-damage is reached, imply that the probability of very severe damage or failure is almost neglegeable, and so will be the related cost of repair and downtime

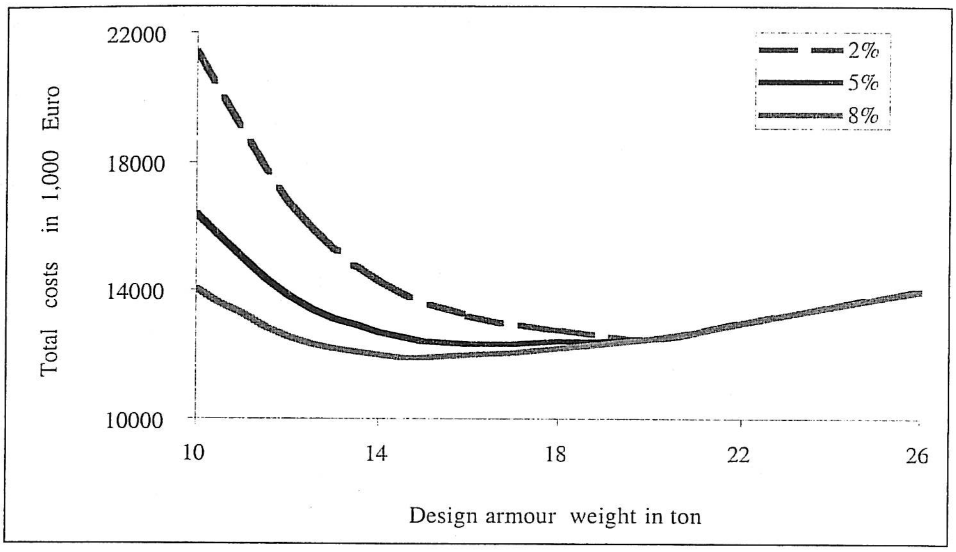


Fig. 8 - *Case 1.1. Total costs in 50 years lifetime. Rock armour. Shallow water of depth 8 m. Damage accumulation included. Downtime costs not included*

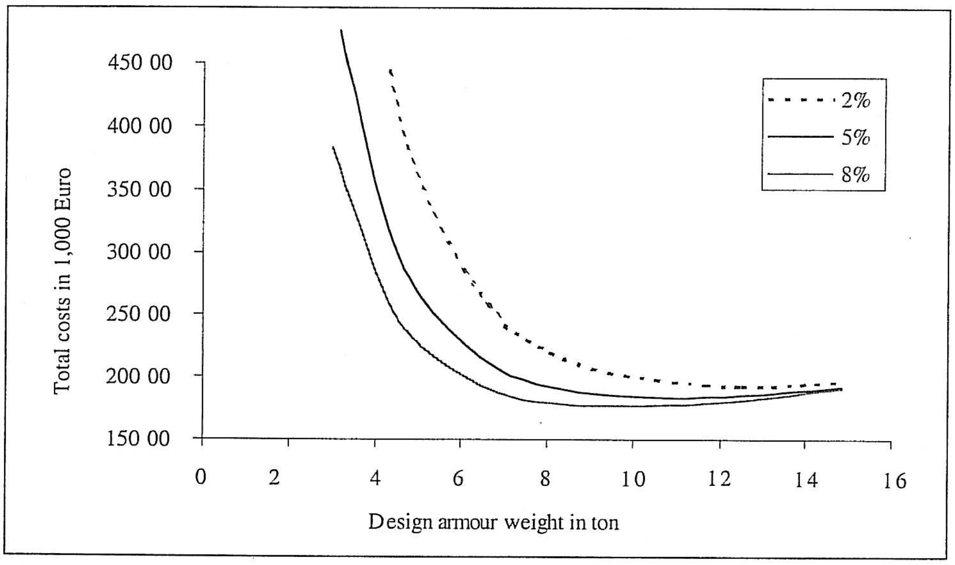


Fig. 9 - *Case 1.3. Total costs in 50 years lifetime. Concrete cube armour. 50 years service lifetime. 15 m water depth. Damage accumulation included. Downtime costs has no influence on total lifetime costs*

costs. This is typical for structures with ductile damage development.

- The identified optimum safety levels corresponds to exceedence of the SLS-moderate damage level in average once to twice within a ser-

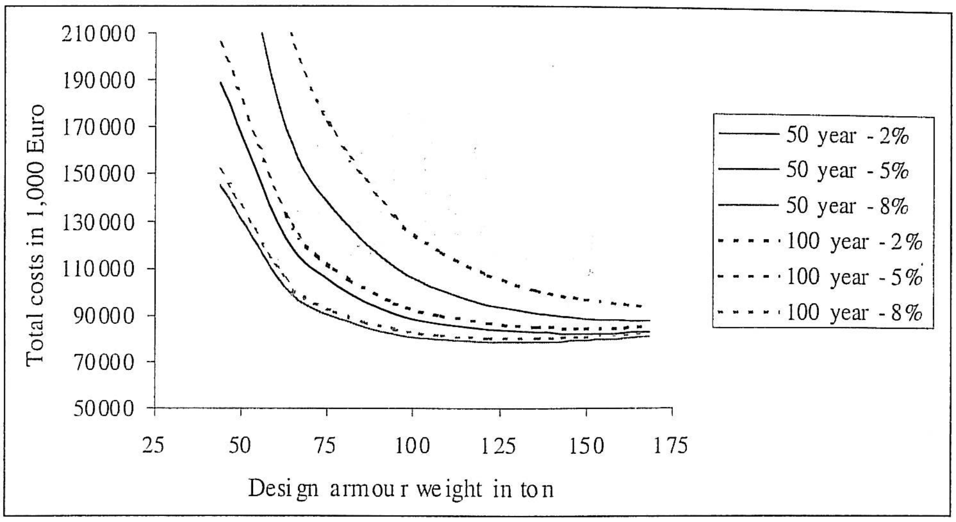


Fig. 10 - *Case 2.3. Total costs in 50 years and 100 years lifetime. Concrete cube armour. 30 m water depth. Damage accumulation included. Downtime costs of 200,000 EURO per day in 3 month for damage D > 15%*

vice life of 50 years, given the yearly interest rates are 2-5%. For higher interest rates the optimum number of exceedences will increase corresponding to less safe structures.

A probabilistic design procedure including all uncertainties must be used when designing for the identified optimum safety levels as these safety levels are related to such design procedure.

- The relations between total lifetime costs and the safety levels (e.g. in terms of armour unit mass) show very flat minima. This means that conservative designs involving fewer repairs are only slightly more expensive than cost optimised designs.
- Knowledge about damage accumulation is important for the assessment of optimum safety levels. Verification of the influence of choice of damage accumulation model is ongoing.
- The obtained results indicate that optimum safety levels for rubble mound breakwaters belonging to the functional classes III and IV (cf. Fig. 3) will be almost the same as for classes I and II. This is because of the marginal influence of downtime costs.
- It is expected that optimum safety levels for caisson breakwaters will be relatively higher than for rubble mound structures due to the more brittle failure of caissons. This is under investigation.
- The present and the coming investigations hopefully form a usefull basis for national code makers to give advice on design safety levels for breakwaters, related to both deterministic and reliability based design procedures.

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