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Department of Civil Engineering, Aalborg University, Denmark
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1 Introduction

No international standards and recommendations provide target safety levels for breakwaters. The EN 1990:2002 and JCSS 2000 provide safety levels but only for buildings and bridges for which probability of human injury is much larger than for breakwaters.

Specifically related to breakwaters the Spanish ROM and the Italian Guidelines are examples of national recommendations providing target design safety levels. No distinction in safety levels for the various types of breakwaters is made in these guidelines.

A comparison of the target safety levels given in the above mentioned publication is presented in Table 1.1. Regarding EN 1990:2002 and JCSS 2000 the reliability and consequence classes most relevant for breakwaters are chosen.

SLS stands for Serviceability Limit State and ULS for Ultimate Limit State. These design limit states are also demanded in the later standard ISO 21650 (2007), Actions from Waves and Currents on Coastal Structures.

Table 1.1. Comparison of limit state tentative target structure failure probabilities corresponding to 50 years working life

Norm or Guideline	Reliability class	P_f in 50 years	
		SLS	ULS
EN 1990:2002	RC1 – RC2	0.1	0.0001
JCSS 2000	Class 1. High to Low rel. cost of safety measure	0.5 - 5.0	0.0005 – 0.05
Italian Guidelines	Limited risk of human life	0.25 – 0.50	0.10 – 0.20
ROM 0.0 (2002)	SERI < 5	0.20	0.20
	0.5≤ SERI<20	0.10	0.10

From Table 1.1 it is seen that only with respect to target failure probabilities related to ULS there is a large deviation between the target failure probabilities given for buildings (EN 1990 and JCSS) and for breakwaters (Italian Guidelines and ROM 0.0). This reflects the different probabilities of human injury in case of structure collapses.

As no international codes and only a couple of national recommendations prescribe safety levels for breakwaters there is need for information on safety levels. A detailed study of safety levels based on lifetime economical optimization has been performed for conventional multi-layer rubble mounds, single layer rubble mounds armoured with interlocking armour units, berm breakwaters and caisson breakwaters on hard and soft seabeds, see Fig. 1.1- 1.5 for typical cross sections.

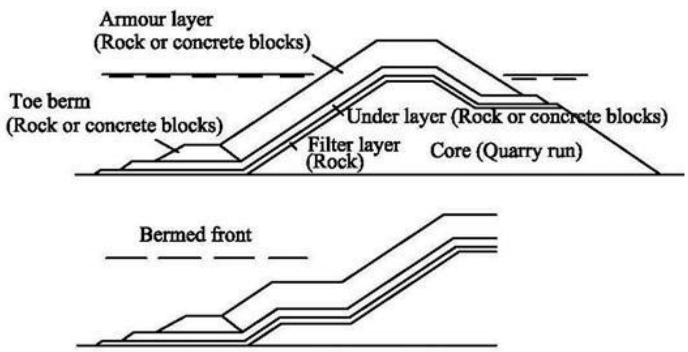


Fig. 1.1. Conventional multi-layer rubble mound breakwater

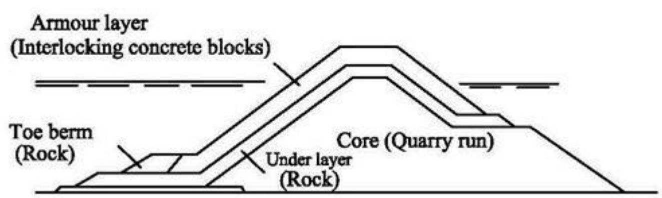


Fig. 1.2. Single layer rubble mound breakwater with interlocking armour units

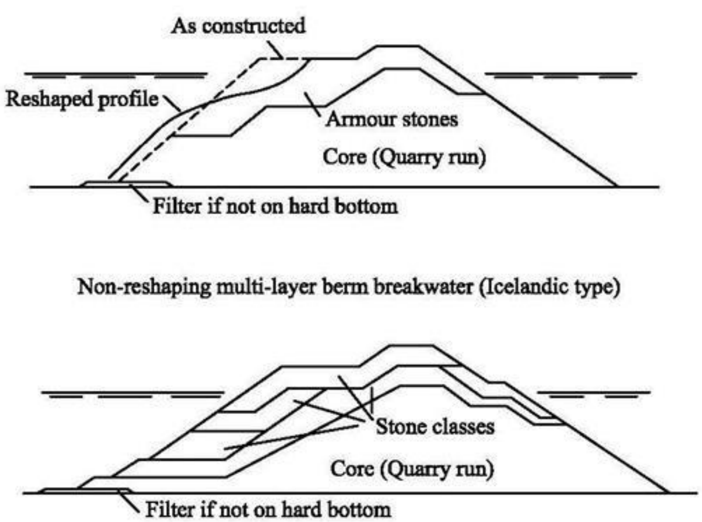


Fig. 1.3. Main types of berm breakwaters

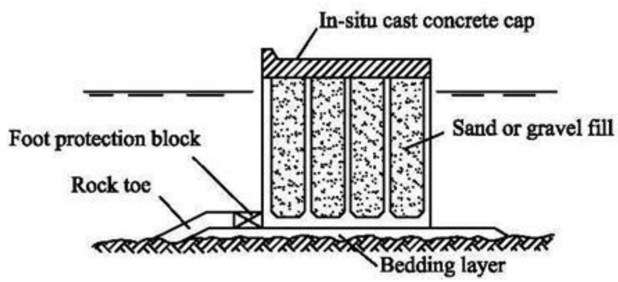


Fig. 1.4. Conventional caisson breakwater on hard seabed

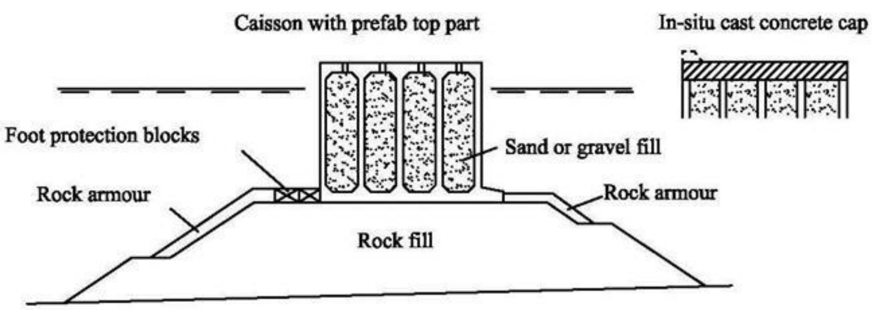


Fig. 1.5. Caisson breakwater with high rock foundation

In the analyses a very large number of breakwater designed by conventional deterministic methods are exposed to lifetime wave climates. The performance in terms of damages and related repairs are identified, and by adding initial construction costs and cost of repairs lifetime costs are obtained. The safety levels of the structures showing the lowest lifetime costs are subsequently analyzed and presented as optimum safety levels. These are given for the design limit states (SLS) and (ULS). Additionally optimum safety levels are given for Repairable Limit State (RLS) being a state for which repairs can be accomplished with foreseen methods and equipment. Downtime cost due to stop of port operations in case of major breakwater damage is considered. Human injuries related to breakwater damages are very seldom and are therefore disregarded in the analyses.

2. Life-cycle analysis and cost optimization

This chapter provides a general background for the parametric study of breakwater reliability based on life-cycle cost optimization presented in Chapters 3 - 5.

As the risk of human injury is marginal it is common to disregard such risk when designing breakwaters. Therefore, design of new breakwaters and rehabilitation of existing breakwaters can be based on life-cycle analysis targeting the minimum lifetime costs i.e. the costs of construction, maintenance, repairs and demolition, depositing and reuse of materials. The last three items are very often omitted due to difficulties in prediction of realizations. The principle of identifying the safety level corresponding to the minimum lifetime costs is illustrated in Fig. 2.1.

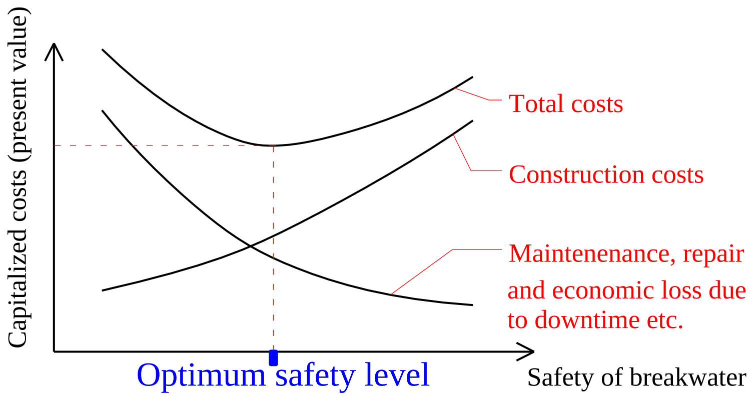


Fig. 2.1. Illustration of principle in determination of safety level corresponding to minimum lifetime costs (Burcharth et al., 2006)

The study covers breakwaters with no berths on the rear side, i.e. cases for which some overtopping and related wave transmission can be allowed. If only very limited overtopping is allowed the structures must generally have higher crest levels, but the optimum safety level will hardly be changed compared to the studied structures.

Only the main failure modes are taken into account. Inclusion of more but less important failure modes will not change the optimum safety levels related to the main failure mechanisms. Moreover, the extra construction costs of strengthening secondary structure elements (e.g. a toe berm in a rubble mound breakwater) to a degree of negligible failure probability are very small. This explains why correlation (interaction) between main failure modes and other failure modes is not included in the simulations.

The applied procedure in solving the optimization problem illustrated in Fig. 2.1 follows the overall procedure listed below. More specifically for this parametric study the optimization problem was solved by a numerical procedure using Monte Carlo simulation in which a very large number of structures are exposed to realistic life time wave histories. The structure geometries were determined by conventional deterministic design for a selected range of water depths (10 – 40 m) and long-term wave statistics applying design waves corresponding to different return periods. Damages as they occur were identified and accumulated, and repairs performed in accordance with defined repair policy. The related costs of repairs were calculated as they appeared in time. Failures (large damages), which introduce downtime costs due to stop of port operations were identified and the related downtime costs calculated. Further, the construction cost of each breakwater was calculated. All costs were added to obtain the total lifetime cost. Among each type of structure and environmental conditions was identified the structure with the lowest life time costs, and for this structure was extracted the related probabilities of reaching SLS, RLS and ULS in the structure working life. These values then represent the optimum design safety levels. The simulations comprised the influence on the optimum safety level of interest rate (2, 5, and 8% p.a., inflation included), structure working life (50 and 100 years) and downtime costs.

In summary the steps in the performed simulations are as follows:

1. Select type of breakwater, water depth and long-term wave statistics.

2. Extract design values of significant wave height $\left(H_S^T\right)$ and wave steepness corresponding to a number of return periods, T = 5, 10, 25, 50, 100, 200 and 400 years.
3. Select working lifetime for the structure, e.g. T_L = 50 and 100 years.
4. Design by conventional deterministic methods the structure geometries corresponding to the chosen H_S^T – values.
5. For each structure geometry calculate the construction costs.
6. Define repair policy and related cost of repair.
7. Specify downtime costs related to damage levels.
8. Define a model for damage accumulation.
9. For each structure geometry use stochastic models for wave climate and structure response (damage) in Monte Carlo simulation of occurrence of damage within structure working life. The structures are exposed to storms corresponding to real long-term statistics occurring in accordance with a Poisson process.
10. For each simulation related to a specific structure geometry, calculate the total capitalized working life costs. Subsequently calculate the mean value and the related safety levels corresponding to the design limit states.
11. Identify the structure safety level corresponding to the minimum total costs.

The formulation of the cost function (used in Step 10) for total costs over the design working life is based on the following assumptions:

- The breakwater is designed corresponding to a design wave height with return period T
- The initial costs, $C_I(T)$, costs of repair for minor damage, $C_{R_1}(T)$, costs of repair for major damage, $C_{R_2}(T)$, and cost of failure, $C_F(T)$, all depend on the design wave height with return period T
- Storms are assumed to be modeled by a Poisson process with occurrence rate λ, i.e. the average number of storms per year
- All costs are discounted back to the time when the breakwater is built

The optimal design is determined from the following optimization problem where the total capitalized costs during the design lifetime T_L are minimized:

$$\min_T \quad C(T) = C_I(T) + \sum_{t=1}^{T_L} \left\{ C_{R_1}(T) P_{R_1}(t) + C_{R_2}(T) P_{R_2}(t) + C_F(T) P_F(t) \right\} \frac{1}{(1+r)^t} \qquad (2.1)$$

where

T return period used for deterministic design

T_L design life time

$C_I(T)$ initial costs (building costs)

$C_{R_1}(T)$ cost of repair for minor damage when SLS is exceeded

$P_{R_1}(t)$ probability of minor damage in year t

$C_{R_2}(T)$ cost of repair for major damage when RLS is exceeded

$P_{R_2}(t)$ probability of major damage in year t

$C_F(T)$ cost of failure including downtime costs when ULS is exceeded

$P_F(t)$ probability of failure in year t

r real rate of interest

No benefits and no costs related to loss of life are included.

Life cycle considerations related to decommissioning, depositing and reuse of construction material have not been included in the analyses of optimum safety levels.

Estimates of construction and repair costs are based built-in volume unit prices for a range of prototype structures, collected by the PIANC MarCom Working Group 47 members. The unit prices correspond to years 2004 - 2007. No update to actual prices has been made because only the ratios between unit prices for structure components determine the minimum working life costs.

All costs are related to 1 km of breakwater. This includes construction costs, total design working life costs and downtime costs. The downtime costs are in all studied cases set to 200,000 EURO per day in three months, i.e. a total of 18,000,000 EURO. This is a relatively large amount when related to the costs of just 1 km of breakwater, but is chosen in the first hand only to see the effect on optimum design safety levels.

The applied long-term wave statistics are based on fitting of 3-parameter Weibull distributions to field data from Follonica (Adriatic Sea), Bilbao (Bay of Biscay), Baltic Sea, and Sines (Atlantic Ocean). Storms are assumed to be modelled by a Poisson process with occurrence rates corresponding to the average number of storms per year.

Characteristics of these wave climates are indicated in Table 2.1 which provides the deep water significant wave heights corresponding to various return periods. More details are given in PIANC (1992).

Table 2.1. Characteristic of wave statistics applied in cost optimization simulations.

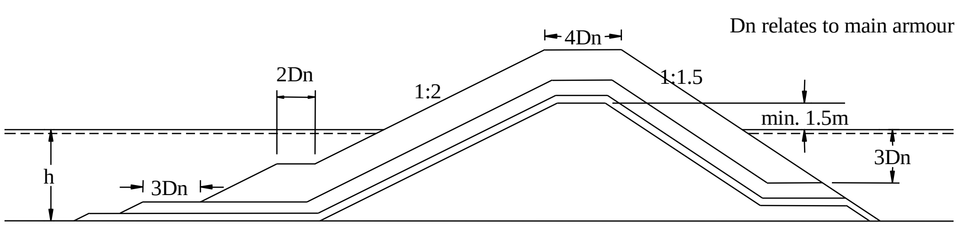
Location	Return period significant wave height H_s (m) related to return periods (years)						
	5	25	50	100	200	400	1000
Follonica	4.35	5.07	5.36	5.64	5.92	6.20	6.56
Bilbao	-	8.09	8.43	8.76	9.08	9.38	9.77
Sines	-	12.16	12.71	13.23	13.71	14.16	14.73
Baltic Sea	3.55	4.71	5.36	6.08	6.88	7.75	9.00

More details about the wave statistics are given in PIANC (1992b). The applied wave steepness is in the range 0.02-0.04.

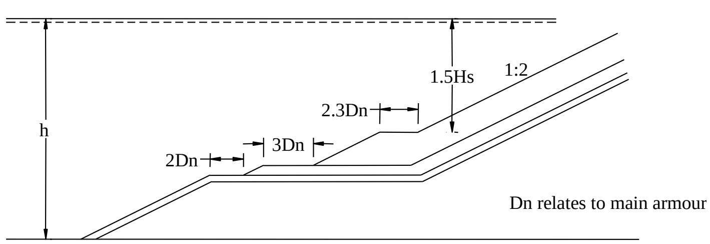
3. Optimum safety levels of conventional rock and cube armoured rubble mound breakwaters

3.1 Cross sections and failure modes

Conventional two-layer armour structures without superstructure as shown in Fig. 3.1 are studied.



Shallow water cross section: h < 1.5 H_S + 2.7$\cdot$ D_n



Deep water cross section: h ≥ 1.5 H_S + 2.7 D_n

Fig. 3.1. Shallow and deep water cross sections.

The crest level is in the deterministic design for both shallow water and deep water conditions determined on the basis of maximum transmitted significant wave height $H_{s,t} = 0.50$ m by overtopping for incoming significant wave height with return period T_L. Moreover, structure damage is assumed solely related to displacement of main armour units, as economic implications of using a conservative design of for example the toe are negligible.

Geotechnical aspects are not considered in the present optimization.

3.2 Limit state performance, repair strategy, costs and case study data

Repair is related to main armour damage given by the relative number of displaced units, *D*, as s

hown in Table 3.1. The damage parameter $S = A_e/D_{n50}^2$, where A_e is the cross sectional eroded area, and D_{n50} = (mean armour unit volume)$^{1/3}$. N_{od} is the number of displaced units within a strip with width D_n.

Table 3.1. Applied repair policy as function of damage levels

Limit state damage levels	***S* (rock)**	**N_{od}(cubes)**	**Estimated *D***	**Repair policy**
Initial	2	0	2 %	no repair
SLS Serviceability (minor damage, only to armour)	5	0.8	5 %	repair armour
RLS Repairable (major damage, armour + filter 1)	8	2.0	15 %	repair armour + filter 1
ULS Ultimate (failure)	13	3.0	30 %	repair armour + filter 1 and 2

The main data including built-in unit prices for the cases are given in Table 3.2. The 100 and 400 years return period expectation values of the deep-water significant wave height H_s are also given in Table 3.2 in order to indicate the tails of the distributions. The applied deep water mean period wave steepness is 0.030 for the rock armour and 0.025 for the cube armour.

The built-in unit prices are based on typical unit prices around year 2002 - 2007 collected from European projects. The rock material unit prices correspond to easy access to nearby quarry.

It is important to notice that it is the ratios between the unit prices of the various structure parts which influence the economical optimum safety level, rather than the actual costs. It is therefore more important that these ratios between the built–in prices are realistic than the correctness of the actual cost level which actually changes with time.

Table 3.2. Case study data

Case	Water depth	Armour density	Waves Origin $H_{S,o}^{100y}$	Distribution $H_{S,o}^{400y}$	Stability formula	Built-in unit prices core/filter 2/ filter 1/armour EURO/m^3
1	10 m	Rock 2.65 t/m^3	Follonica 5.64 m	Weibull 6.20 m	Van der Meer (1988)	10/16/20/40
2	15 m	Concrete cube 2.40 t/m^3	Follonica 5.64 m	Weibull 6.20 m	Van der Meer (1988) modified to slope 1:2	10/16/20/40
3	30 m	Concrete cube 2.40 t/m^3	Sines 13.2 m	Weibull 14.2 m	Van der Meer (1988) modified to slope 1:2	5/10/25/35

3.3 Overview of case studies. Identified optimum safety levels

The case studies are explained in Table 3.2. The identified optimum safety levels and related deterministic design conditions are given in the following tables. All details on assumptions and applied formulae are given in Appendix A1. The data sheets from which the tables presented in this chapter are extracted are given in Appendix A2.

Table 3.3. Case 1. Optimum safety levels for rock armored breakwater. 50 years' service lifetime. 10 m water depth. Damage accumulation included. Downtime: 200,000 EUR/day in 3 month.

Real interest rate (%)	Downtime costs	Deterministic design data			Optimum armor unit mass, W (t)	Optimum limit state average number of events within service lifetime			Construc. costs EUR/m	Life time costs EUR/m
		Optimum design return period, T yrs	H_s^T (m)	Armour unit mass, W (t)		SLS	RLS	ULS		
2	None	200	5.50	13.39	16.70	1.36	0.060	0.010	13500	14564
5		50	5.36	12.36	12.36	4.02	0.286	0.062	11920	13773
8		50	5.36	12.36	12.36	4.02	0.286	0.062	11920	13146
2	Included	400	5.50	13.39	19.15	0.72	0.023	0.002	14233	15084
5		200	5.50	13.39	16.70	1.36	0.060	0.001	13500	14565
8		200	5.50	13.39	16.70	1.36	0.060	0.001	13500	14204

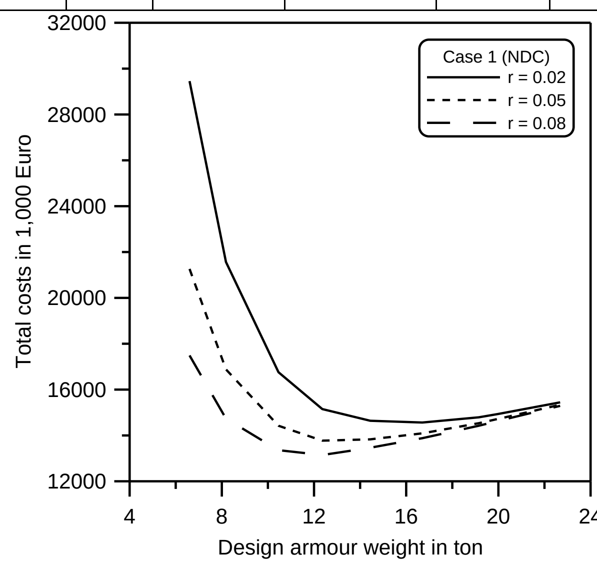


Fig.3.2. Case 1. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation included. No downtime costs included.

For depth limited wave conditions, as in Case 1, the frequency of the highest waves is large. Thus the optimum design corresponds to relatively big armour unit sizes due to the effect of damage accumulation. As seen from Table 3.3 and Fig. 3.2 deterministic design based on maximum wave height H_s = 5.50 m (depth limited) gives armour unit mass of W_{50} = 13.39 t while the numerical simulations show it is more economical to use heavier units. (Note that such heavy rocks are available only in few countries).

Table 3.4. Case 2. Optimum safety levels for cubes armored breakwater. 50 years' service lifetime. 15 m water depth. Damage accumulation included. Downtime: 200,000 EUR/day in 3 month.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Construc costs EUR/m	Life time costs EUR/m
		Optimum design return period, T yrs	H_s^T (m)	Armour unit mass, W (t)	SLS	RLS	ULS		
2	None	100	5.64	9.45	3.35	0.06	0.02	16038	18029
5		50	5.36	8.09	5.31	0.11	0.04	15316	17094
8		50	5.36	8.09	5.31	0.11	0.04	15316	16495
2	Included	200	5.92	10.93	2.13	0.03	0.01	16763	18498
5		100	5.64	9.45	3.35	0.06	0.02	16038	17694
8		100	5.64	9.45	3.35	0.06	0.02	16038	17140

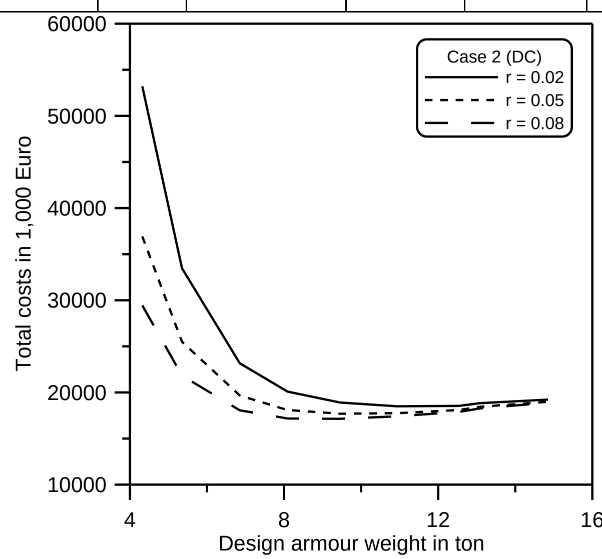


Fig. 11.3. Case 2. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation and downtime costs included.

The influence of service lifetime is illustrated in Fig. 3.4 in which 50 years and 100 years total costs are shown for Case 2.

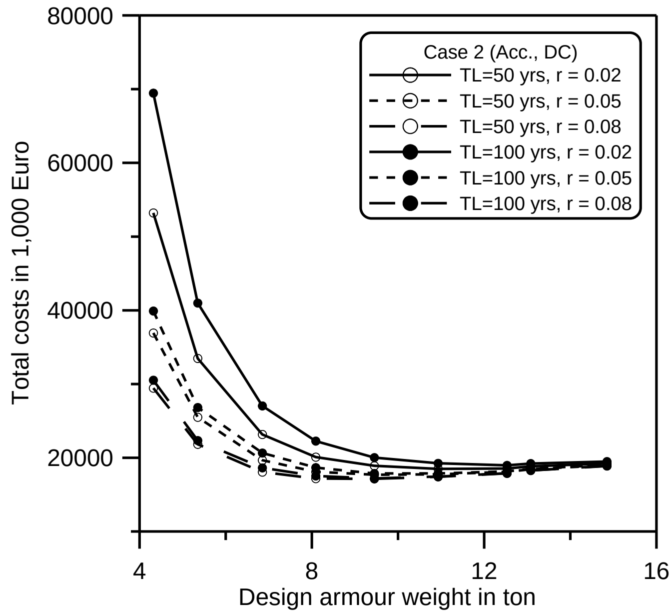


Fig.3.4. Case 2. Total costs in 50 years and 100 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation and downtime costs included.

The influence of damage accumulation on total costs in 50 years lifetime for Case 2 is illustrated in Fig. 3.5.

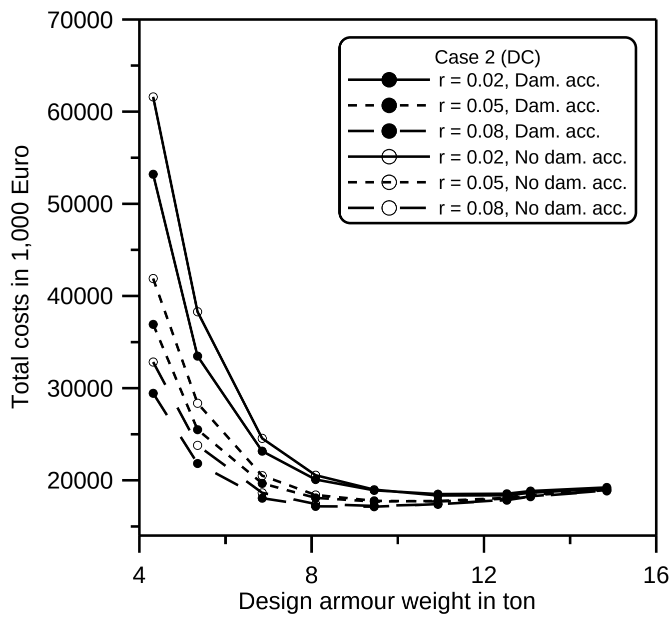


Fig.3.5. Case 2. Influence of damage accumulation on total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Downtime costs included.

Table 3.5. Case 3. Optimum safety levels for cube armored breakwater. 50 years'service lifetime. 30 m water depth. Damage accumulation included. Downtime: 200,000 EUR/day in 3 month.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Constr. costs EUR/m	Life time costs EUR/m
		Optimum design return period, T yrs	H_s^T (m)	Armour unit mass, W (t)	SLS	RLS	ULS		
2	None	200	13.71	135.67	2.74	0.052	0.016	71224	80179
5		100	13.23	121.91	3.72	0.092	0.029	68635	75672
8		50	12.71	108.3	5.02	0.160	0.056	65932	72344
2	Included	200	13.71	135.67	2.74	0.052	0.016	71224	80954
5		100	13.23	121.91	3.72	0.092	0.029	68635	76497
8		50	12.71	108.3	5.02	0.160	0.056	65932	73302

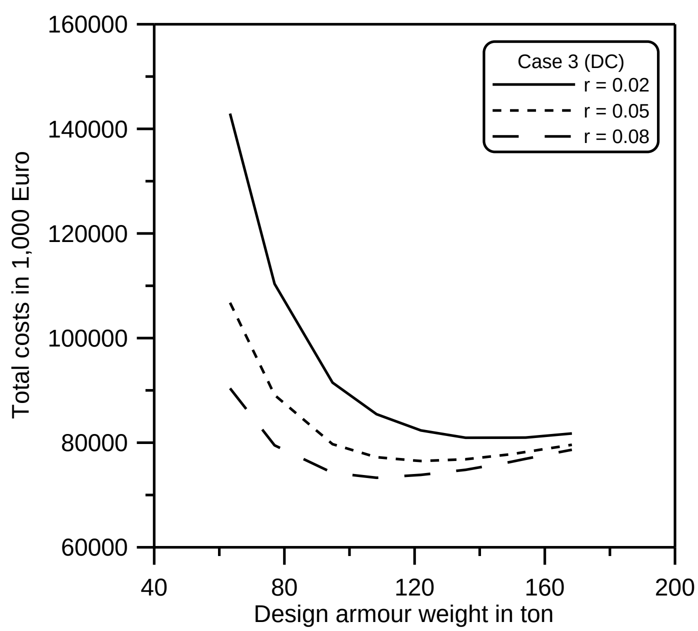


Fig. 3.6. Case 3. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation and downtime costs included.

3.4 Conclusions

3.4.1. Optimum safety levels

From Tables 3.3 – 3.5 it can be deducted that for outer breakwaters armoured with rocks or concrete cubes the optimum reliability levels are roughly for a service lifetime of 50 years one to three times exceedance of the defined SLS, and 2 -6% probability of exceeding the defined RLS, and 1 – 2% probability of exceeding the defined ULS. These values are for 2% p.a. interest rate. For 5% p.a. interest rate the values are one to five times exceedance of the SLS, and 1 – 10 % probability of exceeding RLS, and 0.1 – 6% probability of exceeding the ULS.
The corresponding *annual* optimum reliability levels are determined by dividing the 50 years values by 50. The ranges of optimum annual reliability levels are given in Table 3.7

Table 3.7 Approximate ranges of optimum *annual* reliability levels for rock and cube armoured outer breakwaters with and without downtime costs.

Limit state	2% p.a. interest rate	5% p.a. interest rate
SLS	0.02 – 0.06	0.02 – 0.10
RLS	0.0005 – 0.001	0.001 – 0.002
ULS	0.0002 – 0.0005	0.0004 – 0.001

Higher interest rates reduce the optimum safety level.

Figs. 3.5 – 3.7 show very flat minima of total costs as function of armour unit mass. Thus it is less important to identify the exact optimum failure probability because the lifetime costs are practically independent of the design safety level within a fairly wide range. This is because the larger capital costs of a safer structure are almost balanced by smaller repair costs. As a consequence it is generally preferable to choose a conservative design in order to reduce the political and financial inconveniences related to repairs.

The optimum safety levels correspond to deterministic design applying wave return periods of 200 – 400 years for interest rate 2% p.a., and return periods of 50-200 years for interest rate 5% p.a. The largest return periods correspond to design in which downtime for port operation is included. The choice of return periods within the given intervals is not critical because of the flat minimum for the total costs.

3.4.2 Influence of real interest rate on optimum safety level

Tables 3.3 – 3.5 and Figs. 3.4 – 3.6 show for optimum designs that the lifetime costs and the optimum safety levels decrease rather significantly with increasing interest rate. Thus it is more economical to design for more frequent repairs in case of high interest rates. This however might be practically and politically unacceptable.

3.4.3 Influence of damage accumulation on optimum safety level

The three cases are based on damage accumulation. If no damage accumulation is assumed then optimum design failure probability within lifetime is reduced as illustrated in Fig. 3.5. This underlines the importance of choosing a correct model for damage accumulation. Damage accumulation should in any case be taken into account.

3.4.4 Influence of downtime costs on optimum safety levels

Tables 3.2 and 3.3 show that even fairly large downtime costs of 200,000 EURO/day in 3 months, i.e. 18,000,000 EURO in case of more than 15% damage to the armour layer, has a marginal influence on the optimum safety level. This indicates that for conventionally designed rubble mound breakwaters downtime costs, unless relatively very high, has little influence on optimum design safety levels. The explanation for this is that for conventional rubble mound breakwaters the probability of a major failure leading to downtime costs is very small for cost optimised designs, as SLS is the critical design limit state for this type of structures.

3.4.5 Influence of service life on optimum safety level

The ratio of optimum design failure probability to service lifetime is almost constant for each of the design limit states. This means that if for SLS the optimum number of exceedances of the SLS-damage level is one within a service life of 50 years, then it will be roughly two within a service life of 100 years.

3.5 Partial safety factors corresponding to optimum safety levels

Partial safety factors for rubble mound breakwaters were developed in PIANC (1992). A complete overview is given in Burcharth and Sorensen (2000). The present explanation and computations can be regarded as a check on the PIANC safety factors. Table 3.11 presents a partial comparison of the two sets of partial coefficients.

For the determination of the partial safety factors for rock and cube armour, the results of cost optimization shown in Tables 3.8 to 3.10 were used. The data used satisfy the following condition

$$C_T / C_O \leq 1.05 \tag{3.1}$$

where C_T is the total cost and C_O is the optimal total cost for each cases.

Overall safety factors for rock and cube are calculated as

$$\gamma_S \, \gamma_R \frac{H_s}{\Delta D_n} = 6.2 \, S^{0.2} \, P^{0.18} \, N_z^{-0.1} \, s_{om}^{0.25} \tan \alpha^{-0.5} \tag{3.2}$$

$$\gamma_S \, \gamma_R \frac{H_s}{\Delta D_n} = \left(\frac{2}{1.5} \right)^{1/3} \left(6.7 \frac{N_{od}^{0.4}}{N_z^{0.3}} + 1 \right) s_{om}^{-0.1} \tag{3.3}$$

where in the rock formula, $\Delta = 1.57$, H_s is the significant wave height of 50 years return period, $P = 0.04$, $S = 5\,\text{(SLS)},\ 13\,\text{(ULS)}$, $N_z = 1000$, $s_{om} = 0.025$, $\tan\alpha = 0.5$. In the cubes formula, $\Delta = 1.33$. The number of waves and wave steepness are the same in the case of rock formula.

The partial safety factors for each limit state are evaluated with the probability of failure using Eq. (3.4) and estimates from Fig. 3.7.

$$\gamma_S\,\gamma_R = 1.0862\,P_f^{\,-0.0658} \tag{3.4}$$

The coefficient of correlation R is 0.90. γ_S and γ_R are the load and resistance safety factors, respectively.

In Eq. (3.4) the probability of failure was estimated using the following considerations:

There is a conceptual difference between average number of event and probability of failure within service lifetime. In the Monte Carlo simulations the annual probability of failure was calculated as as

$$P_f^{\,1year} = N_f \,/\, N \tag{3.5}$$

where N_f is the number of exceedance for some criterion and N is the total number of simulation. The annual probability of failure can by using the average number of event within the service lifetime be estimated as

$$P_f^{\,1year} = N_E \,/\, 50 \tag{3.6}$$

where N_E is the average number of event which exceeds each limit state damage criterion. Finally, assuming that every failure event occur during the service lifetime, the probability of failure within service lifetime 50 years is calculated as

$$P_f^{\,50years} = 1 - (1 - P_f^{\,1year})^{50} \tag{3.7}$$

When the average number of event within service lifetime N_E is less than 0.1, the probability of failure in service lifetime $P_f^{\,50}$ can be almost equal to N_E.

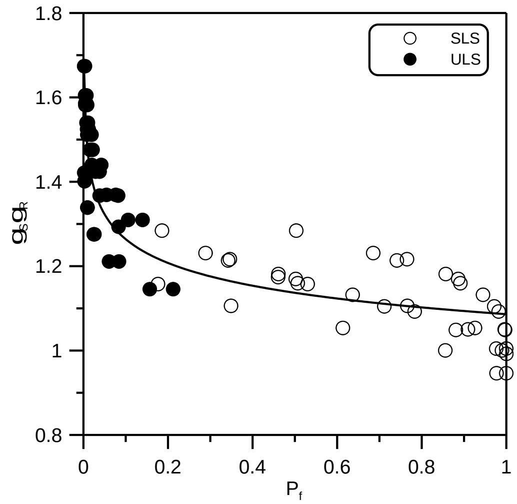


Fig. 3.7. The relationship between the partial safety factors and failure probability within 50 years' lifetime

Table 3.8. Case 1(Rock), Partial safety factor corresponding to the failure probability on each limit state. No downtime cost included.

Damage accumulation	Deterministic design data	Average number of event exceeding each damage level ($=N_E$)						C_T/C_O ≤ 1.05
	Armor unit mass, W (t)	SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)			
		N_E	P_f^{50}	$\gamma_S\,\gamma_R$	N_E	P_f^{50}	$\gamma_S\,\gamma_R$	
With	6.60	25.6399	1.0000	0.8117	1.3678	0.7501	0.9826	-
	8.18	14.9505	1.0000	0.8719	0.5633	0.4325	1.0555	-
	10.46	7.4390	0.9997	0.9464	0.1702	0.1567	1.1456	O
	12.36	4.3675	0.9896	1.0005	0.0623	0.0604	1.2112	O
	14.44	2.5347	0.9258	1.0537	0.0246	0.0243	1.2756	O
	16.70	1.4302	0.7657	1.1061	0.0096	0.0096	1.3390	O
	19.15	0.7496	0.5301	1.1577	0.0024	0.0024	1.4015	O
	19.98	0.6130	0.4603	1.1742	0.0017	0.0017	1.4215	O
	22.68	0.2935	0.2550	1.2249	0.0006	0.0006	1.4828	-
Without	6.60	15.7048	1.0000	0.8117	2.4263	0.9169	0.9826	-
	8.18	8.3943	0.9999	0.8719	0.9437	0.6143	1.0555	-

	10.46	3.6210	0.9767	0.9464	0.2377	0.2120	1.1456	O
	12.36	1.8979	0.8556	1.0005	0.0875	0.0839	1.2112	O
	14.44	0.9413	0.6134	1.0537	0.0262	0.0259	1.2756	O
	16.70	0.4272	0.3489	1.1061	0.0091	0.0091	1.3390	O
	19.15	0.1934	0.1762	1.1577	0.0029	0.0029	1.4015	O
	19.98	0.1406	0.1313	1.1742	0.0017	0.0017	1.4215	-
	22.68	0.0581	0.0565	1.2249	0.0007	0.0007	1.4828	-

Table 3.9. Case 2(Cube), Partial safety factor corresponding to the failure probability on each limit state. Downtime cost included.

Damage accumulation	Deterministic design data	Average number of event exceeding each damage level ($=N_E$)						C_T/C_O ≤ 1.05
	Armor unit mass, W (t)	SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)			
		N_E	P_f^{50}	$\gamma_S \gamma_R$	N_E	P_f^{50}	$\gamma_S \gamma_R$	
With	4.32	28.0658	1.0000	0.8511	0.6952	0.5035	1.1093	-
	5.35	16.6598	1.0000	0.9139	0.2722	0.2389	1.1912	-
	6.85	8.8406	0.9999	0.9924	0.0865	0.0829	1.2935	O
	8.09	5.4571	0.9969	1.0490	0.0389	0.0382	1.3673	O
	9.45	3.4284	0.9713	1.1048	0.0196	0.0194	1.4400	O
	10.93	2.1696	0.8912	1.1597	0.0090	0.0090	1.5116	O
	12.53	1.3329	0.7410	1.2137	0.0043	0.0043	1.5820	O
	13.08	1.1422	0.6851	1.2312	0.0038	0.0038	1.6048	O
	14.85	0.6945	0.5031	1.2844	0.0016	0.0016	1.6741	O
Without	4.32	14.8286	1.0000	0.8511	1.1482	0.6870	1.1093	-
	5.35	8.0649	0.9998	0.9139	0.5226	0.4087	1.1912	-
	6.85	3.6821	0.9782	0.9924	0.1820	0.1667	1.2935	-

	8.09	2.0800	0.8805	1.0490	0.0854	0.0819	1.3673	O
	9.45	1.2275	0.7114	1.1048	0.0429	0.0420	1.4400	O
	10.93	0.7014	0.5066	1.1597	0.0192	0.0190	1.5116	O
	12.53	0.4168	0.3420	1.2137	0.0087	0.0087	1.5820	O
	13.08	0.3393	0.2886	1.2312	0.0069	0.0069	1.6048	O
	14.85	0.2049	0.1856	1.2844	0.0035	0.0035	1.6741	O

Table 3.10. Case 3(Cube), Partial safety factor corresponding to the failure probability on each limit state. Downtime cost included.

Damage accumulation	Deterministic design data	Average number of event exceeding each damage level ($=N_E$)						C_T/C_O ≤ 1.05
	Armor unit mass, W (t)	SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)			
		N_E	P_f^{50}	$\gamma_S\,\gamma_R$	N_E	P_f^{50}	$\gamma_S\,\gamma_R$	
With	63.32	16.5253	1.0000	0.8783	0.7628	0.5364	1.1449	-
	76.99	11.3599	1.0000	0.9375	0.3118	0.2686	1.2219	-
	94.80	7.1749	0.9996	1.0048	0.1115	0.1056	1.3097	O
	108.30	5.2347	0.9960	1.0504	0.0559	0.0544	1.3691	O
	121.91	3.8431	0.9817	1.0927	0.0290	0.0286	1.4242	O
	135.67	2.8123	0.9447	1.1323	0.0156	0.0155	1.4759	O
	149.60	2.1255	0.8861	1.1698	0.0085	0.0085	1.5248	O
	154.13	1.9049	0.8566	1.1815	0.0071	0.0071	1.5400	O
	168.31	1.4276	0.7650	1.2167	0.0042	0.0042	1.5859	O
Without	63.32	10.8785	1.0000	0.8783	0.8903	0.5927	1.1449	-
	76.99	6.6448	0.9992	0.9375	0.3939	0.3266	1.2219	-
	94.80	3.5930	0.9760	1.0048	0.1503	0.1397	1.3097	O
	108.30	2.3416	0.9091	1.0504	0.0793	0.0763	1.3691	O

	121.91	1.5050	0.7831	1.0927	0.0388	0.0381	1.4242	O
	135.67	1.0014	0.6364	1.1323	0.0217	0.0215	1.4759	O
	149.60	0.6921	0.5019	1.1698	0.0124	0.0123	1.5248	O
	154.13	0.6140	0.4609	1.1815	0.0106	0.0105	1.5400	O
	168.31	0.4232	0.3462	1.2167	0.0067	0.0067	1.5859	O

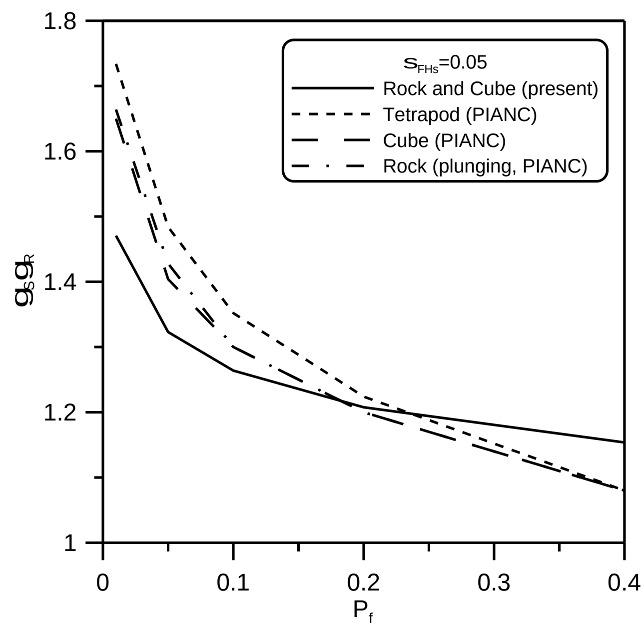


Fig. 3.8. Comparison of partial safety factors between different methods for rock and Cube armor units

Table 3.11. The present and the PIANC (1992) partial safety factors for rock and cube armor units

P_f	Present	PIANC ($\sigma_{FH_s}=0.05$)	
	Rock and Cube	Rock	Cube
0.01	1.47	1.66	1.65
0.05	1.32	1.43	1.40
0.10	1.26	1.30	1.30
0.20	1.21	1.20	1.20
0.40	1.15	1.08	1.08

4. Optimum safety levels of berm breakwaters

4.1 Cross sections and failure modes

Berm breakwaters can be designed as reshaping or non-reshaping as illustrated in Fig.4.1.

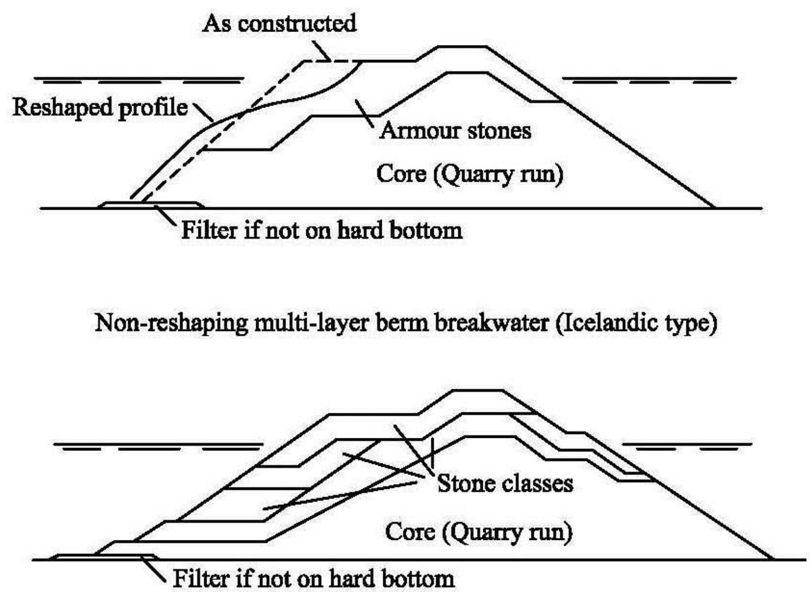


Fig. 4.1. Main types of rubble mound berm breakwaters.

The berm of the reshaping type is initially unstable but will reshape during normal and more severe wave conditions into more stable gentle s-curved slopes which change/adjust to the various sea states. Oblique waves over a certain threshold cause transport of stones along the structure which can cause problems in terms of lack of stones in some sections, Van der Meer and Veldman (1992) and Tomasicchio et al. (2013). The structures are designed for a maximum reshaping/recession of the berm in the design storm.

The non-reshaping type is designed for practically no erosion of the berm under more severe wave actions. Only for design storm conditions is some limited recession of the berm allowed. Before recession of the berm takes place, erosion of the front slope might take place if the berm level is more than approximately half a significant wave height over SWL, see Sigurdarson and Van der Meer (2011) and Burcharth (2013).

The two failure modes recession R_{ec} and front slope erosion area A_e are illustrated in Fig. 4.2.

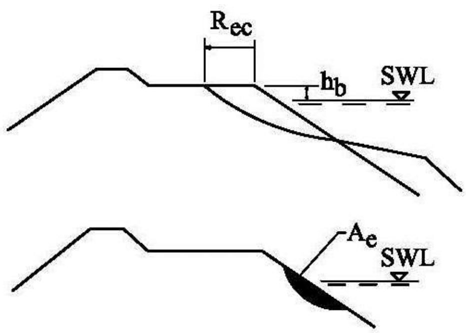


Fig. 4.2. Definition of the failure modes recession and front erosion

The main stability parameters are $N_s = H_o = H_s/(\Delta D_{n50})$ and $H_o\ T_{om} = H_o\ T_m(g/D_{n50})^{0.5}$.

Berm breakwaters are, according to PIANC MarCom Report of WG 40 (2003), classified as shown in Table 4.1. The N_s – values correspond to design wave conditions.

Table 4.1. Classification of berm breakwaters according to PIANC MarCom Report of WG 40 (2003)

Type	$N_s = H_o = H_s/(\Delta D_{n50})$	H_oT_{om}
Statically stable, no reshaping of berm	< 1.5 - 2	< 20 - 40
Statically stable, some reshaping of berm in design sea states	1.5 – 2.7	40 - 70
Dynamically stable, larger reshaping, movements of stones	> 2.7	> 70

Another classification as shown in Table 4.2 is introduced by Sigurdarson and Van der Meer (2013). The influence of wave period is omitted as the authors found the influence insignificant for the studied geometries of the breakwaters.

Table 4.2. Classification of berm breakwaters based on 100 years return period wave conditions (Sigurdarson and Van der Meer, 2013)

Berm breakwater type	$N_s = H_o = H_s/(\Delta D_{n50})$	R_{ec}/D_{n50}
Hardly reshaping Icelandic-type	1.7 – 2.0	0.5 – 2
Partly reshaping Icelandic-type	2.0 – 2.5	1 – 5
Partly reshaping mass armoured type	2.0 – 2.5	1 – 5
Reshaping mass armoured type	2.5 – 3.0	3 - 10

The cost optimization procedure applied for the berm breakwaters is the same as applied for the conventional rubble breakwaters, cf. Chapter 2..

Berm breakwater cross sections vary a lot with respect to number of stone classes. The Årviksand berm breakwater in Norway shown in Fig. 4.3 is an example of the simple cross section of a mass armoured berm breakwater. The Sirevåg breakwater in Norway shown in Fig. 4.4 is an example of an Icelandic-type multi-layer berm breakwater. In this case are applied six classes of stones. This involves a lot of sorting of the stones and a more complicated construction procedure. The advantage is optimum use of the available rock material with respect to resistance against wave impact.

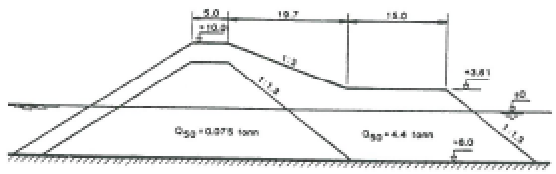


Fig.4.3. Cross section of the Årvikssand berm breakwater in Norway

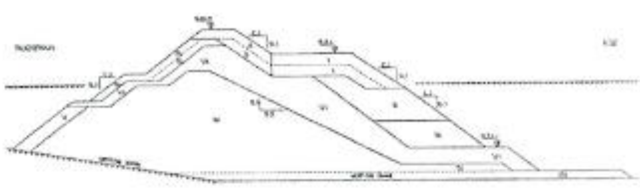


Fig.4.4. Cross section of the Sirevåg berm breakwater in Norway

The cross section applied in the present optimization analyses is based on experience from Island where berm breakwaters have been built for many years and a preferred multi-layer cross section has been developed. Fig. 4.5 shows the parameterized Icelandic cross section which is applied in the analyses. The recession R_{ec} of the berm shoulder shown in Fig. 4.2 is the only damage parameter used in the analyses. Three classes of stones are considered although more classes are used in some berm breakwaters. This however has no importance because the damage calculated in the present analyses is related only to the recession of the berm and therefore only affecting the Class 1 berm stones. This on the other hand necessitates that the berm of Class 1 stone must be so deep that the eroded surface does not extend to the under-laying Class 2 stones.

The nominal diameter D_{n50} of the three stone classes is for simplicity denoted D1, D2 and D3.

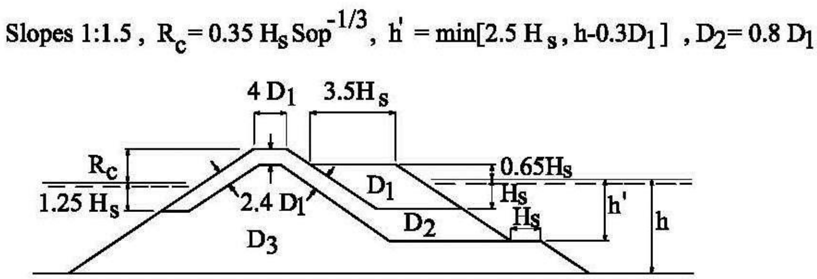


Fig.4.5. Parameterized cross section of the berm breakwater

4.2 Limit state performance, repair strategy and costs

Table 4.3 explains the applied limit state damage definitions and the related repair strategy.

Table 4.3. Limit state performance and related repair strategy

Limit state	Damage definition	Repair strategy
SLS	Recession reaches half of the berm width	Eroded volume replaced
RLS	Some erosion of crest and rear side	Eroded volume replaced plus extra volume
ULS	Recession exceeds the width of the berm	Eroded volume replaced

The built-in unit prices for the quarry rock stones are based on bids for the construction of the Sirevåg berm breakwater in Norway, regulated to the 2007 cost level, Sigurdarson et al. (2007). In the optimization analyses is it the relative costs between the stone classes which are important, not the actual costs which vary from year to year. Table 4.4 gives the built-in unit prices for the various sizes of stones in EURO per m^3 bulk volume, i.e. stones plus voids. The applied bulk volume built-in unit price for the core material is 10.1 EUR/m^3.

Table 4.4. Bulk volume built-in unit prices for stones

Mean mass (t)	Unit price (EUR/m^3)
0.1	10.1
0.6	14.7
2	15.0
6	18.9
13.3	23.5
23.3	27.0

For RLS repairs the unit prices are increased by 50%. For ULS repair the unit price is increased by 150%.

Detailed information on design limit states, repairs, costs and formulae for prediction of recession are given in Appendix B1.

4.3 Overview of case studies and identified optimum safety levels

Cost optimization analyses are made for structures in 11 m and 20 m water depths. Table 4.5 gives an overview of the case study simulations. In each case study are identified the service lifetime costs of the berm breakwaters cross sections designed deterministically for H_s values corresponding to return periods T = 5, 25, 50, 100, 200, and 400 years, and $H_o = N_s$- values of 1.8, 2.0, 2.4, 2.8 and 3.2.

The deep water wave steepness is set to $s_{op} = 0.035$, and the mass density of the stones to 2.70 t/m^3. Interest rate including inflation is 5% p.a. Structure service lifetime is 50 years.

Downtime costs are set to 18.000 EURO/m breakwater for 1 km breakwater.

The formulae for prediction of recession listed in Table 4.5 are given in Appendix B1. Optimization raw data are given in Appendix B2.

Table 4.5. Case studies

Case study	Water depth (m)	Waves (see Table 11.1)	Formula for recession
1.1	11	Follonica	Sigurdarson et al. (2007)
1.2	-	-	Sigurdarson et al. (2008)
1.3	-	-	Sigurdarson et al. (2013)
1.4	-	-	Lykke Andersen et al. (2014)
2.1	20	Baltic Sea	Sigurdarson et al. (2007)
2.2	-	-	Sigurdarson et al. (2008)
2,3	-	-	Sigurdarson et al. (2013)
2.4	-	-	Lykke Andersen et al. (2014)

The results of the case studies are given in Tables 4.6 – 4.14 and Figs. 4.6-4.13 in terms of lifetime costs as function of H_o and H_s- design return period T. The nominal diameter of the main berm armour, D1, the probability of Repair1, P_{R1}, the probability of Repair 2, P_{R2}, and the probability of failure , P_{failur} all within the 50 years lifetime of the structure, are values related to the minimum total costs shown in bold in the tables.

The data shown in the tables are extracted from the raw data tables presented in Annex B2. The extracted numbers shown might be marginally different from the raw data tables due to repeated simulations. The tables allow identification of all combinations of design parameters, costs and probabilities of repair and failure.

Table 4.6. Case study 1.1 results. 11 m water depth, Sigurdarson et al. 2007 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	4.35	**10.360**	10.404	12.427	17.777	28.131	1.48	0.6980	0.0000	0.0000
25	5.07	12.465	11.922	**11.430**	12.043	14.463	1.29	0.7992	0.0000	0.0000
50	5.36	13.541	12.872	12.045	**11.932**	12.993	1.17	1.1752	0.0000	0.0014
100	5.64	14.668	13.899	12.825	**12.348**	12.612	1.24	0.6139	0.0000	0.0000
200	5.92	15.845	14.981	13.723	12.992	**12.772**	1.14	0.8564	0.0000	0.0000
400	6.20	17.070	16.108	14.688	13.755	**13.276**	1.19	0.4684	0.0000	0.0000
1000	6.56	18.756	17.671	16.039	14.917	**14.181**	1.26	0.2132	0.0000	0.0000

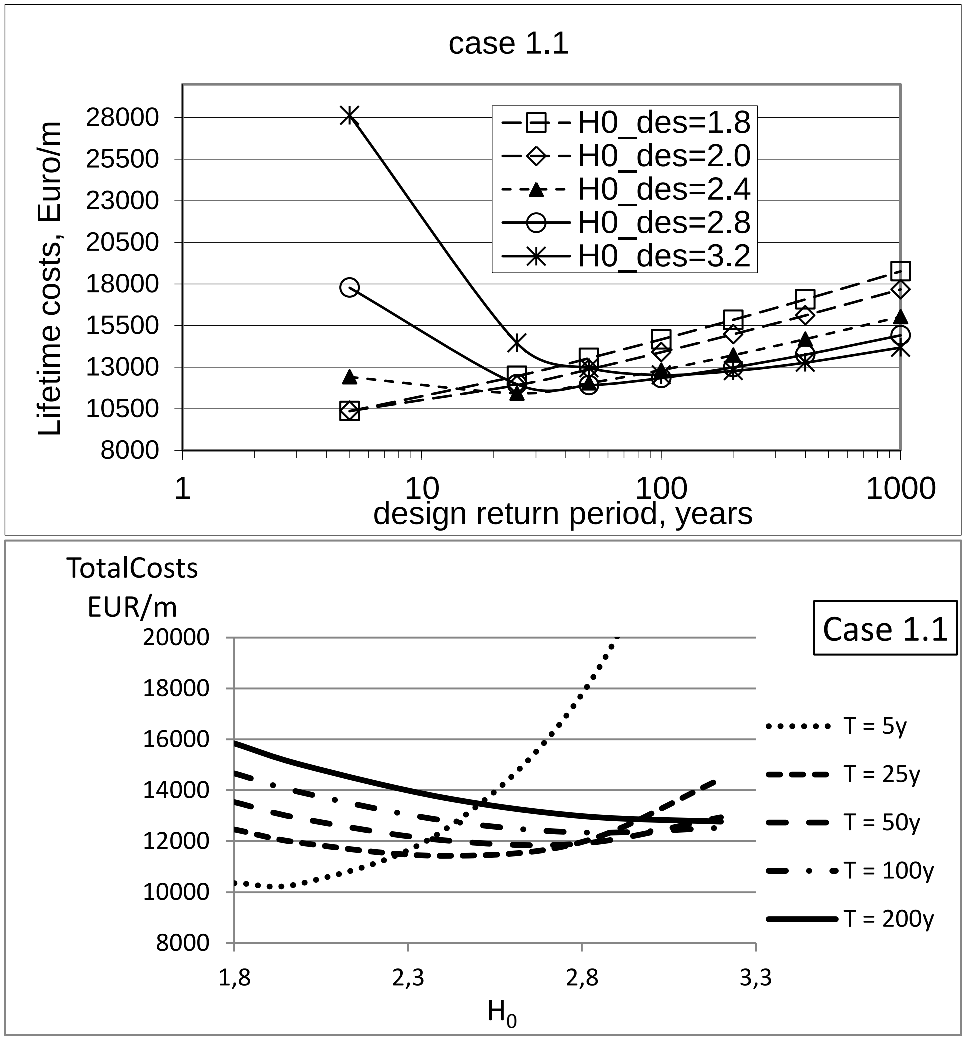


Fig. 4.6 . Case 1.1 results

Table 4.7. Case study 1.2 results. 11 m water depth, Sigurdarson et al. 2008 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m H_o					Cost minimum values			
		1.8	2.0	2.4	2.8	3.2	D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
5	4.35	9.956	**9.677**	11.508	19.776	42.869	1.33	0.2716	0.0000	0.0056
25	5.07	12.403	11.773	**10.983**	11.864	16.191	1.29	0.2198	0.0000	0.0008
50	5.36	13.513	12.797	11.794	**11.664**	13.628	1.17	0.5137	0.0000	0.0204
100	5.64	14.659	13.863	12.685	**12.044**	12.742	1.24	0.2512	0.0000	0.0011
200	5.92	15.844	14.965	13.636	12.794	**12.696**	1.14	0.5229	0.0000	0.0139
400	6.20	17.070	16.105	14.639	13.637	**13.094**	1.19	0.2796	0.0000	0.0005
1000	6.56	18.756	17.671	16.024	14.841	**14.064**	1.26	0.1229	0.0000	0.0000

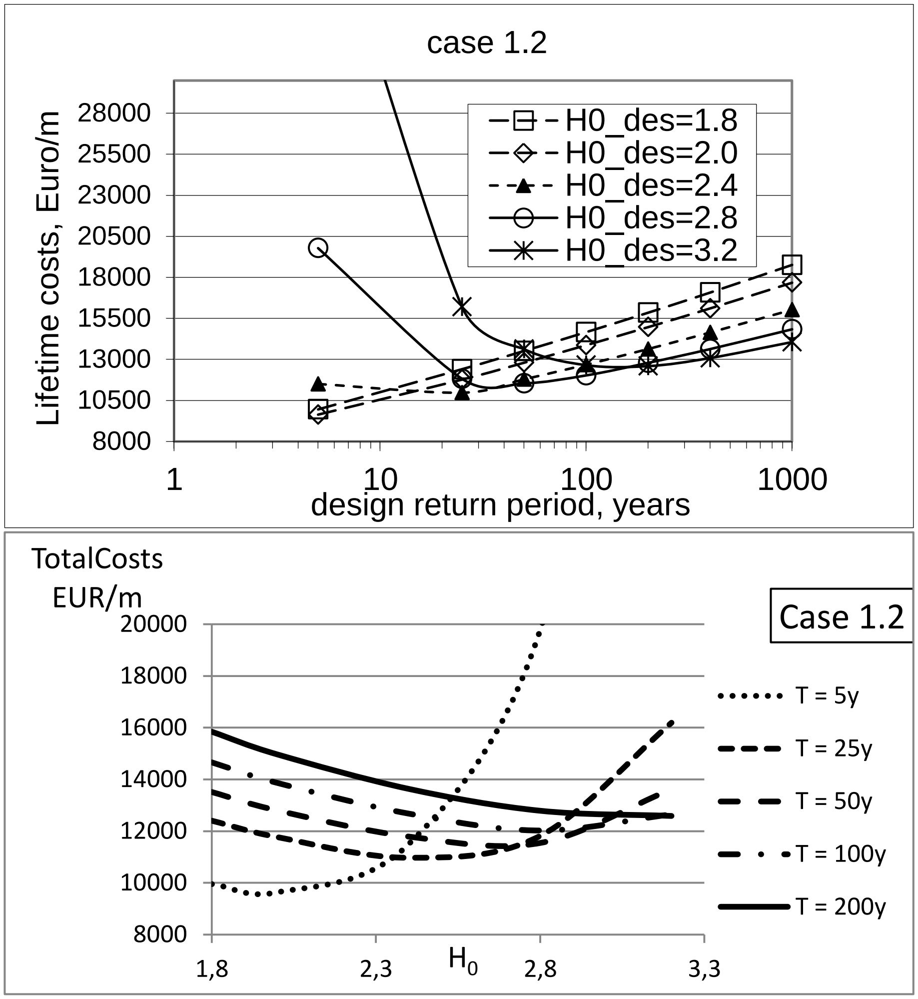


Fig. 4.7. Case 1.2 results

Table 4.8 Case 1.3 results. 11 m water depth, Sigurdarson et al. 2013 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	4.35	10.038	**10.067**	13.031	25.171	55.556	1.33	0.4285	0.0000	0.0260
25	5.07	12.407	11.795	**11.295**	13.284	19.964	1.29	0.3561	0.0000	0.0144
50	5.36	13.513	12.807	**11.911**	12.352	15.548	1.37	0.1650	0.0000	0.0028
100	5.64	14.659	13.864	12.730	**12.442**	14.109	1.24	0.4014	0.0000	0.0182
200	5.92	15.844	14.965	13.657	**12.945**	13.549	1.14	0.2000	0.0000	0.0039
400	6.20	17.070	16.105	14.649	13.714	**13.631**	1.19	0.4313	0.0000	0.0209
1000	6.56	18.756	17.671	16.025	14.877	**14.262**	1.26	0.2036	0.0000	0.0039

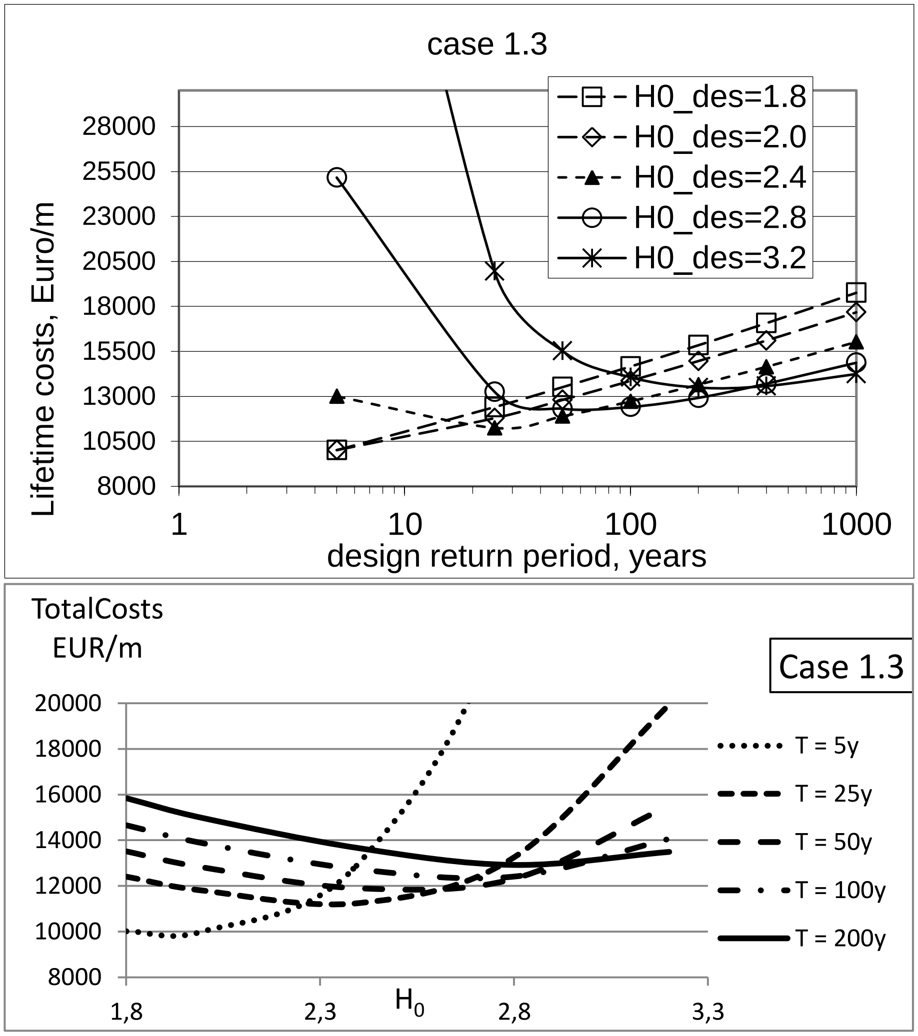


Fig. 4.8. Case 1.3 results

Table 4.9 Case study 1.4 results. 11 m water depth. Lykke Andersen et al. 2014 formula. All data

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	4.35	9.918	9.450	8.797	**8.553**	8969	0.95	0.7392	0.0000	0.0000
25	5.07	12.405	11.763	10.789	10.107	**9.668**	0.97	0.2244	0.0000	0.0000
50	5.36	13.514	12.797	11.707	10.914	**10.345**	1.03	0.0723	0.0000	0.0014
100	5.64	14.659	13.863	12.651	11.767	**11.107**	1.08	0.0234	0.0000	0.0000
200	5.92	15.844	14.965	13.628	12.654	**11.909**	1.14	0.0053	0.0000	0.0000
400	6.20	17.070	16.105	14.638	13.586	**12.748**	1.19	0.0007	0.0000	0.0000
1000	6.56	18.756	17.671	16.024	14.824	**13.905**	1.26	0.0004	0.0000	0.0000

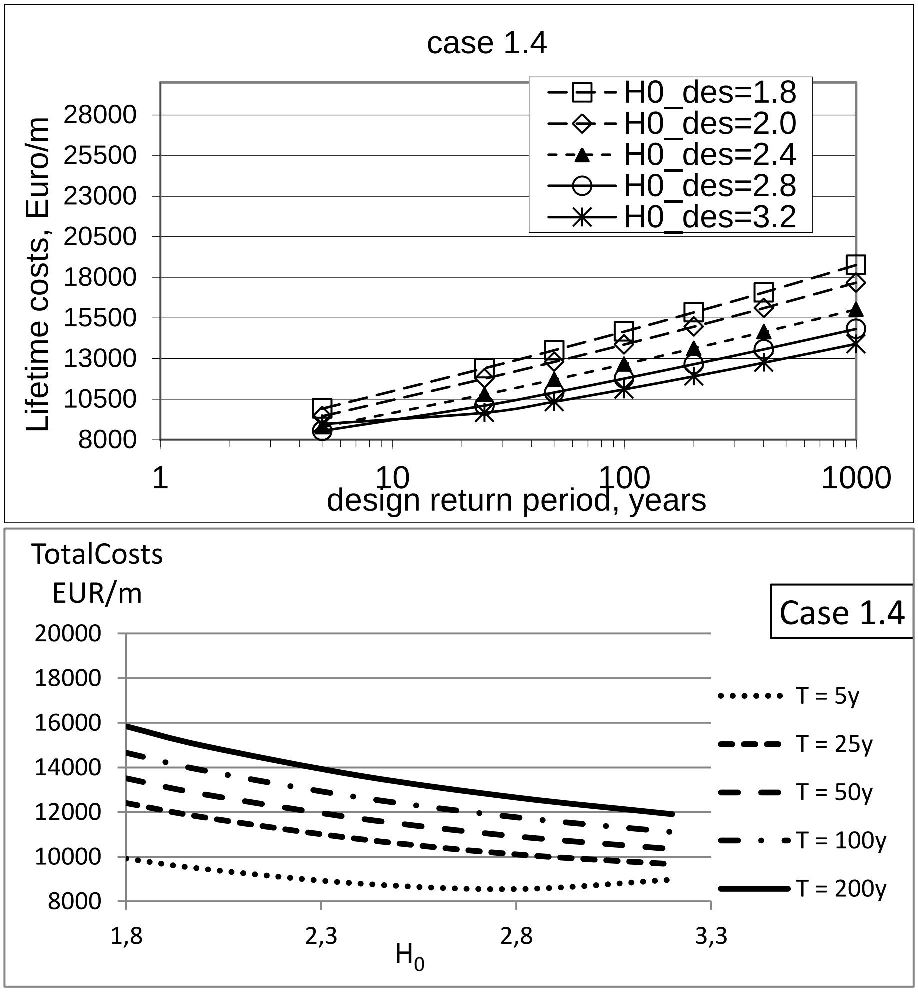


Fig. 4.9. Case 1.4 results

Table 4.10. Case study 2.1 results. 20 m water depth, Sigurdarson et al. 2007 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	3.55	**20.365**	21.996	26.504	33.218	42.853	1.21	1.6423	0.2903	0.4553
25	4.71	21.108	**20.988**	21.580	22.912	25.034	1.45	0.5048	0.0368	0.1207
50	5.36	23.704	23.052	**22.684**	23.116	23.926	1.37	0.4677	0.0050	0.1063
100	6.08	27.364	26.549	25.236	**24.666**	24.643	1.33	0.3817	0.0012	0.0793
200	6.88	32.512	31.181	29.093	27.975	**27.272**	1.32	0.3025	0.0002	0.0642
400	7.75	39.115	37.241	34.399	32.490	**31.087**	1.49	0.1570	0.0000	0.0286
1000	9.00	49.268	46.595	42.600	39.702	**37.742**	1.73	0.0698	0.0000	0.0126

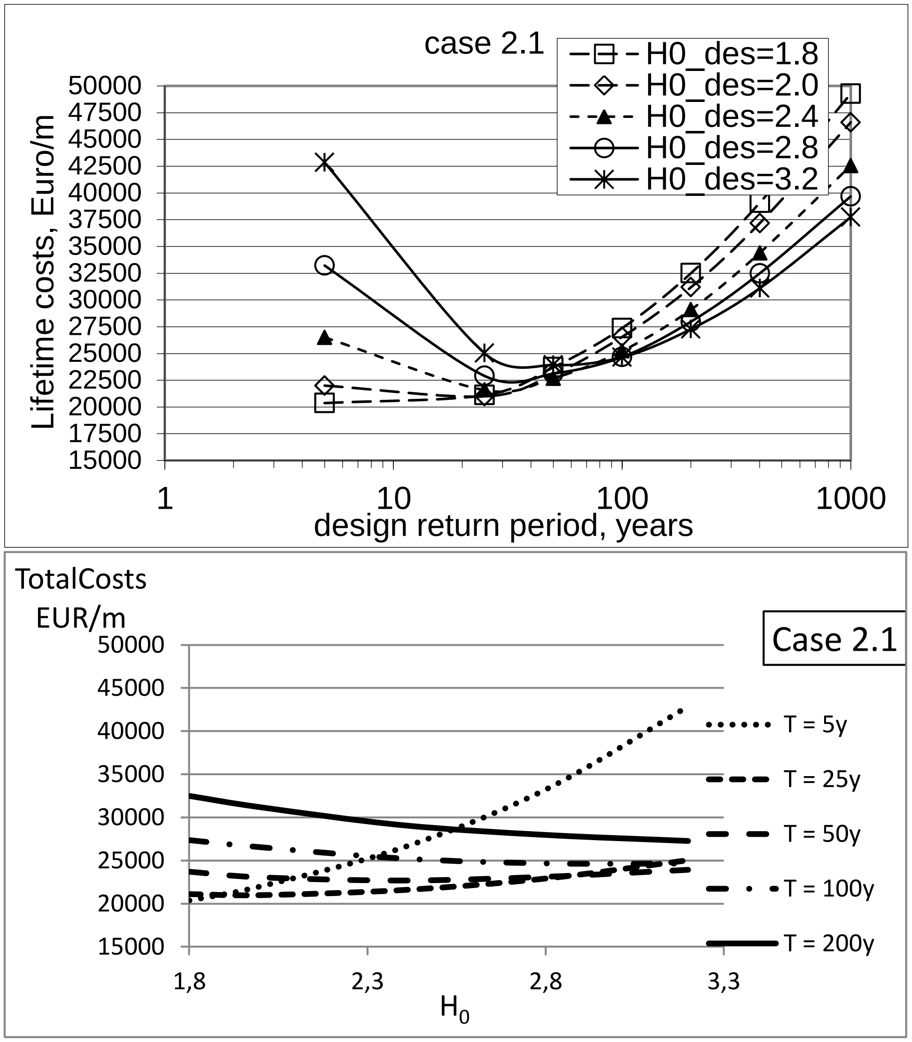


Fig. 4.10. Case 2.1 results

Table 4.11. Case study 2.2 results. 20 m water depth, Sigurdarson et al. 2008 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	3.55	**19.894**	21.974	28.867	40.305	56.927	1.21	0.6412	0.2807	0.4470
25	4.71	**20.932**	21.020	22.040	25.094	29.162	1.61	0.1420	0.0624	0.0869
50	5.36	23.571	**22.958**	22.996	24.034	26.094	1.64	0.1060	0.0083	0.0599
100	6.08	27.219	26.397	**25.439**	25.484	26.248	1.56	0.1206	0.0000	0.0692
200	6.88	32.494	31.113	29.185	28.178	**27.954**	1.32	0.2115	0.0000	0.1049
400	7.75	39.070	37.187	34.456	32.761	**31.614**	1.49	0.1155	0.0000	0.0520
1000	9.00	49.233	46.561	42.573	39.910	**37.904**	1.73	0.0513	0.0000	0.0184

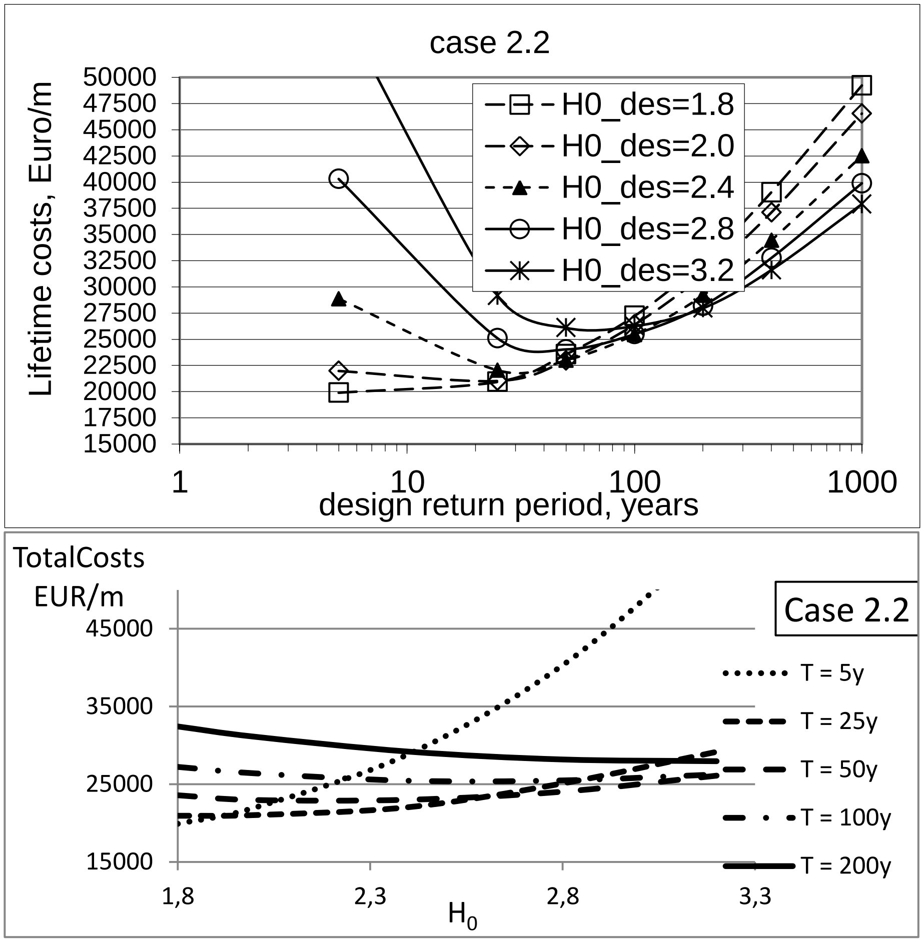


Fig. 4.11. Case 2.2 results

Table 4.12. Case study 2.3 results. 20 m water depth, Sigurdarson et al. 2013 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	3.55	**20.485**	22.873	30.309	43.145	63.395	1.21	0.8129	0.2453	0.5003
25	4.71	**21.146**	21.285	22.635	25.454	30.261	1.61	0.1737	0.0548	0.0983
50	5.36	23.638	**23.275**	23.317	24.565	27.150	1.64	0.1318	0.0106	0.0760
100	6.08	27.424	26.506	**25.483**	25.634	26.765	1.56	0.1394	0.0013	0.0671
200	6.88	32.556	31.178	29.305	**28.472**	28.386	1.51	0.1437	0.0003	0.0659
400	7.75	39.130	37.206	34.571	32.729	**31.939**	1.49	0.1251	0.0003	0.0600
1000	9.00	49.241	46.575	42.668	39.910	**38.089**	1.73	0.0528	0.0000	0.0229

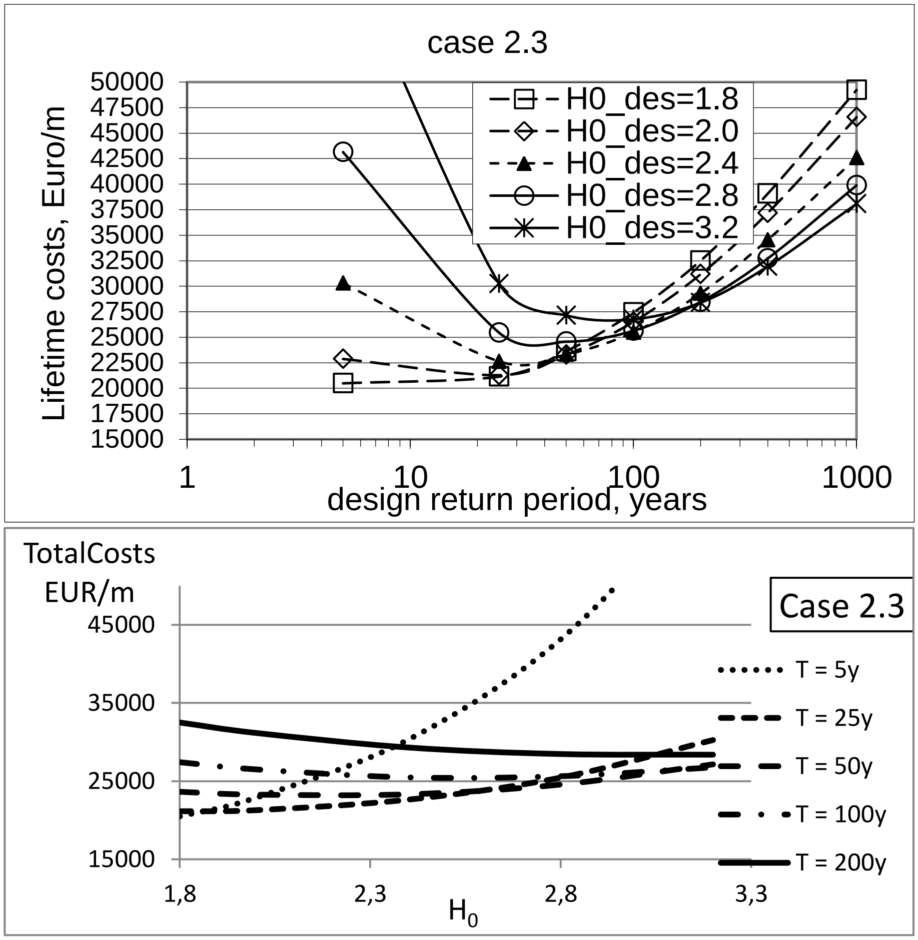


Fig. 4.12. Case 2.3 results

Table 4.13. Case study 2.4 results. 20 m water depth, Lykke Andersen et al. 2014 formula

H_s design return period T (years)	H_s (m)	Lifetime costs in 1000 EUR/m					Cost minimum values			
		H_o					D1 (m)	P_{R1}	P_{R2}	$P_{failure}$
		1.8	2.0	2.4	2.8	3.2				
5	3.55	**17.034**	17.487	19.349	22.144	25.452	1.21	0.4193	0.5420	0.1785
25	4.71	19.978	19.512	**18.969**	19.251	19.525	1.20	0.3067	0.0757	0.0673
50	5.36	22.932	22.129	21.112	20.551	**20.202**	1.03	0.4791	0.0312	0.0651
100	6.08	26.939	25.825	24.202	23.094	**22.278**	1.17	0.2431	0.0247	0.0147
200	6.88	32.284	30.770	28.476	26.846	**25.652**	1.32	0.1176	0.0119	0.0022
400	7.75	39.039	37.048	34.005	31.790	**30.114**	1.49	0.0472	0.0043	0.0004
1000	9.00	49.232	46.541	42.437	39.442	**37.159**	1.73	0.0151	0.0000	0.0000

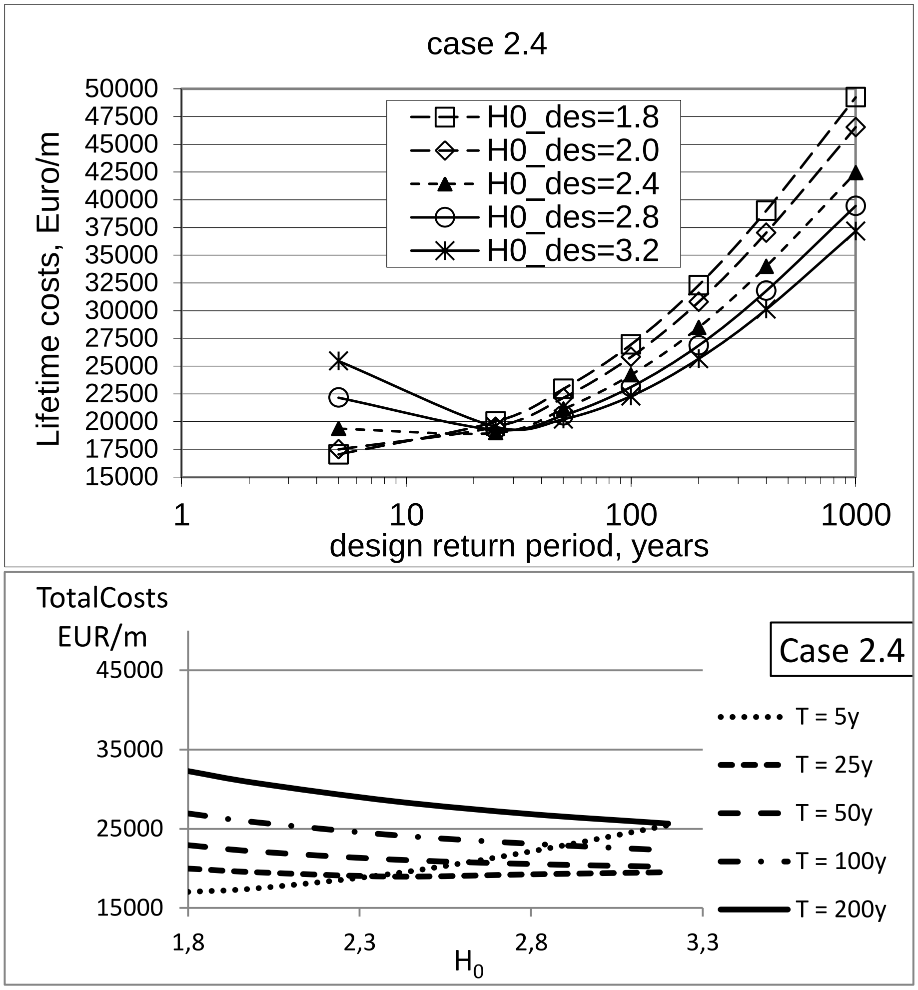


Fig.4.13. Case 2.4 results

4.4 Conclusions on optimum safety levels

The simulations show that that the berm breakwater concept is very robust as very low probabilities of damage and failures are obtained for a large range of combinations of H_o – values and design wave return periods. The fairly flat minima of lifetime costs indicate flexibility in the combined application of the two parameters.

For *shallow water wave conditions* Tables 4.6 – 4.8 and Figs. 4.6 – 4.8, based on formulae Sigurdarson et al. (2007, 2008, 2013), show that the most economical designs are obtained by applying the 5 year return period waves and H_o = 1.8 – 2.0. Table 4.9 and Fig. 4.9, based on formula Lykke Andersen et al. (2014), show that the most economical design is obtained by applying the 5 years return period waves and H_o = 2.4 – 2.8. The results for shallow water conditions as extracted from the tables given in Section 4.3 and Appendix B2 are summarized in Table 4.14. The difference in predictions between formula B4 and the other three formulae is discussed in Appendix B1. Formula B1 which gives larger values of P_{R1} is regarded less reliable related to Icelandic type berm breakwaters than the other formulae

Table 4.14. Summary of optimum design conditions for shallow waters. 50 years' lifetime

Case	Formula	H_o	Wave return period (y)	D1(m)	P_{R1}	P_{R2}	$P_{failure}$	Total costs EUR/m	Construc.costs EUR/m
1.1	B1 Sigurdarson et al. (2007)	1.8 2.0	5 -	1.48 1.33	0.70 1.40	0.000 -	0.000 -	10360 10366	9909 9437
1.2	B2 Sigurdarson et al. (2008)	1.8 2.0	5 -	1.48 1.33	0.07 0.27	0.000 -	0.000 -	9952 9677	9909 9437
1.3	B3 Sigurdarson et al. (2013)	1.8 2.0	5 -	1.48 1.33	0.14 0.43	0.000 -	0.000 -	10011 10016	9909 9437
1.4	B4 L.Andersen et al. (2014)	2.4 2.8	5 -	1.14 0.95	0.15 0.74	0.000 -	0.000 -	8795 8553	8716 8186

For *deep water wave conditions* Tables 4.10 – 4.13 and Figs. 4.10 – 4.13 all show that the most economical designs are obtained by applying the 5years or 25 years return period waves and H_o = 1.8. The results as extracted from the tables given in Section 4.3 and Appendix B2 are summarized in Table 4.15.

Table 4.15. Summary of optimum design conditions for deep waters. 50 years' lifetime

Case	Formula	H_o	Wave return period (y)	D1 (m)	P_{R1}	P_{R2}	$P_{failure}$	Total costs EUR/m	Construc. costs EUR/m
1.1	B1 Sigurdarson et al. (2007)	1.8 2.0	5 25	1.21 1.45	1.6 0.50	0.290 0.037	0.455 0.121	20365 20988	14633 18759
1.2	B2 Sigurdarson et al. (2008)	1.8 1.8	5 25	1.21 1.61	0.64 0.14	0.281 0.062	0.447 0.087	19894 20932	14633 19399
1.3	B3 Sigurdarson et al. (2013)	1.8 1.8	5 25	1.21 1.61	0.81 0.17	0.245 0.055	0.500 0.098	20485 21146	14633 19399
1.4	B4 L.Andersen et al. (2014)	1.8 1.8	5 25	1.21 1.61	0.42 0.08	0.542 0.113	0.179 0.027	17034 19978	14633 19399

The identified small return periods for the design waves is an unconventional result but is a consequence of the parameterized cross section shown in Fig 4.5 in which the height and volume of the structure increase with H_s and thereby with the design wave return period. Also the very ductile damage development and the relatively low repair costs favor small return period design waves. Consequently the construction/initial costs of the structure are smaller the smaller the H_s -value applied in the design. The more frequent repairs which are a consequence of the related smaller stone size do not change this picture, even if the repair costs are increased by 20 – 30%.

If designing for the small return period waves then low values of H_o should be used resulting in fairly large armour stone sizes which limit the probability of repair and failure.

Designing for larger return period waves leads in any case to higher lifetime costs. The H_o –values corresponding to the cost minimum increase with the design wave return period.

If designing for H_o - values > 2.8, larger reshaping takes place and transport of stones along the structure in case of oblique waves might occur. This is outside the range for the Icelandic type berm breakwaters.

It might be reasonable - as design basis - to choose a maximum probability of P_{R1} = app. 0.5 within a structure lifetime of 50 years. The related optimum design conditions correspond to design wave return periods in the range 5 - 25 years and H_0 – values in the range 1.8 – 2.0. For shallow water wave conditions most probably H_o = 2.4.

The optimum design conditions are very much dependent on the availability and costs of the various rock sizes.

5. Optimum safety levels of Accropode breakwaters

5.1 Armour characteristics, cross sections and failure modes

Accropodes belong to the class of single layer type of armour units the characteristics of which are the complex shape which assure interlocking of the blocks when placed in the armour layer. The interlocking works better on steeper slopes for which reason slopes equal to or steeper than 1:1.5 are used.

Examples of complex type blocks are Tetrapods, Dolos, Accropodes, CoreLocs and Xblocks.

In the present work is used Accropods as representative for this type of blocks. Fig. 5.1 shows an Accropode.

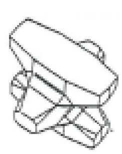


Fig.5.1 Accropod

The height of the Accropode block is denoted H. The block volume V = 0.34 H^3.

The cost optimization procedure, repair strategy and costs of downtime are the same as applied for the conventional rubble mound breakwaters, Chapter 3.

The parameterized cross section applied in the analyses is shown in Fig. 5.2.

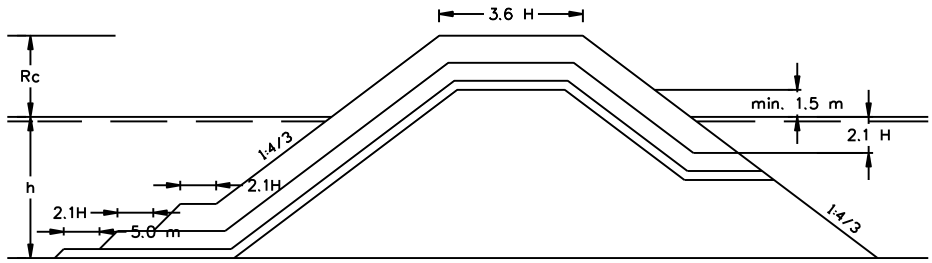


Fig.5.2 Parameterized cross section.

The requests to the freeboard R_c and the level for the construction road are the same as were applied for the conventional rubble mound breakwaters, cf. Chapter 3.

The only failure mode considered is displacements of armour units.

5.2 Limit state performance and repair strategy

More details are given in Appendix C1.

The limit state performance and repair strategy are the same as applied for the conventional rubble mound breakwaters, Chapter 3.

The patent holders for single layer complex types of armour units, e.g. Sogreah/CLI for Accropodes, DMC for Xblocs, recommend for design a specific value of the Hudson formula stability factor K_D but information on uncertainties and damage as function of K_D – values are not given as it does not exist. In the present analyses the formula by Burcharth et al. (1998) is applied because it provides information on the development of damage and the related scatter which is a necessity for the optimization simulations. The formula reads

$$N_s = \frac{H_s}{\Delta D_n} = A(D^{0.2} + 7.70) \tag{5.1}$$

in which the mean of A is 0.46, the coefficient of variation is $0.02 + 0.05(1-D)^6$ and D is the relative number of units displaced more than the distance D_n = (block volume)$^{1/3}$.

The formula is valid for irregular, head-on waves, breaking and non-breaking waves, Accropodes placed on slope 1:1.33 in accordance with SOGREAH/CLI recommendations. Range of minimum stability, $\xi_p = 3.5 - 4.5$ corresponding to wave steepness s_{op} = 0.03 – 0.05.

5.3 Overview of case studies, case study data, costs and identified optimum safety levels

Table 5.1 gives the main data for the case studies including the built-in unit prices for the various parts of the structures. The characteristics of the applied waves denoted Follonica and Bilbao are given in Table 2.1.

The damage accumulation model given in Chapter 3 for rubble mounds is applied.

Table 5.1. Case study data

Case	Water depth	Concrete mass density	Origin of waves	Stability formula	Built-in unit prices core/filter 1/filter 2/armor in EURO/m^3
1	10 m	Accropode $2.4\,t/m^3$	Follonica	Burcharth et al. (1998)	15/20/30/ 80 or 160*
2	20 m		Bilbao		
3			North Sea		

* Costs of repair doubled (i.e. 160) corresponding to the fact that almost twice the number of Accropodes must be replaced due to the interlocking

It is important to notice that CLI recommends limits to the size of the Accropodes. Such limits are not implemented in the simulations for which reason the very large sizes shown in the following tables exceed the recommended sizes.

Table 5.2 and Figs. 5.3 – 5.6 show the results of Case 1 extracted from the simulation raw data sheets given in Appendix C2. In Table 5 and the following tables, N_L indicates the average number of occurrence of limit state within the service lifetime of the breakwater.

Fig. 5.4 shows the variation of total cost, initial construction cost, and repair cost with respect to the armour weight. While the initial cost increases almost linearly with the armour weight, the repair cost rapidly decreases to almost zero at 12 ton armour weight. As expected, the repair costs contribute significantly to the total cost when the armour weight is small, but the total cost approaches to the initial cost as the armor weight increases.

Fig. 5.5 compares total costs versus armour weight normalized with respect to the optimal value for different interest rates. The same variation is seen for all interest rates.

Table 5.2. Case 1. Optimum safety levels for Accropode armoured breakwaters. 50 year' service lifetime. 10 m water depth. Damage accumulation included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS		
2	None	100 (2.9)	5.64	7.31	0.0374	0.0060	0.0245	14118	15097
5		50 (2.9)	5.36	6.26	0.0449	0.0128	0.0569	13297	14502
8		50 (2.9)	5.36	6.26	0.0516	0.0130	0.0598	13297	14143
2	Included	100 (2.9)	5.64	7.31	0.0327	0.0064	0.0248	14118	15459
5		5 (2.3)	4.35	6.70	0.0419	0.0097	0.0409	13649	14903
8		50 (2.9)	5.36	6.26	0.0509	0.0127	0.0584	13297	14458

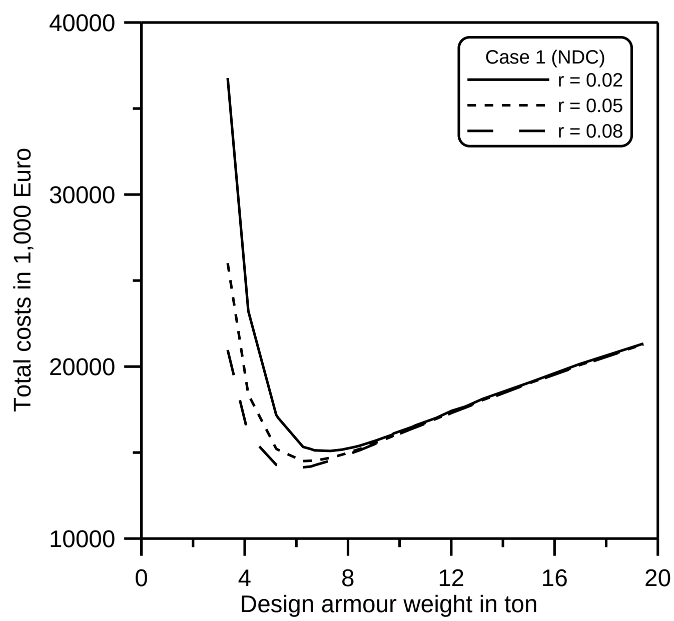


Fig.5.3. Case 1. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation included. No downtime costs included.

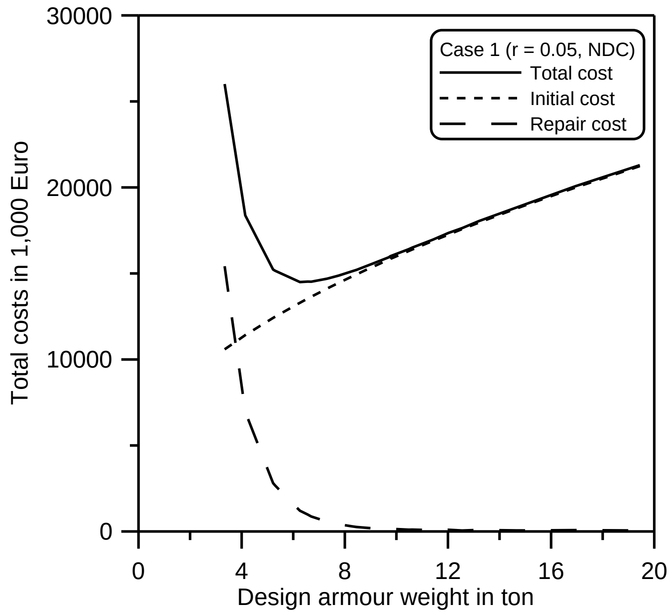


Fig.5.4. Case 1. Costs in 50 years lifetime as total cost, initial cost and repair cost and armour unit mass used in deterministic design. 5% interest rate. Damage accumulation included. No downtime costs included.

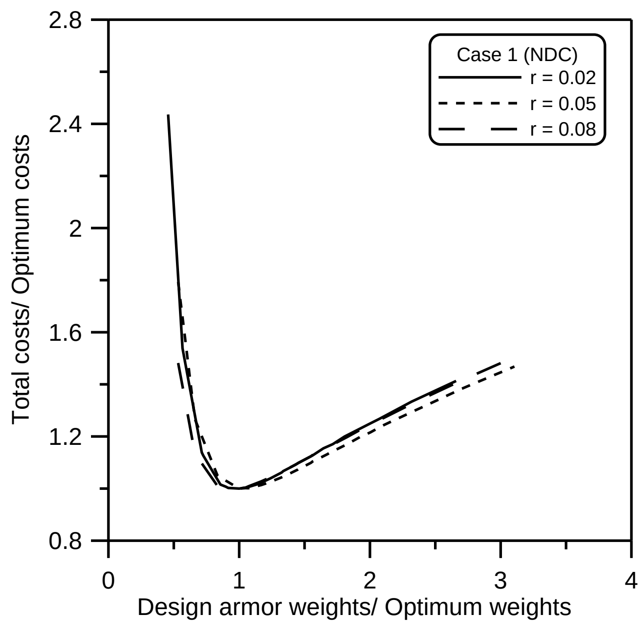


Fig.5.5. Case 1. Normalized total costs in 50 years lifetime versus normalized deterministic design armour weight as function of real interest rate. Damage accumulation included. No downtime costs included.

For practical design it is of interest to analyze the near optimal safety levels, i.e. within a range corresponding to slightly larger lifetime cost than the identified minimum cost. As examples, values corresponding to up to 5%

increase in lifetime costs are shown in Tables 5.3 – 5.5 and Figs. 5.6 – 5.8. In Table 5.3 seven cases within the + 5% costs are identified. Such information is a better basis for the designer to select the preferred design. It is generally preferable to choose a conservative design in order to reduce the political and financial inconveniences related to repairs. As an example taken from Table 6 for 2% interest rate, the economical optimum corresponds to armour mass 7.31 t and the SLS and ULS failure probabilities correspond to 3.7% and 2.5%, respectively. If an armour unit mass of 8.46 t is chosen the lifetime costs will increase by 2 %, but the SLS and ULS failure probabilities reduce to 2.3% and 1%, respectively.

Table 5.3. Case 1. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 2% interest rate. 10 m water depth. Damage accumulation included. No downtime costs

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
2	None	50 (2.9)	5.36	6.26	0.0449	0.0128	0.0569	13297	15331	1.02
		25 (2.7)	5.07	6.56	0.0439	0.0103	0.0452	13539	15206	1.01
		5 (2.3)	4.35	6.7	0.0446	0.0093	0.0395	13649	15131	1.00
		100 (2.9)	5.64	7.31	0.0374	0.006	0.0245	14118	15097	1.00
		50 (2.7)	5.36	7.76	0.0346	0.0042	0.017	14449	15169	1.00
		25 (2.5)	5.07	8.27	0.0318	0.0033	0.0114	14817	15331	1.02
		200 (2.9)	5.92	8.46	0.0234	0.0027	0.01	14951	15398	1.02
		100 (2.7)	5.64	9.06	0.0256	0.0019	0.0063	15371	15690	1.04

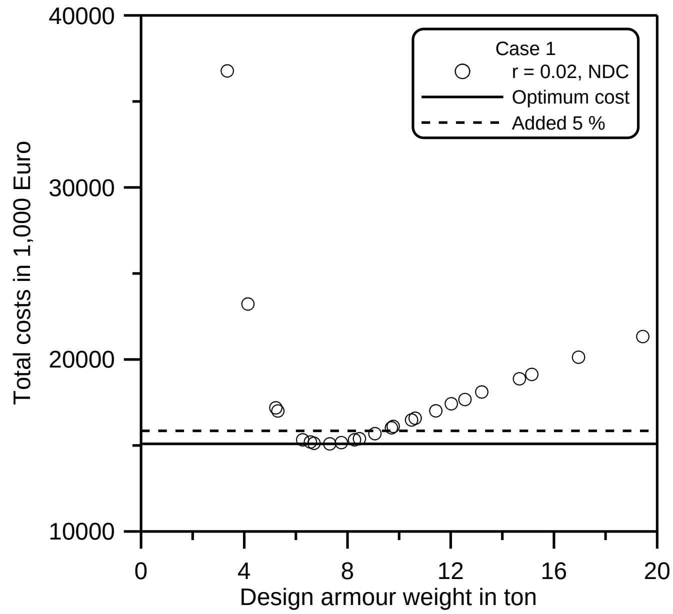


Fig.5.6. Case 1. Design armour weight in relation to minimum and + 5% lifetime costs. 2 % interest rate. 50 year service lifetime. Damage accumulation included. No downtime costs included.

Table 5.4. Case 1. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 5% interest rate. 10 m water depth. Damage accumulation included. No downtime costs

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs (Kd)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
5	None	5 (2.5)	4.35	5.22	0.0771	0.0303	0.1484	12417	15226	1.05
		25 (2.9)	5.07	5.3	0.0718	0.0276	0.1391	12485	15157	1.05
		50 (2.9)	5.36	6.26	0.0449	0.0128	0.0569	13297	14502	1.00
		25 (2.7)	5.07	6.56	0.0439	0.0103	0.0452	13539	14526	1.00
		5 (2.3)	4.35	6.7	0.0446	0.0093	0.0395	13649	14521	1.00
		100 (2.9)	5.64	7.31	0.0374	0.006	0.0245	14118	14694	1.01
		50 (2.7)	5.36	7.76	0.0346	0.0042	0.017	14449	14874	1.03
		25 (2.5)	5.07	8.27	0.0318	0.0033	0.0114	14817	15118	1.04
		200 (2.9)	5.92	8.46	0.0234	0.0027	0.01	14951	15212	1.05

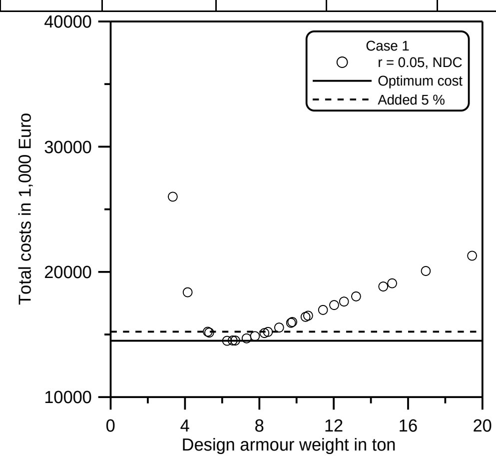


Fig.5.7. Case 1. Design armour weight in relation to minimum and + 5% lifetime costs. 5 % interest rate. 50 year service lifetime. Damage accumulation included. No downtime costs included.

Table 5.5. Case 1. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 8% interest rate. 10 m water depth. Damage accumulation included. No downtime costs.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs(Kd)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
8	None	5 (2.5)	4.35	5.22	0.0768	0.0293	0.1443	12417	14289	1.01
		25 (2.9)	5.07	5.30	0.081	0.0268	0.1353	12485	14273	1.01
		50 (2.9)	5.36	6.26	0.0516	0.013	0.0598	13297	14143	1.00
		25 (2.7)	5.07	6.56	0.0358	0.0103	0.0448	13539	14188	1.00
		5 (2.3)	4.35	6.70	0.0467	0.0091	0.0416	13649	14262	1.01
		100 (2.9)	5.64	7.31	0.0467	0.0059	0.0239	14118	14529	1.03
		50 (2.7)	5.36	7.76	0.0422	0.0043	0.0176	14449	14750	1.04

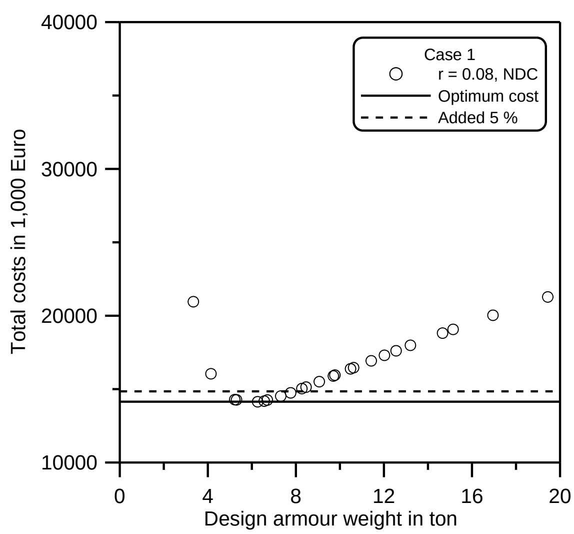


Fig.5.8. Case 1. Design armour weight in relation to minimum and + 5% lifetime costs. 8 % interest rate. 50 year service lifetime. Damage accumulation included. No downtime costs included.

Tables 5.6 – 5.9 and Figs. 5.9 – 5.14 show some results for Case 2 which represents a deep water condition with almost no depth limitation of wave height. Case 2 examines the influences of downtime cost, damage accumulation and structure lifetime on optimal design.

It is important to note that there is a limit to the size of the Accropode unit. SOGREAH/CLI must be consulted about this question. So far units exceeding app. 40 - 50 t are not recommended.

Table 5.6. Case 2. Optimum safety levels for Accropode armoured breakwater. 50 year service lifetime. 20 m water depth. Damage accumulation included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS		
2	None	25 (2.7)	9.93	49.52	0.0261	0.0100	0.0477	49291	55565
5		25 (2.7)	9.93	49.52	0.0232	0.0102	0.0502	49291	53088
8		50 (2.9)	10.52	47.52	0.0273	0.0130	0.0606	48500	51721
2	Included	100 (2.9)	11.09	55.65	0.0175	0.0060	0.0260	51624	55595
5		50 (2.9)	10.52	47.52	0.0267	0.0127	0.0622	48500	53604
8		50 (2.9)	10.52	47.52	0.0267	0.0127	0.0622	48500	51947

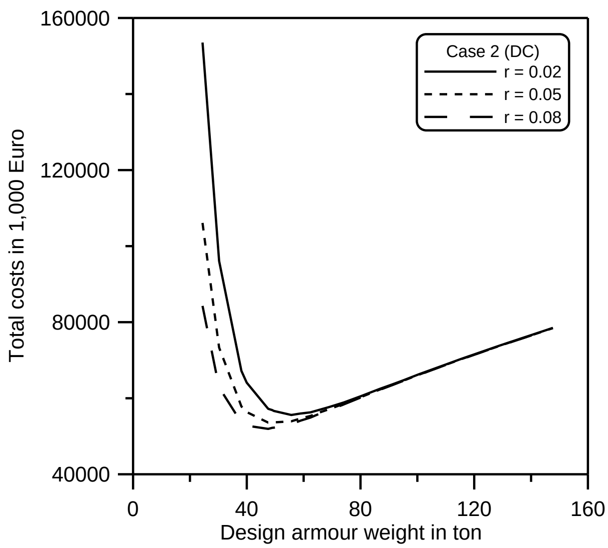


Fig.5.9. Case 2. Total costs in 50 years lifetime as function of real interest rate and deterministic armour unit mass. Damage accumulation included. Downtime costs included.

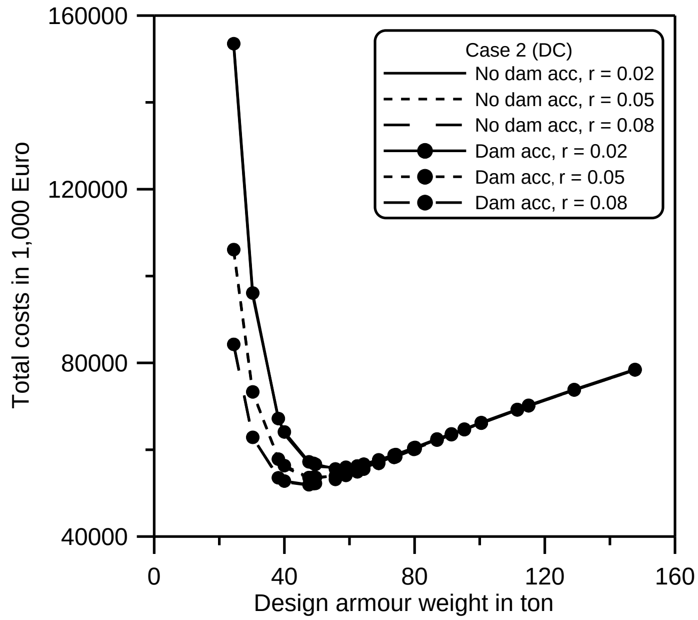


Fig.5.10. Case 2. Influence of damage accumulation on total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Downtime costs included.

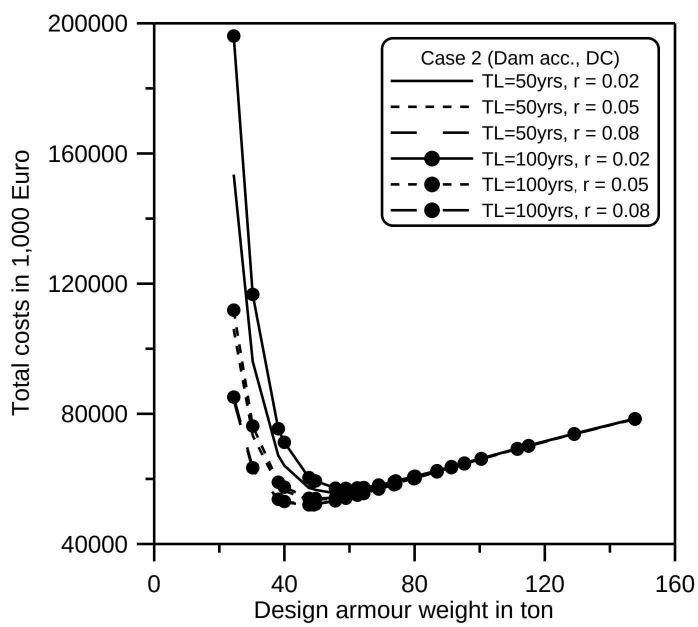


Fig.5.11. Case 2. Total costs in 50 years and 100 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation and downtime costs included.

Table 5.7. Case 2. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 2% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
2	Included	50 (2.9)	10.52	47.52	0.0267	0.0127	0.0622	48500	57224	1.03
		5 (2.3)	8.43	48.96	0.0275	0.0113	0.0537	49068	56871	1.02
		25 (2.7)	9.93	49.52	0.0257	0.0111	0.05	49291	56652	1.02
		100 (2.9)	11.09	55.65	0.0175	0.006	0.026	51624	55595	1.00
		50 (2.7)	10.52	58.88	0.017	0.005	0.0197	52810	55982	1.01
		25 (2.5)	9.93	62.39	0.0148	0.0036	0.0135	54061	56280	1.01
		200 (2.9)	11.64	64.36	0.0146	0.003	0.0112	54751	56692	1.02
		100 (2.7)	11.09	68.95	0.0133	0.0019	0.0075	56321	57683	1.04

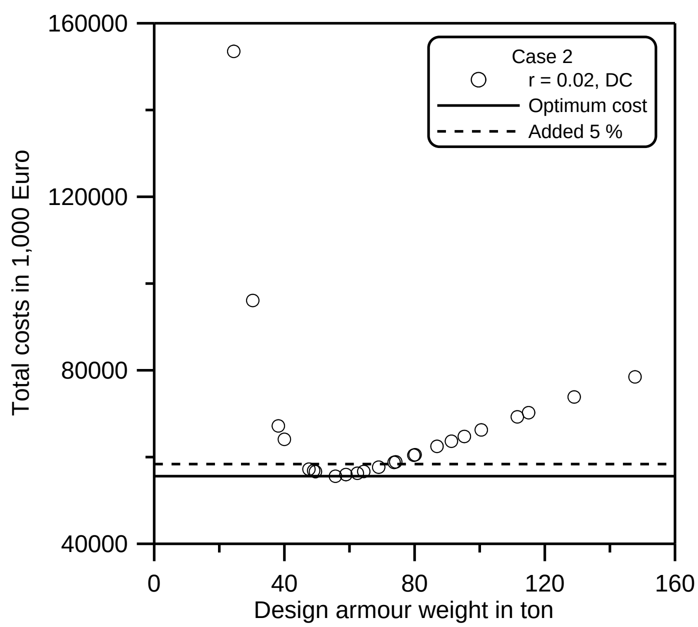


Fig.5.12. Case 2. Design armour weight in relation to minimum and + 5% lifetime costs. 2 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

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Table 5.8. Case 2. . Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 5% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
5	Included	25 (2.9)	9.93	39.97	0.0503	0.0278	0.1438	45366	56355	1.05
		50 (2.9)	10.52	47.52	0.0267	0.0127	0.0622	48500	53604	1.00
		5 (2.3)	8.43	48.96	0.0275	0.0113	0.0537	49068	53692	1.00
		25 (2.7)	9.93	49.52	0.0257	0.0111	0.05	49291	53650	1.00
		100 (2.9)	11.09	55.65	0.0175	0.006	0.026	51624	53937	1.01
		50 (2.7)	10.52	58.88	0.017	0.005	0.0197	52810	54692	1.02
		25 (2.5)	9.93	62.39	0.0148	0.0036	0.0135	54061	55347	1.03
		200 (2.9)	11.64	64.36	0.0146	0.003	0.0112	54751	55906	1.04

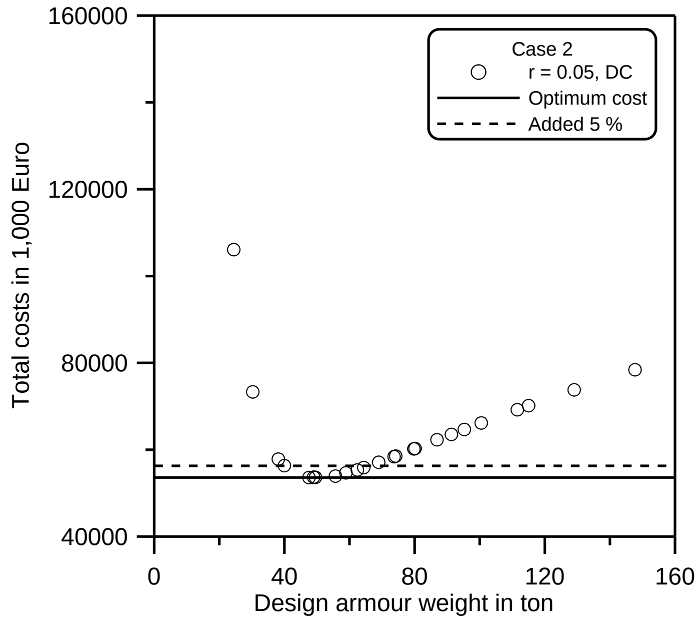

Fig.5.13. Case 2. Design armour weight in relation to minimum and + 5% lifetime costs. 5 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

Table 5.9. Case 2. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 8% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum costs
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
8	Included	5 (2.5)	8.43	38.12	0.0582	0.0326	0.1772	44559	53556	1.03
		25 (2.9)	9.93	39.97	0.0503	0.0278	0.1438	45366	52795	1.02
		50 (2.9)	10.52	47.52	0.0267	0.0127	0.0622	48500	51947	1.00
		5 (2.3)	8.43	48.96	0.0275	0.0113	0.0537	49068	52226	1.01
		25 (2.7)	9.93	49.52	0.0257	0.0111	0.050	49291	52255	1.01
		100 (2.9)	11.09	55.65	0.0175	0.0060	0.0260	51624	53176	1.02
		50 (2.7)	10.52	58.88	0.0170	0.0050	0.0197	52810	54097	1.04

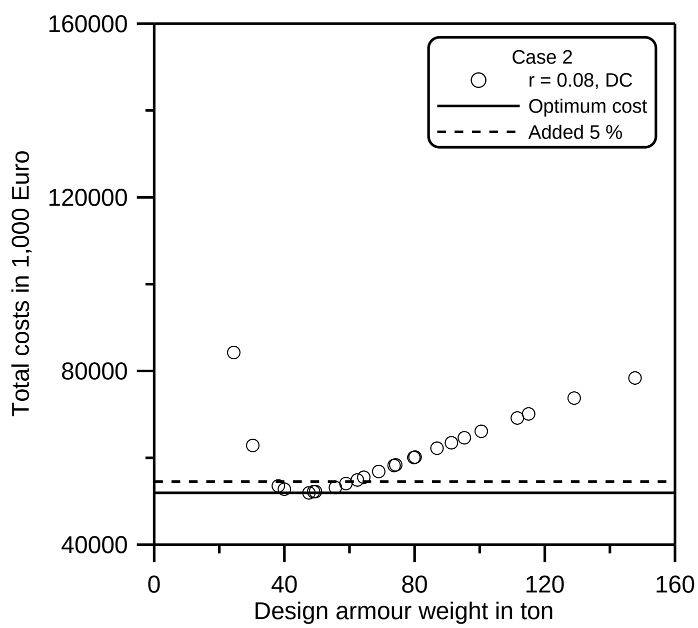

Fig.5.14. Case 2. Design armour weight in relation to minimum and + 5% lifetime costs. 8 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

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Tables 5.10 – 5.13 and Figs. 5.15 – 5.18 show the result of Case 3 which also represents a deep water condition. It is interesting to notice that the average number of occurrence of repairable limit state is much smaller than those of serviceability or ultimate limit state. The same results are observed for Cases 1 and 2. These results are again attributed to the brittle failure development of Accropode armor layer.

Table 5.10. Case 3. Optimum safety levels for concrete Accropode armored breakwater. 50 year service lifetime. 20 m water depth. Damage accumulation included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)
		Optimum design return period, T yrs (Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS		
2	None	100 (2.9)	8.76	27.41	0.0313	0.0057	0.0187	39459	41377
5		5 (2.5)	7.21	23.86	0.0453	0.0132	0.0471	37567	40168
8		25 (2.9)	8.09	21.61	0.0573	0.0239	0.0888	36298	39297
2	Included	100 (2.9)	8.76	27.41	0.0313	0.0057	0.0187	39459	41552
5		50 (2.9)	8.43	24.45	0.0415	0.0114	0.0411	37895	40505
8		5 (2.5)	7.21	23.86	0.0420	0.0136	0.0486	37567	39579

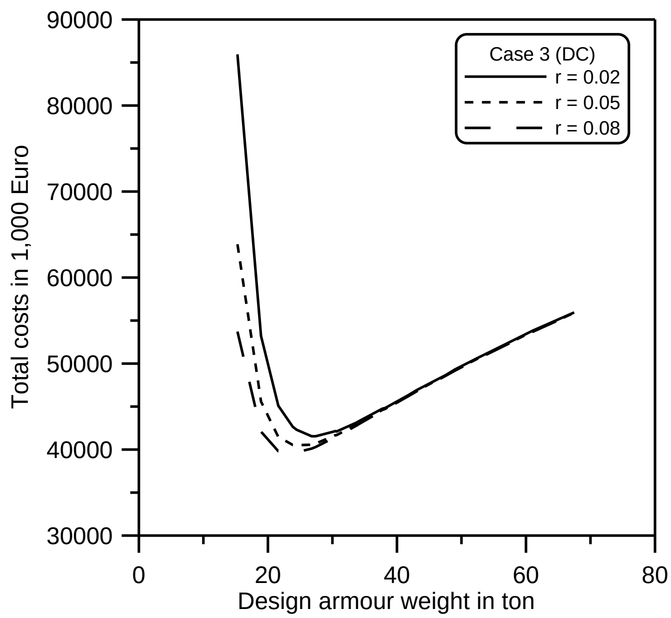

Fig.5.15. Case 3. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation included. Downtime costs included.

Table 5.11. Case 3. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 2% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum Costs
		Optimum design return period, T yrs(Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
2	Included	5 (2.5)	7.21	23.86	0.042	0.0136	0.0486	37567	42650	1.03
		50 (2.9)	8.43	24.45	0.0415	0.0114	0.0411	37895	42300	1.02
		25 (2.7)	8.09	26.77	0.0305	0.0063	0.021	39132	41552	1.00
		100 (2.9)	8.76	27.41	0.0244	0.0052	0.0183	39459	41552	1.00
		50 (2.7)	8.43	30.3	0.0264	0.0032	0.0089	40918	42106	1.01
		200 (2.9)	9.08	30.47	0.0268	0.0028	0.0087	41002	42148	1.01
		5 (2.3)	7.21	30.64	0.0191	0.0027	0.008	41082	42101	1.01
		400 (2.9)	9.38	33.66	0.0192	0.0014	0.0039	42529	43115	1.04
		25 (2.5)	8.09	33.73	0.0199	0.0012	0.0038	42562	43129	1.04
		100 (2.7)	8.76	33.96	0.0173	0.0012	0.0039	42669	43244	1.04

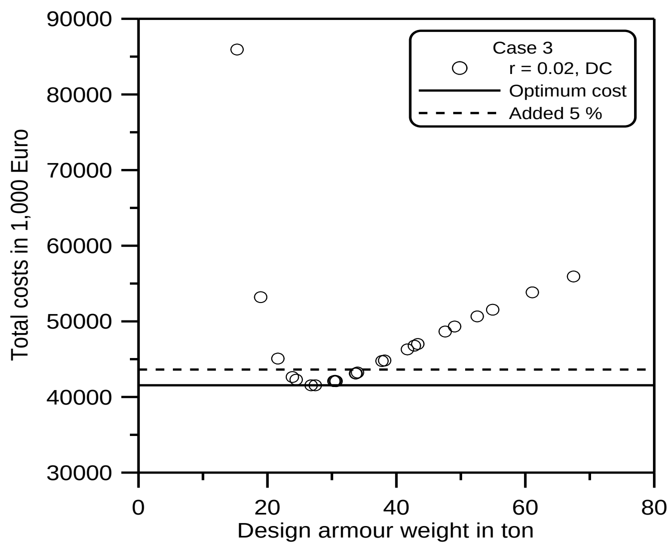

Fig.5.16. Case 3. Design armour weight in relation to minimum and + 5% lifetime costs. 2 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

Table 5.12. Case 3. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 5% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum Costs
		Optimum design return period, T yrs(Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
5	Included	25 (2.9)	8.09	21.61	0.0537	0.0234	0.0879	36298	41472	1.02
		5 (2.5)	7.21	23.86	0.042	0.0136	0.0486	37567	40541	1.00
		50 (2.9)	8.43	24.45	0.0415	0.0114	0.0411	37895	40505	1.00
		25 (2.7)	8.09	26.77	0.0305	0.0063	0.0210	39132	40570	1.00
		100 (2.9)	8.76	27.41	0.0244	0.0052	0.0183	39459	40697	1.00
		50 (2.7)	8.43	30.3	0.0264	0.0032	0.0089	40918	41618	1.03
		200 (2.9)	9.08	30.47	0.0268	0.0028	0.0087	41002	41675	1.03
		5 (2.3)	7.21	30.64	0.0191	0.0027	0.008	41082	41685	1.03

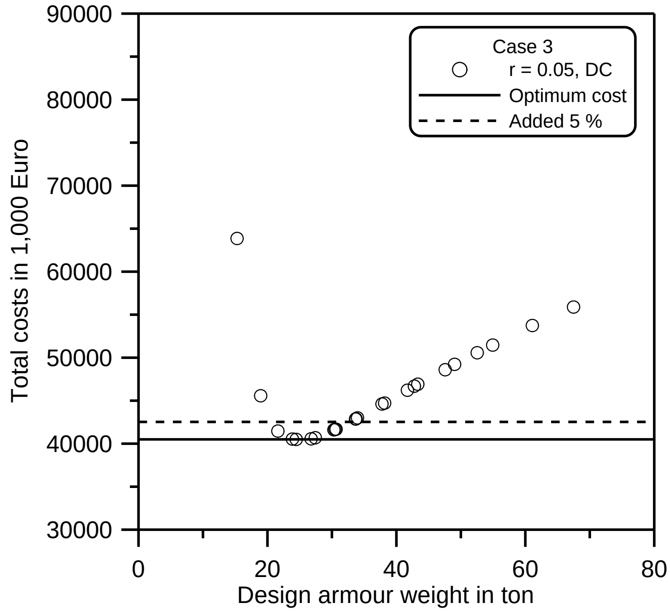

Fig.5.17. Case 3. Design armour weight in relation to minimum and + 5% lifetime costs. 5 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

Table 5.13. Case 3. Variation in safety levels within the range corresponding to minimum lifetime costs + 5%. 50 year service lifetime. 8% interest rate. 20 m water depth. Damage accumulation and downtime costs included.

Real interest rate (%)	Downtime Costs	Deterministic design data			Optimum limit state average number of events within service lifetime			Initial costs (1,000 EURO)	Total costs (1,000 EURO)	Total costs/ Optimum Costs
		Optimum design return period, T yrs(Ns)	H_s^T (m)	Armor unit mass, W (t)	SLS	RLS	ULS			
8	Included	25 (2.9)	8.09	21.61	0.0537	0.0234	0.0879	36298	39810	1.01
		5 (2.5)	7.21	23.86	0.0420	0.0136	0.0486	37567	39579	1.00
		50 (2.9)	8.43	24.45	0.0415	0.0114	0.0411	37895	39677	1.00
		25 (2.7)	8.09	26.77	0.0305	0.0063	0.0210	39132	40115	1.01
		100 (2.9)	8.76	27.41	0.0244	0.0052	0.0183	39459	40303	1.02
		50 (2.7)	8.43	30.30	0.0264	0.0032	0.0089	40918	41391	1.05
		200 (2.9)	9.08	30.47	0.0268	0.0028	0.0087	41002	41457	1.05
		5 (2.3)	7.21	30.64	0.0191	0.0027	0.0080	41082	41493	1.05

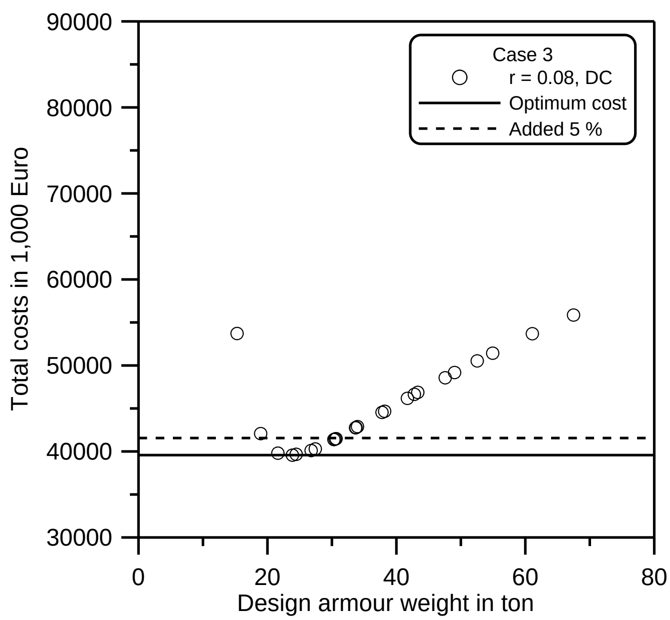

Fig.5.18. Case 3. Design armour weight in relation to minimum and + 5% lifetime costs. 8 % interest rate. 50 year service lifetime. Damage accumulation and downtime costs included.

5.4 Conclusions

5.4.1 Optimum safety levels

The identified optimum safety levels in terms of failure probability within 50 years' service lifetime are given in Table 5.14. Safety level values are given for the range 1 – 1.05 of the total costs/ optimum costs.

Table 5.14. Identified safety levels for conventional Accropode armoured breakwaters corresponding to minimum lifetime costs and slightly larger lifetime costs. Probabilities within 50 years' service lifetime

Wave conditions	Interest rate % p.a.	SLS	RLS	ULS	Deterministic design Wave return Period (years)	N_s	Total costs /optimum costs
Shallow Water Case 1	2	0.035	0.004	0.027	50	2.7	1.00
		0.037	0.006	0.025	100	2.9	1.00
	5	0.044	0.010	0.045	25	2.7	1.00
		0.035	0.009	0.017	50	2.7	1.03
		0.045	0.013	0.057	50	2.9	1.00
		0.037	0.006	0.025	100	2.9	1.01
	8	0.036	0.010	0.045	25	2.7	1.00
		0.042	0.004	0.018	50	2.7	1.04
		0.052	0.013	0.060	50	2.9	1.00
Deep Water Case 2	2	0.017	0.005	0.020	50	2.7	1.01
		0.018	0.006	0.026	100	2.9	1.00
	5	0.026	0.011	0.05	25	2.7	1.00
		0.017	0.005	0.020	50	2.7	1.02
		0.027	0.013	0.062	50	2.9	1.00
	8	0.026	0.011	0.050	25	2.7	1.01
		0.017	0.005	0.020	50	2.7	1.04
		0.027	0.013	0.062	50	2.9	1.00
Deep Water Case 3	2	0.026	0.003	0.009	50	2.7	1.01
		0.024	0.005	0.018	100	2.9	100
	5	0.026	0.003	0.009	50	2.7	1.03
		0.042	0.011	0.041	50	2.9	1.00
		0.024	0.005	0.018	100	2.9	1.00
	8	0.026	0.003	0.009	50	2.7	1.05
		0.042	0.011	0.041	50	2.9	1.00

The results given in Table 5.14 are in Table 5.15 concentrated to give an easier overview.

Table 5.15. Identified safety levels for conventional Accropode armoured breakwaters corresponding to minimum lifetime costs and slightly larger lifetime costs. Probabilities within 50 years service lifetime

Wave	Interest	SLS	RLS	ULS	Deterministic design	Total costs

conditions	rate % p.a.				Wave return Period (years)	N_s	/optimum costs
Shallow Water Case 1	2	0.04 0.04	0.004 0.006	0.03 0.03	50 100	2.7 2.9	1.00 1.00
	5 – 8	0.04 0.05	0.004 0.004	0.02 0.06	50 50	2.7 2.9	1.03 – 1.04 1.00
Deep Water Case 2 and Case3	2	0.02 – 0.03 0.02 – 0.03	0.004 0.006	0.01 - 0.02 0.02 – 0.03	50 100	2.7 2.9	1.01 1.00
	5 - 8	0.02 – 0.03 0.03 – 0.04	0.004 0.009	0.01 – 0.02 0.04 – 0.06	50 50	2.7 2.9	1.02 – 1.05 1.00

The optimum safety levels for the analyzed *shallow water* conditions correspond for 2% p.a. interest rate roughly to exceedance of both SLS and ULS with a probability of app. 0.04 and 0.03 respectively within the 50 year service lifetime corresponding to a probability of app. 0.001 per year. In terms of deterministic design this corresponds to N_s = 2.7 and 50 years wave return period, or N_s = 2.9 and wave return period 100 years. For interest rates of 5% and 8% p.a. the optimum probability of exceedance of SLS and ULS are 0.05 and 0.06, respectively corresponding to N_s = 2.9 and 50 years wave return period. If 3 – 4% higher lifetime costs are accepted the probability of exceedance of SLS and ULS are 0.04 and 0.02 respectively corresponding to N_s = 2.7 and 50 years wave return period.

For the analyzed *deep water* conditions the optimum probabilities of exceeding SLS and ULS for interest rate 2% are 0.02 and 0.01 respectively corresponding to N_s = 2.7 and 50 years wave return period. For interest rates 5% and 8% p.a. the figures are app. 0.03 and 0.04 respectively, corresponding to N_s = 2.9 and wave return period 50 years.

If up to 5% higher lifetime costs are accepted the safety level for all cases might be simplified to 2% probability of exceeding SLS and ULS within 50 years lifetime corresponding to N_s = 2.7 and 50 years wave return period.

The identified optimum safety levels enclose the value Ns = 2.7 recommended for trunks by CLI and SOGREAH for deterministic design of Accropode armour. This Ns – value corresponds to the Hudson formula stability factor KD = 15. CLI and SOGREAH do not recommend the wave return period to be applied. However, if 50 or 100 years return periods are generally used, the identified optimum safety level corresponds quite accurately to the applied deterministic design praxis.

The identified optimum safety margin is much larger than identified for conventional rubble mound breakwater armour, see Chapter 3. The reason is the brittle damage development of single layer armour consisting of complex types of units placed on steep slopes, see Appendix C1. Application of a high safety margin implies that initiation of armour unit displacements takes place for significantly higher Ns – values than for conventional rock and cube armour layers.

The analyses show rather distinct minima of the total costs as function of armour unit mass, see for example Fig.5.3. Consequently it is beneficial to design for a more narrow range of armour unit masses than for conventional rubble mound armour which generally shows more flat cost minima, cf. Chapter 3. The reason for this difference is that for Accropode armour designed to cost minima the probability of SLS-repairs is practically zero. This means that the increase in costs for larger units solely represents the extra costs of the larger units. There is no counteracting savings related to fewer repairs as is the case for rock and cube armour for which an increase in armour unit size gives significantly less repairs.

5.4.2 Influence of real interest rate on optimum safety levels

The safety levels decreases slightly when the interest rate increases, c.f. Tables 5.15 and 5.15.

Fig. 5.3 shows that the optimal total cost and armour weight decrease as the real interest rate increases. Therefore, if the interest rate is high it is more economical to construct a breakwater with smaller initial cost and repair more frequently. However, it may not be easy to accept this design concept in practical financial and political perspectives. For rocks and cubes, sometime it is difficult to determine the optimal cost and armour weight because of the flat minimum, i.e. the total costs only change insignificantly near the optimal design point (Burcharth and Sorensen, 2005). However, it is relatively easy to find the optimal design point for Accropodes due to the more distinct minimum caused by the brittle behavior of the armour layer.

Fig.5.11 compares the total costs with respect to the armour weight for different interest rates between the lifetime of 50 years and 100 years. In general, the total cost increases as the structure lifetime increases. However, the difference between different structure lifetimes becomes small as the interest rate increases, because for high interest rates the monetary value rapidly decrease with time so that repair costs during longer lifetime become minimal when converted to the present value.

5.4.3 Influence of damage accumulation on optimum safety levels

Inclusion of damage accumulation has no influence on optimum armour unit mass, total costs and the ULS safety level but reduces slightly the optimum SLS safety level, cf. Fig.5.10. In conclusion the influence of damage accumulation is negligible.

5.4.4 Influence of down time costs on optimum safety level

As shown in Tables 5.2 and 5.6 the inclusion of downtime costs have negligible influence on total lifetime costs but increase the optimum safety levels. Even though the downtime costs of 200,000 EURO/day for 3 months are rather large, the influence on total lifetime cost is small. This is because the probability of breakwater failure leading to stoppage of harbour operations is very small.

5.4.5 Influence of service lifetime on optimum safety level

The ratio of optimum design failure probability to service lifetime is almost constant for each of the design limit states. This means that if for SLS the optimum number of exceedances of the SLS-damage level is one within a service life of 50 years, then it will be roughly two within a service life of 100 years.

More detailed information on the analyses is given in Appendix C1. The simulation raw data sheets are presented in Appendix C2.

5.5 Partial safety factors corresponding to optimum safety levels

Partial safety factor systems have been introduced to design of breakwaters (e.g. Burcharth and Sorensen, 2000). Since they only consider the structural safety, however, an additional optimal design should be performed afterward to consider the functionality and economics of the breakwater. To overcome this problem, we propose the partial safety factors for Accropode-armoured breakwaters based on the cost optimization results obtained in the previous section.

The stability formula for Accropode including the partial safety factors can be written as

$$\gamma_S \gamma_R \frac{H_s^{50}}{\Delta D_n} = \mu_A \left(D_C^{0.2} + 7.7 \right) \tag{5.2}$$

where γ_S and γ_R are the load and resistance safety factors, respectively, H_s^{50} is the significant wave height of 50-year return periods, $\mu_A (= 0.46)$ is the mean value of empirical coefficient of Accropode, D_C is the critical relative damage which is 0.05 and 0.3 for serviceability and ultimate limit state, respectively. In the previous section, we already evaluated the optimum wave height and armor weight and the average number of occurrence of limit states within the service lifetime of the breakwater. Partial safety factors are calculated using the all simulation results. At the same time, we observed the range of partial safety factors which were evaluated by using the following condition because the total cost changes rather slowly with respect to the armor weight in the vicinity of the optimal point.

$$C_T / C_O \leq 1.05 \tag{5.3}$$

where C_O is the optimal total cost. The data which satisfy this criterion are given in Tables 5.14 – 5.16.

To calculate the partial safety factors corresponding to the probability of failure, the failure probability for a service lifetime has to be determined for all the limit states (i.e., serviceability, repairable, and ultimate limit states). First, the annual probability of failure is calculated as

$$P_f^1 = N_E / 50 \tag{5.4}$$

If the annual failure events are independent over the service lifetime, the probability of failure during 50 years may be expressed as

$$P_f^{\,50} = 1 - (1 - P_f^{\,1})^{50} \tag{5.5}$$

This assumption simplifies the probability estimation, and it is reasonable in the case of rubble mound armor stability (US Army, 2006).

Using Eqs. (5.2) and (5.5), we calculated the partial safety factors. Since the downtime cost has little influence on the cost optimization, some data calculated with or without the damage costs were not included in Tables 5.14 – 5.16. The partial safety factors were given separately for each limit state in Tables 5.14 – 5.16. However, the safety factors should be proposed in such a way that they can be used for all the limit states. Fig.5.19 shows the plot of partial safety factor versus probability of failure for both limit states. Especially, In Fig. 5.19, the lower bound of probability of failure is set as 0.01. Since the data for each limit state are close each other, we calculate the best-fitting equation using all the data as

$$\gamma_S \gamma_R = 1.1418\left(P_f^{50}\right)^{-0.0576} \tag{5.6}$$

The coefficient of correlation is 0.95. Even though it may be possible to calculate γ_S and γ_R separately, we propose the overall safety factor $\gamma_S \gamma_R$ because the armor weight is ultimately calculated by Eq. (5.2).

Fig.5.20 compares the present safety factors for Accropode with those for Tetrapod, Cube, and rocks developed by PIANC (Burcharth and Sorensen, 2000) with respect to the probability of failure. Accropode is not included in the PIANC safety factor system. The curve for Accropode is quite similar to those for other armor units. For larger probability of failure at which Accropode shows brittle failure, a larger safety factor is necessary.

Table 5.14. Case 1, Partial safety factor corresponding to the failure probability on each limit state. Damage accumulation and no downtime cost included.

Downtime Costs	Deterministic design data	Average number of event exceeding each damage level ($=N_E$)					
	Armor unit mass, W (t)	SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)		
		N_E	$P_f^{\,50}$	$\gamma_S \gamma_R$	N_E	$P_f^{\,50}$	$\gamma_S \gamma_R$
None	5.22	0.2558	0.2262	1.2301	0.1484	0.1381	1.2654
	5.30	0.2385	0.2126	1.2363	0.1391	0.1300	1.2718
	6.26	0.1146	0.1084	1.3068	0.0569	0.0553	1.3444
	6.56	0.0994	0.0947	1.3274	0.0452	0.0442	1.3655

	6.70	0.0934	0.0893	1.3368	0.0395	0.0387	1.3751
	7.31	0.0679	0.0657	1.3762	0.0245	0.0242	1.4157
	7.76	0.0558	0.0543	1.4039	0.017	0.0169	1.4441
	8.27	0.0465	0.0455	1.4340	0.0114	0.0113	1.4751
	8.46	0.0361	0.0355	1.4449	0.01	0.0100	1.4863
	9.06	0.0338	0.0332	1.4782	0.0063	0.0063	1.5207

Table 5.15. Case 2. Partial safety factor corresponding to the failure probability on each limit state. Damage accumulation and downtime cost included.

Downtime Costs	Deterministic design data	Average number of event exceeding each damage level ($=N_E$)					
	Armor unit mass, W (t)	SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)		
		N_e	P_f^{50}	$\gamma_S \gamma_R$	N_e	P_f^{50}	$\gamma_S \gamma_R$
Included	38.12	0.2680	0.2356	1.2159	0.1772	0.1627	1.2508
	39.97	0.2219	0.1994	1.2353	0.1438	0.1341	1.2707
	47.52	0.1016	0.0967	1.3086	0.0622	0.0603	1.3462
	48.96	0.0925	0.0884	1.3217	0.0537	0.0523	1.3596
	49.52	0.0868	0.0832	1.3267	0.0500	0.0488	1.3648
	55.65	0.0495	0.0483	1.3793	0.0260	0.0257	1.4189
	58.88	0.0417	0.0409	1.4055	0.0197	0.0195	1.4459
	62.39	0.0319	0.0314	1.4329	0.0135	0.0134	1.4740
	64.36	0.0288	0.0284	1.4478	0.0112	0.0111	1.4894
	68.95	0.0227	0.0224	1.4815	0.0075	0.0075	1.5240

Table 5.16. Case 3. Partial safety factor corresponding to the failure probability on each limit state. Damage accumulation and downtime cost included.

Downtime Costs	Deterministic design data	Average number of event exceeding each damage level ($= N_E$)					
		SLS (D $\geq$ 5 %)			ULS (D $\geq$ 30 %)		
	Armor unit mass, W (t)	N_e	P_f^{50}	$\gamma_S\,\gamma_R$	N_e	P_f^{50}	$\gamma_S\,\gamma_R$
Included	21.61	0.1650	0.1523	1.2558	0.0879	0.0842	1.2918
	23.86	0.1042	0.0991	1.2980	0.0486	0.0475	1.3352
	24.45	0.0940	0.0898	1.3086	0.0411	0.0403	1.3461
	26.77	0.0578	0.0562	1.3487	0.0210	0.0208	1.3874
	27.41	0.0479	0.0468	1.3594	0.0183	0.0181	1.3984
	30.3	0.0385	0.0378	1.4056	0.0089	0.0089	1.4459
	30.47	0.0383	0.0376	1.4082	0.0087	0.0087	1.4486
	30.64	0.0298	0.0294	1.4108	0.0080	0.0080	1.4513
	33.66	0.0245	0.0242	1.4557	0.0039	0.0039	1.4975
	33.73	0.0249	0.0246	1.4567	0.0038	0.0038	1.4985
	33.96	0.0224	0.0222	1.4600	0.0039	0.0039	1.5019

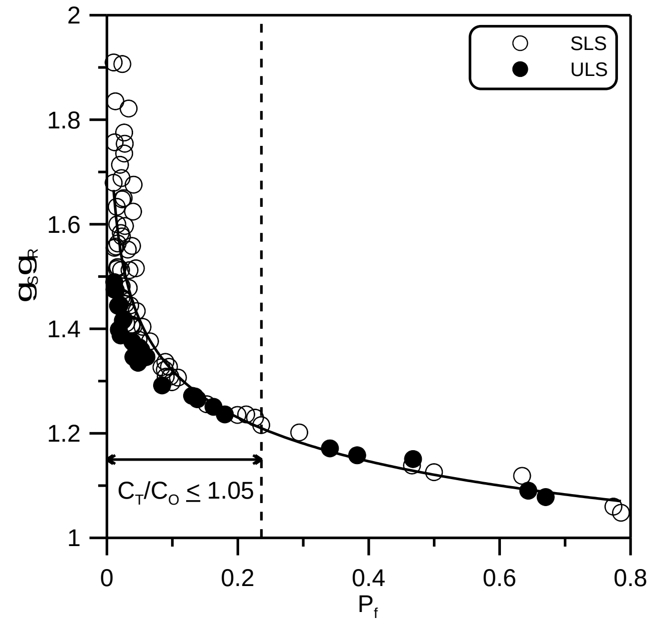

Fig.5.19. Relationship between the partial safety factors and failure probability within lifetime 50 years

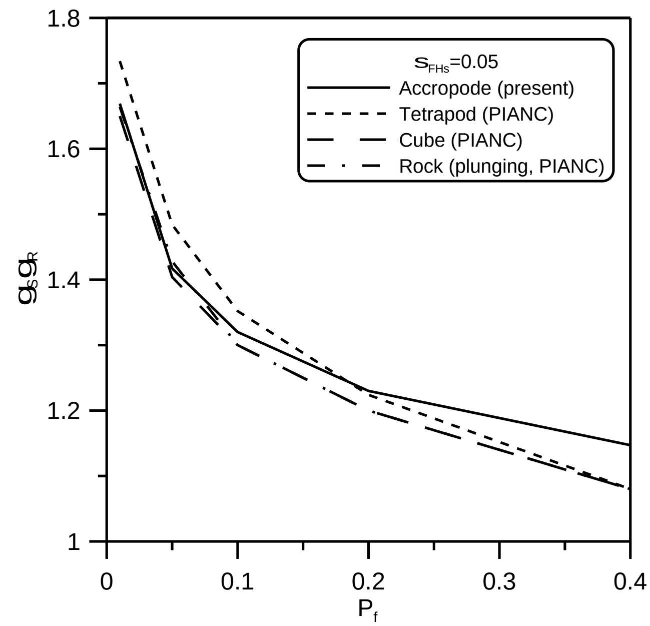

Fig.5.20. Comparison of partial safety factors between Accropode and other armor units

Table 5.17. Comparison of partial safety factors for armour units

P_f	Present	PIANC ($\sigma_{FH_s} = 0.05$)	
	Accropode	Rock	Cube
0.01	1.67	1.66	1.65
0.05	1.42	1.43	1.40
0.10	1.32	1.30	1.30
0.20	1.23	1.20	1.20
0.40	1.15	1.08	1.08

6. Optimum safety levels of caisson breakwaters

6.1 Cross sections and failure modes

Fig. 6.1 shows the cross sections dealt with in the simulations. In accordance with Japanese recommendations given by OCDI (2002) for outer breakwaters is chosen a freeboard of $h_c = 0.6 \cdot H_s^{T_L}$, where T_L is the design life time of the structure.

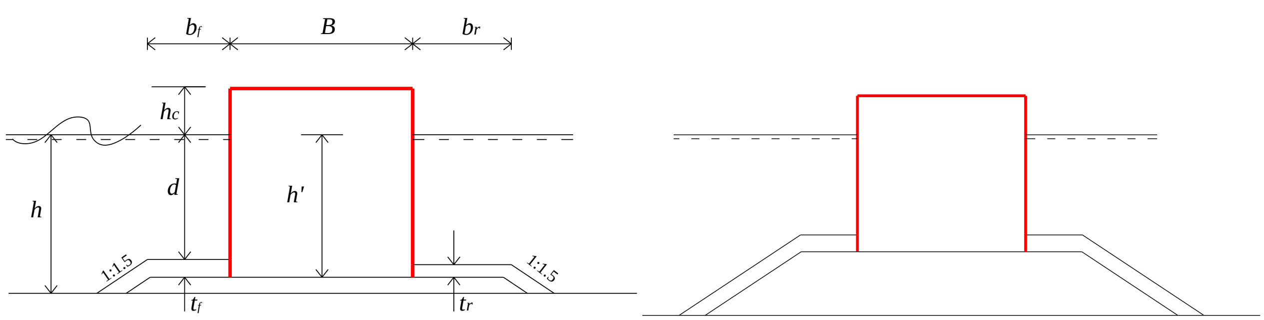

Fig. 6.1. Cross sections of outer caisson breakwaters on bedding layer (top) and on high mound foundation (bottom).

The ratio between the caisson draft *h'* and the water depth *h* has been varied in all the simulations in order to identify the most economical ratios.

Conditions both with sea bed materials strong enough to resist slip failures (hard bottom) and sandy sea beds have been analysed. The studied failure modes are shown in Fig. 6.2. For the slip failure the angle θ giving the lowest resistance has been identified.

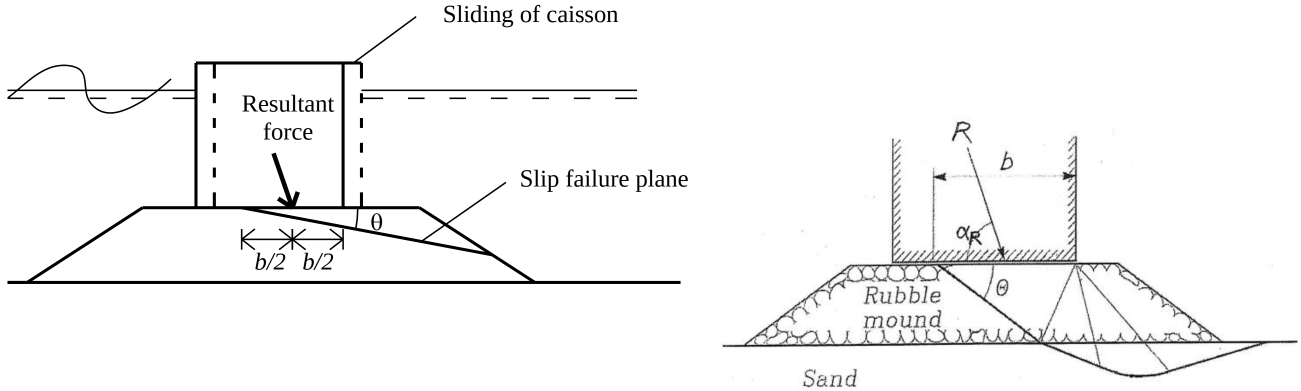

Fig. 6.2. Failure modes included in the optimization

Toe berm stability has not been included because the extra cost of maкıng the berm armour very safe is too small to have significant influence on the cost optimization.

6.2 Limit state performance, strategy and costs of repair

Two methods of repair/stabilization as shown in Fig. 6.3 are considered; armour blocks in front and/or a rubble mound behind the caisson.

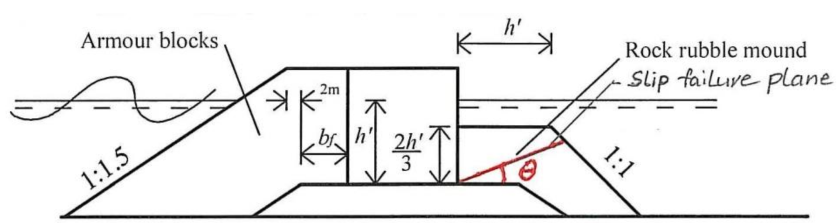

Fig. 6.3. Armour blocks in front of caisson and rubble mound behind caisson as means of repair.

The used limit state performances and related methods of repair are given in Table 6.1.

Table 6.1. Limit state performances and repair.

Limit states	Failures	Repair
SLS	Sliding distance 0.2 m	No
RLS	Sliding distance 0.5 m	Armour blocks in front or mound behind
ULS	Sliding distance 2.0 m Slip failure	Both Both , double unit costs

The chosen sliding distances are assumed reasonable values for outer breakwaters with no berths arranged on the harbour side of the caissons.

Table 6.2 provides the average built-in bulk unit prices collected by the PIANC MarCom Working Group 47 members. The Japanese prices are used in the present analyses. For the identification of the optimum safety level only the ratio between the costs of the various parts of the structure including repairs are of importance.

The unit price for the caissons is kept constant although the price will increase if the height of the caissons demands special production plants. The consequence of this simplification is that the optimization calculations will show that it is more economical even in very deep water to have the caissons placed on a thin bedding layer rather than on a high rubble foundation. In order to avoid such bias the maximum draught of the caissons is set to 24 m in the simulations.

Table 6.2. Average built-in bulk unit prices in Euro/m^3 (app. 2007).

Structure part	Europe	Japan
Caisson	90	150
Armour layers	150	235
Foundation core	25	37
Armour blocks in front for repair	150	200
Mound behind for repair	30	50

The ratio of unit prices in Japan and Europe is approximately 1.6.

6.3 Stability calculations

Wave loads on caissons are determined by the formula by Goda (2000). It is assumed that large impulsive forces are avoided by imposing the conditions that the sea bed slope is gentler than 1:50, and $d/h \geq 0.6$, see Fig. 6.1.

Deterministic design

The caisson width *B* in the deterministic design is determined by applying the design wave height $H_{design} = 1.8 \cdot H_{so}^{TL}$ for non-depth limited conditions. $H_{so}^{T_L}$ is the deep water significant wave height corresponding to return period T_L, i.e. the service life time of the structure. As wave length is applied the one corresponding to local water depth *h* given a deep water wave steepness of s_o=0.04. For depth limited conditions is used max. H_{design}=0.8 *h*.

For the sliding failure mode a safety factor of 1.2 is applied. A safety factor of 2.5 on tilting around the heel of the caisson was implied as well, but was never critical.

The average normal stress σ over the effective foundation width *b*, see Fig. 6.2, was calculated in order to get a simple measure for the foundation loading.

Reliability calculations

In the probabilistic calculation of the performances of the deterministic designs, the actual time series of Rayleigh distributed wave heights obtained from sample simulations in accordance with the long-term statistics, see PIANC (1992) are used, including uncertainties on the distribution parameters. In order to avoid unrealistic wave heights, double truncated Weibull distributions are used (Tae-Min Kim, 2004). The number of waves in each storm is set to 1,000.

A limit for the maximum wave height of 0.8 times the local water depth *h* is used.

Wave loads were determined from the Goda formula without safety factor, corrected for bias and including uncertainty by introducing truncated Normal-distributed factors on the horizontal wave loads and the vertical uplift loads.

The friction factor *f* is modelled by a double truncated normal distribution with mean value $\mu_f = 0.6, \quad \sigma_f / \mu_f = 0.1,$ and cut-off limits 0.7 < *f* < 1.4.

In accordance with OCDI (2002), the following factors in the Goda formula for the reduction of the wave loads in case of repair with armour blocks in front of the caisson is used:

$$\lambda_1 = \lambda_3 = \begin{cases} 1.0 & \text{for } H_{max}/h < 0.3 \\ 1.2 - 0.67\, H_{max}/h & \text{for } 0.3 \leq H_{max}/h < 0.6 \\ 0.8 & \text{for } H_{max}/h \geq 0.6 \end{cases} \qquad \lambda_2 = 0$$

The resistance to sliding R_m provided by the mound behind the caisson is calculated in accordance with OCDI (2002) and with mound dimensions as shown in Fig. 6.4.

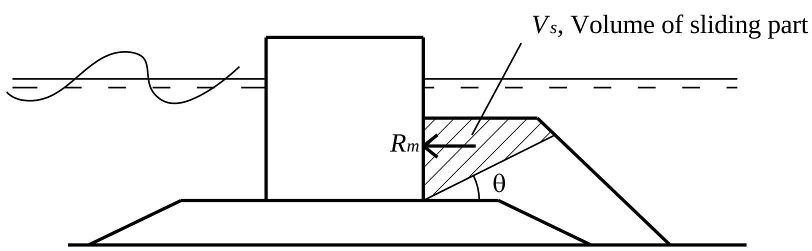

Fig. 6.4. Illustration of resistance of mound to sliding.

For the strength of the quarry rock rubble foundation are used the friction angled 37°, 40° and 45°. For the sand seabed are used the friction angles 30° and 35°. These friction angles are the effective friction angles, i.e. they include the effect of the dilation angles of the materials. The uncertainty on the friction angles is modelled as a Normal distribution with a coefficient of variation of 10%.

The equations related to the slip failures are given in Sorensen and Burcharth (2000).

The sliding distance SD of the caisson should preferably be determined from the dynamic equation of motion assuming a model for the time history of the loading by each wave. In order to save computation time diagrams like those shown in Fig. 6.5 are used. The ordinate is the ratio of the actual horizontal wave force F_H of a single wave to the wave force $F_{H,\text{limit}}$ which just causes the caisson to slide.

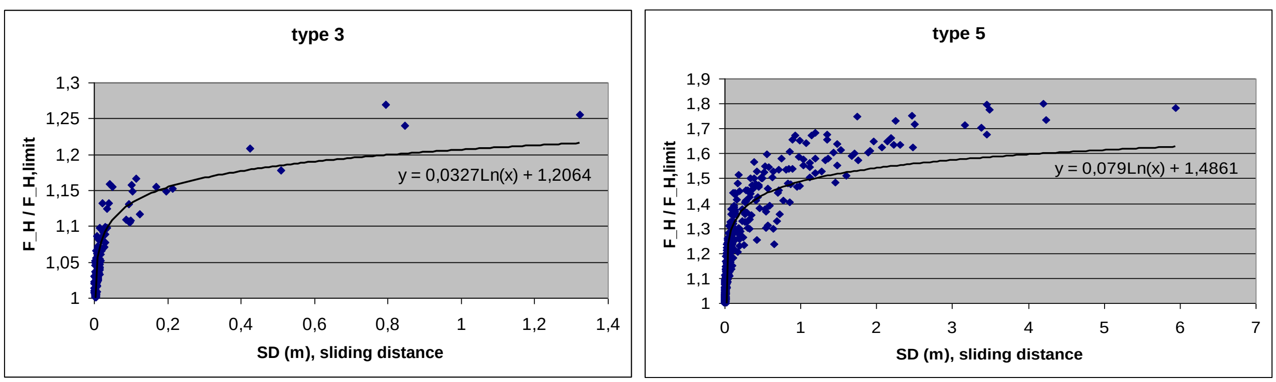

Fig. 6.5. Diagrams for the estimation of caisson sliding distance. Data by Tae-Min Kim (2005)

The graphs fitted to the data points are deliberately chosen to be on the safe side for larger values of $F_H/F_{H,\text{limit}}$ because the dimensions of the caissons and the wave conditions applied in the simulations deviate somewhat form those studied by Tae-Min Kim. A sensitivity analyses has shown that if a graph following more closely the larger data points for Type 5 caisson is used in the simulations then the probability of occurrence of the limit state sliding distances will be approximately halved. The slip failure probabilities and the minimum lifetime costs are not significantly changed.

6.4 Overview of case studies, case study data and identified optimum safety levels

6.4.1 Caissons on hard seabed

Table 6.3 gives an overview of the studied cases. A deep water wave steepness of 0.04 and an interest rate of 5% p.a. are used in all cases. No downtime costs are included.

Table 6.3. Case studies. Caissons on hard bottom. Structure lifetime T_L = 100 years

Case	Water depth, h (m)	Wave climate		Rubble friction angle φ (degrees)	Dimensions of berms and armour (Fig. 14.1), (m)				Sliding distance eq. No. Fig.14.4	RLS repair
		Location	$H_s^{100\ y}$		b_f	t_f	b_r	t_r		
F1-a- 37	15	Follonica	5.64	37	8.00	1.50	10.00	1.50	3	Armour blocks front
F1-b- 37	-	-	-	-	-	-	-	-	-	Mound behind
F1-b-40	-	-	-	40	-	-	-	-	-	-
F1-b-45	-	-	-	45	-	-	-	-	-	-
B1-a-37	25	Bilbao	8.76	37	10.00	2.00	12.00	1.5	5	Armour blocks front
B1-b-37	-	-	-	-	-	-	-	-	-	Mound behind
B2-b-37	-	.	-	-	-	-	24.00	-	-	-
B1-b-40	-	-	-	40	-	-	12.00	-	-	-
B1-b-45	-	-	-	45	-	-	-	-	-	-
S1-b-37	40	Sines	13.2	37	12.00	3.00	14.00	2.00	5	-
S2-b-37	-	-	-	-	-	-	28.00	-	-	-
S1-b-40	-	-	-	40	-	-	14.00	-	-	-
S2-b-40	-	-	-	-	-	-	28.00	-	-	-
S1-b-45	-	-	-	45	-	-	14.00	-	-	-
S2-b-45	-	-	-	-	-	-	28.00	-	-	-
FD-b-40	-	Follonica	5.64	40	8.00	1.50	14.00	1.50	5	-

Tables 6.4 – 6.7 and Figs. 6.6 – 6.9 show the results of Cases F, i.e. caissons in 15 m water depth exposed to Follonica waves.

Table 6.4. Case F1-a-37. Outer caisson breakwater in 15 m water depth. RLS repair with block in front

: F1-a-37	Structure lifetime T_L = 100 years, Water depth h = 15 m, Wave steepness s_o = 0.04, Rear berm width 10m
Seabed : Hard	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L}$ = 5.64 m, $H_S^{T_L}/h$ = 0.38 , Freeboard $h_c = 0.6H_S^{T_L}$ = 3.38 m
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 37°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m²)	Sliding SLS	RLS	ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
10.5	9.0	1000	6.56	20.9	12	258	0.027	0.023	0.009	0.080	64157	68620
11.5	10.0	1000	6.56	19.9	11	290	0.014	0.011	0.003	0.050	61701	64581
12.5	11.0	100	5.64	17.1	10	317	0.035	0.025	0.006	0.057	54787	58479
13.5	12.0	50	5.36	16.4	9	339	0.039	0.031	0.020	0.025	52876	55344
14.5	13.0	25	5.07	15.9	9	360	0.041	0.030	0.020	0.010	51104	52311

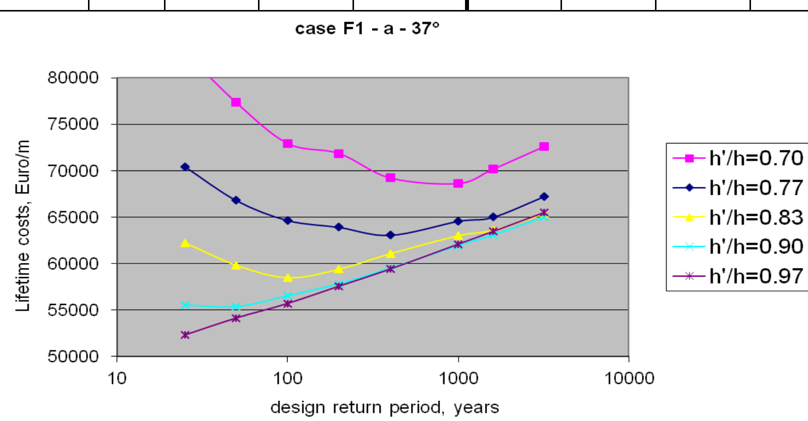

Fig. 6.6. Case F1-a-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.5 and Fig.6.7 show Case F1-b-37 which only differs from Case F1-a-37 by repair with mound behind instead of blocks in front of caisson. Very small differences are seen between the optimum safety levels and lifetime costs in the two cases.

Table6.5. Case F1-b-37. Optimum safety levels for outer breakwater in 15 m water depth. RLS repair with mound behind caisson.

Case: **F1-b-37**	Structure lifetime T_L = 100 years, Water depth h = 15 m, Wave steepness s_o = 0.04, Rear berm width = 10 m
Seabed : Hard	Waves: Follonica , $H_S^{T_L}$ = 5.64 m, $H_S^{T_L} / h$ = 0.38, Freeboard $h_C = 0.6 H_S^{T_L}$ = 3.38 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6, Rubble foundation friction angle φ = 37°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)					(€/m)	(€/m)
10.5	9.0	1000	6.56	20.9	12	258	0.027	0.023	0.013	0.077	64157	68022
11.5	10.0	400	6.20	18.7	10	293	0.038	0.030	0.022	0.076	58993	62697
12.5	11.0	100	5.64	17.1	10	317	0.035	0.025	0.015	0.052	54787	57878
13.5	12.0	50	5.36	16.4	9	339	0.039	0.031	0.024	0.022	52876	54685
14.5	13.0	25	5.07	15.9	9	360	0.041	0.030	0.022	0.010	51104	51926

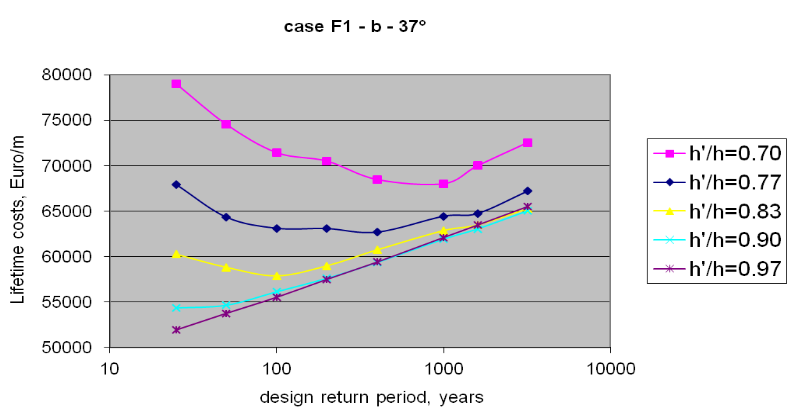

Fig.6.7. Case F1-b-37. Dependence of lifetime costs on relative height of caisson rubble foundation and on wave return period applied in deterministic design.

Table 6.6. Case F1-b-40. Optimum safety levels for outer breakwater in 15 m water depth. RLS repair with mound behind caisson.

Case: **F1-b-40**	Structure lifetime T_L = 100 years, Water depth h = 15 m, Wave steepness s_o = 0.04, Rear berm width = 10 m
Seabed : Hard	Waves: Follonica , $H_S^{T_L}$ = 5.64 m, $H_S^{T_L} / h$ = 0.38, Freeboard $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6, Rubble foundation friction angle φ = 40°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
10.5	9.0	400	6.20	18.9	10	274	0.059	0.043	0.028	0.078	59596	63601
11.5	10.0	50	5.36	16.2	9	300	0.085	0.070	0.053	0.104	52967	58813
12.5	11.0	25	5.07	15.5	9	322	0.086	0.075	0.060	0.059	50759	55194
13.5	12.0	25	5.07	15.6	9	341	0.044	0.037	0.024	0.014	50815	51875
14.5	13.0	25	5.07	15.9	9	360	0.030	0.025	0.019	0.003	51104	51774

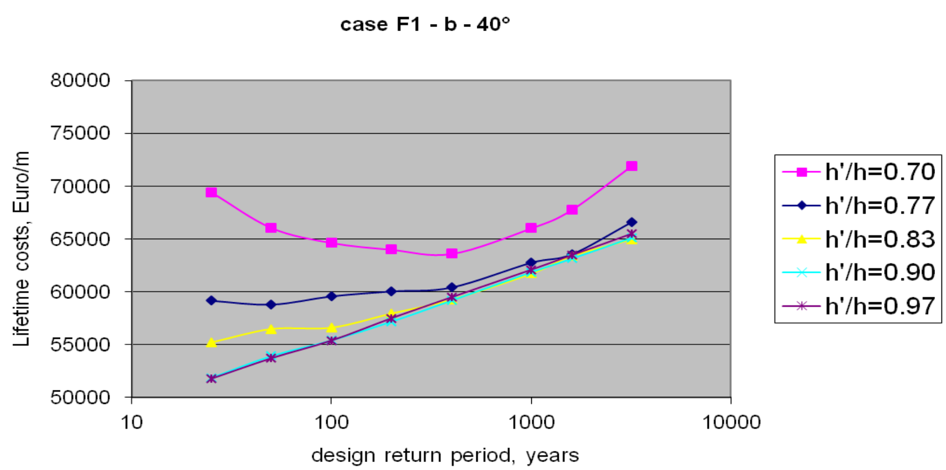

Fig.6.8. Case F1-b-40. Dependence of lifetime costs on relative height of caisson rubble foundation and on wave return period applied in deterministic design.

Table 6.7. Case F1-b-45. Optimum safety levels for outer breakwater in 15 m water depth. RLS repair with mound behind caisson.

Case: **F1-b-45**	Structure lifetime T_L = 100 years, Water depth h = 15 m, Wave steepness s_o = 0.04 Rear berm width = 10 m
Seabed : Hard	Waves: Follonica , $H_S^{T_L}$ = 5.64 m, $H_S^{T_L} / h$ = 0.38 Freeboard $h_C = 0.6 H_S^{T_L}$ = 3.38 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 45°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure	Construction	Lifetime
							SLS	RLS	ULS			
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)					(€/m)	(€/m)
10.5	9.0	100	5.64	17.1	10	279	0.109	0.096	0.068	0.060	55496	60187
11.5	10.0	50	5.36	16.2	9	300	0.094	0.088	0.062	0.029	52967	55464
12.5	11.0	25	5.07	15.5	9	322	0.094	0.076	0.066	0.010	50759	53137
13.5	12.0	25	5.07	15.6	9	341	0.058	0.050	0.030	0.001	50815	51325
14.5	13.0	25	5.07	15.9	9	360	0.048	0.033	0.024	0.000	51104	51698

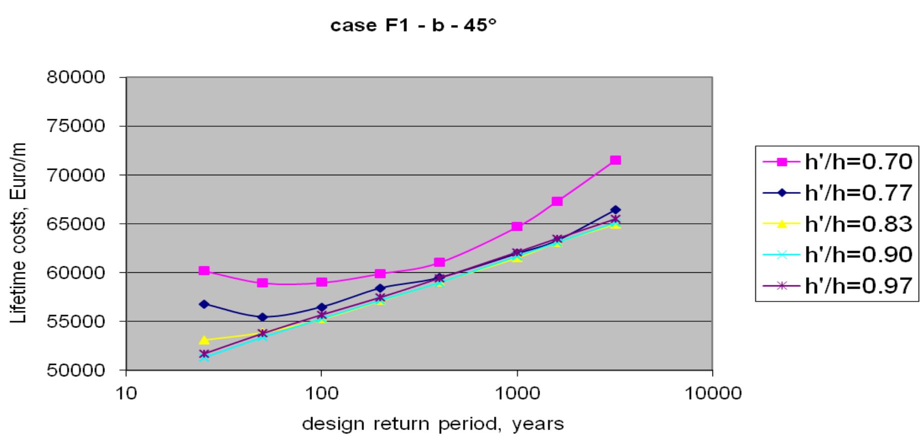

Fig. 6.9. Case F1-b-45. Dependence of lifetime costs on relative height of caisson rubble foundation and on wave return period applied in deterministic design

Tables 6.8 – 6.12 and Figs.6.10 – 6.14 show the results of Cases B, i.e. caissons in 25 m water depth exposed to Bilbao waves.

Table 6.8. Case B1-a-37. Optimum safety levels for outer breakwater in 25 m water depth. RLS repair with blocks in front of caisson.

Case: B1-a-37	Structure lifetime T_L = 100 years, Water depth h = 25 m, Wave steepness s_o = 0.04 , Rear berm width 12 m
Seabed : Hard	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.76 m $H_S^{T_L}/h = 0.35$ Freeboard $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Rubble foundation friction angle φ = 37°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Average normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construc-tion (€/m)	Life-time (€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.015	0.005	0.001	0.190	149001	173740
18.0	16.0	1600	9.97	30.0	17	456	0.016	0.009	0.003	0.158	144655	166367
20.0	18.0	400	9.38	28.2	16	499	0.013	0.007	0.003	0.077	138140	149395
22.0	20.0	50	8.43	25.9	15	546	0.012	0.007	0.002	0.054	128790	135381
24.0	22.0	25	8.09	25.4	15	587	0.009	0.004	0.002	0.011	127059	128261

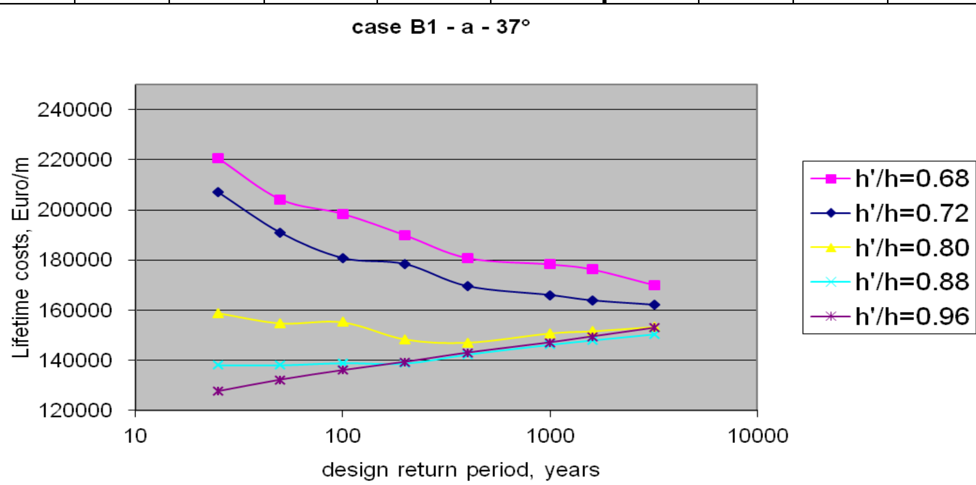

Fig. 6.10. Case B1-a-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.9 and Fig. 6.11 show Case B1-b-37 which differ from Case B1-a-37 by repair with mound behind caisson. No significant differences in optimum safety levels are seen between the two cases.

Table 6.9. Case B1-b-37. Optimum safety levels for outer breakwater in 25 m water depth. RLS repair with mound behind caisson.

Case: B1-b-37	Structure lifetime T_L = 100 years, Water depth h = 25 m, Wave steepness s_o = 0.04 , Rear berm width 12 m
Seabed : Hard	Waves: Bilbao , $H_S^{T_L}$ = 8.76 m; $H_S^{T_L}/h = 0.35$; Freeboard $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6; Rubble foundation friction angle φ = 37°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Average normal stress, σ (KN/m^2)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construc-tion (€/m)	Life-time (€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.015	0.010	0.005	0.159	149001	169363
18.0	16.0	3200	10.25	30.9	17	453	0.010	0.010	0.001	0.109	148065	161851
20.0	18.0	400	9.38	28.2	16	499	0.008	0.006	0.002	0.070	138140	146823
22.0	20.0	200	9.08	27.7	16	540	0.009	0.002	0.000	0.026	136177	138643
24.0	22.0	25	8.09	25.4	15	587	0.003	0.001	0.000	0.004	127059	127594

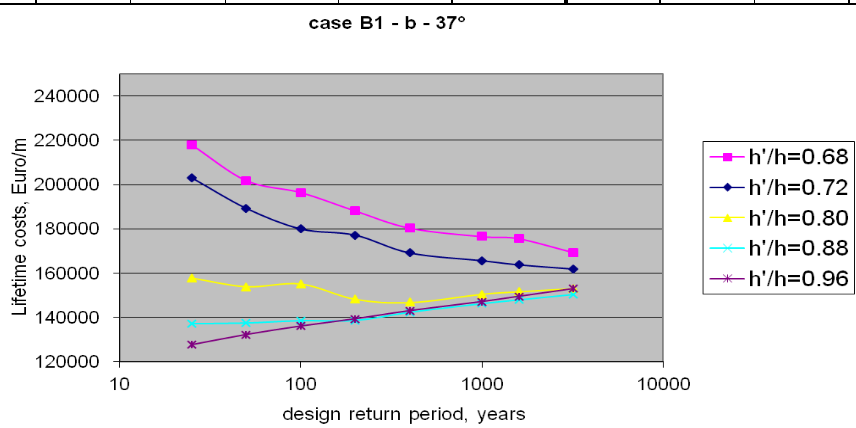

Fig. 6.11. Case B1-b-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.10. Case B2-b-37. Optimum safety levels for outer breakwater in 25 m water depth. RLS repair with mound behind caisson. Wide rear berm.

Case: B2-b-37	Structure lifetime T_L = 100 years, Water depth h = 25 m, Wave steepness s_o = 0.04 , Rear berm width 24 m
Seabed : Hard	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.76 m $H_S^{T_L}/h = 0.35$ Freeboard $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Rubble foundation friction angle φ = 37°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Average normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
17.0	15.0	1600	9.97	30.1	17	437	0.018	0.009	0.003	0.116	154271	169642
18.0	16.0	1600	9.97	30.0	17	456	0.021	0.011	0.004	0.085	153403	162911
20.0	18.0	400	9.38	28.2	16	499	0.012	0.007	0.001	0.037	146000	150852
22.0	20.0	50	8.43	25.9	15	546	0.011	0.006	0.002	0.028	135762	138485
24.0	22.0	25	8.09	25.4	15	587	0.010	0.008	0.004	0.004	133143	133693

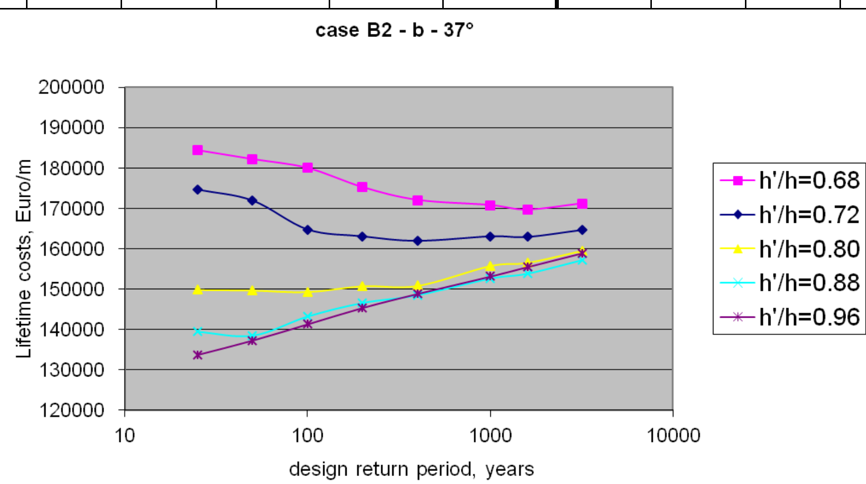

Fig. 6.12. Case B2-b-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.11. Case B1-b-40. Optimum safety levels for outer breakwater in 25 m water depth. RLS repair with mound behind caisson.

Case: B1-b-40	Structure lifetime T_L = 100 years, Water depth h = 25 m, Wave steepness s_o = 0.04 , Rear berm width 12 m
Seabed : Hard	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.76 m $H_S^{T_L}/h = 0.35$ Freeboard $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Rubble foundation friction angle φ = 40°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Average normal stress, σ	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction	Life-time
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m^2)					(€/m)	(€/m)
17.0	15.0	1600	9.97	30.1	17	437	0.023	0.017	0.003	0.124	145079	159294
18.0	16.0	400	9.38	28.1	16	461	0.028	0.018	0.008	0.123	137765	151626
20.0	18.0	50	8.43	25.5	15	507	0.020	0.014	0.005	0.081	127410	138014
22.0	20.0	50	8.43	25.9	15	546	0.012	0.007	0.004	0.022	128790	132228
24.0	22.0	25	8.09	25.4	15	587	0.007	0.002	0.001	0.005	127059	127667

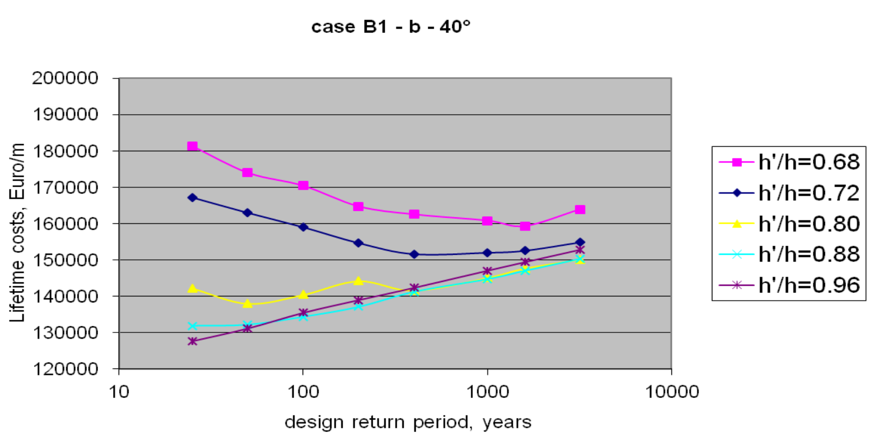

Fig. 6.13. Case B1-b-40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.12. Case B1-b-45. Optimum safety levels for outer breakwater in 25 m water depth. RLS repair with mound behind caisson.

Case: B1-b-40	Structure lifetime T_L = 100 years, Water depth h = 25 m, Wave steepness s_o = 0.04 , Rear berm width 12 m
Seabed : Hard	Waves: Bilbao , $H_S^{T_L}$ = 8.76 m $H_S^{T_L}/h = 0.35$ Freeboard $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 45°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Average normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construc-tion (€/m)	Life-time (€/m)
17.0	15.0	400	9.38	28.2	16	442	0.027	0.017	0.012	0.061	138112	144799
18.0	16.0	200	9.08	27.2	15	464	0.032	0.020	0.008	0.049	134265	140245
20.0	18.0	25	8.09	24.6	14	510	0.033	0.022	0.006	0.038	123629	128447
22.0	20.0	25	8.09	25.0	14	549	0.014	0.009	0.003	0.008	124902	126306
24.0	22.0	25	8.09	25.4	15	587	0.009	0.004	0.002	0.000	127059	127109

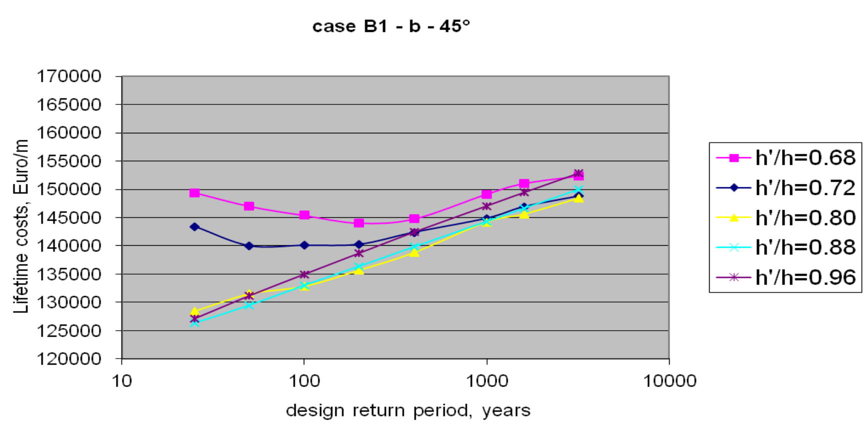

Fig. 6.14. Case B1-b-45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Tables 6.13 – 6.18 and Figs. 6.15 – 6.20 show the results of Cases S, i.e. caissons in 40 m water depth exposed to Sines waves.

Table 6.13. Case S1-b-37. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson.

Case: **S1-b-37**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 14 m
Seabed : Hard	
Unit prices: Japanese	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L} / h$ = 0.33 Freeboard $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 37°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m^2)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
18.0	15.0	1600	15.01	68.6	48	393	0.036	0.026	0.018	0.335	448794	513994
20.0	17.0	1600	15.01	58.8	39	452	0.044	0.030	0.011	0.362	404009	482652
22.0	19.0	3200	15.40	55.3	35	506	0.034	0.022	0.011	0.285	388009	463226
24.0	21.0	3200	15.40	50.0	29	583	0.039	0.030	0.011	0.313	361314	450331

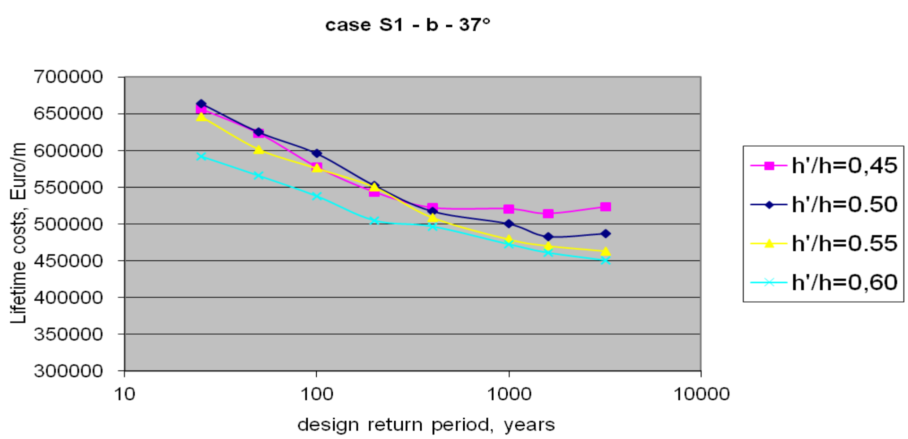

Fig. 6.15. Case S1-b-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.14. Case S2-b-37. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson. Wide rear berm

Case: **S1-b-37**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 28 m
Seabed : Hard	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L} / h$ = 0.33 Freeboard $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 37°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m^2)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
18.0	15.0	1600	15.01	68.6	48	393	0.046	0.034	0.025	0.260	470060	513246
20.0	17.0	3200	15.40	62.7	42	440	0.022	0.014	0.005	0.193	443525	477888
22.0	19.0	3200	15.40	55.3	35	506	0.025	0.015	0.008	0.202	407203	453028
24.0	21.0	3200	15.40	50.0	29	583	0.022	0.014	0.006	0.181	379472	420691

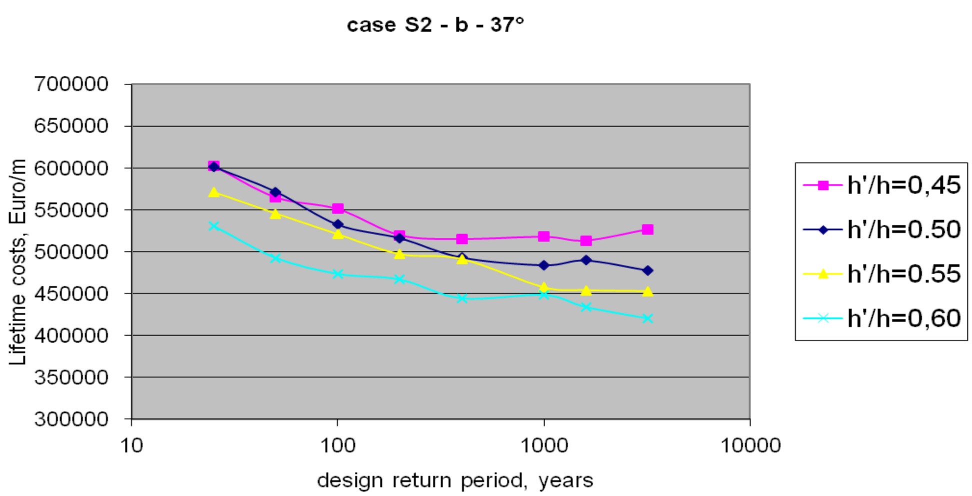

Fig. 6.16. Case S2-b-37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.15. Case S1-b-40. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson.

Case: **S1-b-40**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 14 m
Seabed : Hard	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L}/h$ = 0.33 Freeboard $h_C = 0.6H_S^{T_L}$ = 7.92 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 40°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure	Construction	Lifetime
							SLS	RLS	ULS			
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)					(€/m)	(€/m)
18.0	15.0	1600	15.01	68.6	48	393	0.040	0.026	0.016	0.208	448794	488598
20.0	17.0	1600	15.01	58.8	39	452	0.035	0.023	0.015	0.199	404009	447205
22.0	19.0	1600	15.01	52.0	32	522	0.040	0.027	0.016	0.225	371290	424103
24.0	21.0	1600	15.01	47.2	27	604	0.025	0.016	0.003	0.219	346387	403843

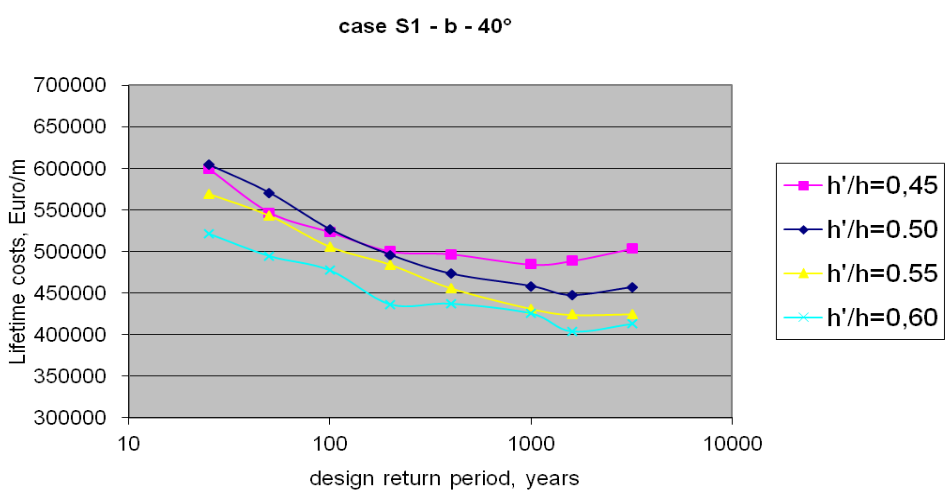

Fig. 6.17. Case S1-b-40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.16. Case S2-b-40. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson. Wide rear berm.

Case: **S1-b-40**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 28 m
Seabed : Hard	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L}/h$ = 0.33 Freeboard $h_c = 0.6H_S^{T_L}$ = 7.92 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 40°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure	Construction	Lifetime
							SLS	RLS	ULS			
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)					(€/m)	(€/m)
18.0	15.0	400	14.16	59.1	40	415	0.069	0.069	0.048	0.286	425125	477292
20.0	17.0	1000	14.73	56.1	37	461	0.044	0.038	0.026	0.179	411308	451985
22.0	19.0	1600	15.01	52.0	32	522	0.022	0.016	0.009	0.156	390484	420564
24.0	21.0	1000	14.73	45.3	25	621	0.047	0.031	0.022	0.192	354459	398792

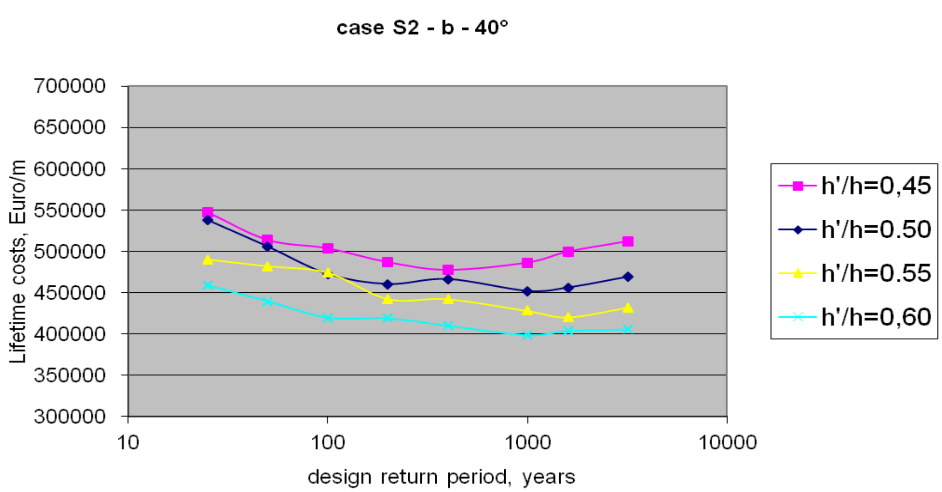

Fig. 6.18. Case S2-b-40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.17. Case S1-b-45. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson.

Case: **S1-b-45**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 14 m
Seabed : Hard	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L} / h$ = 0.33 Freeboard $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 45°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
18.0	15.0	400	14.16	59.1	40	415	0.089	0.067	0.042	0.203	403859	443319
20.0	17.0	1600	15.01	58.8	39	452	0.042	0.026	0.018	0.085	404009	419581
22.0	19.0	400	14.16	45.6	27	562	0.069	0.055	0.035	0.207	338244	392261
		3200	15.40	55.3	35	506	0.014	0.006	0.003	0.061	388009	397198
24.0	21.0	1600	15.01	47.2	27	604	0.026	0.020	0.009	0.101	346387	367763
		3200	15.40	50.0	29	583	0.024	0.015	0.007	0.063	361314	376938

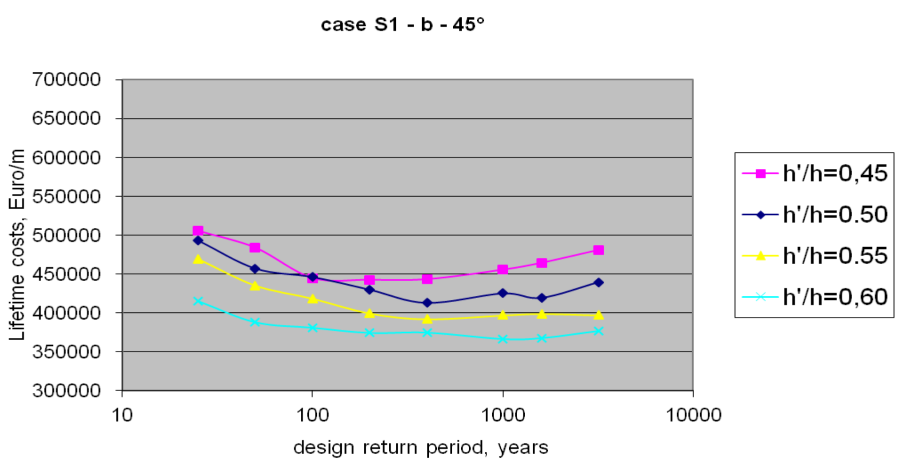

Fig. 6.19. Case S1-b-45. Dependence of lifetime costs on relative height of caisson on rubble mound foundation

Table 6.18. Case S2-b-45. Optimum safety levels for outer breakwaters in 40 m water depth. RLS repair with mound behind caisson. Wide rear berm.

Case: S1-b-45	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 28 m
Seabed : Hard	
Unit prices: Japanese	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L}/h$ = 0.33 Freeboard $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Rubble foundation friction angle φ = 45°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
18.0	15.0	100	13.23	49.8	32	445	0.187	0.153	0.113	0.241	381542	436617
20.0	17.0	400	14.2	51.1	32	482	0.094	0.073	0.050	0.121	386398	415878
22.0	19.0	400	14.2	45.6	27	562	0.090	0.064	0.032	0.114	357438	382161
24.0	21.0	1000	14.73	45.3	25	621	0.045	0.033	0.014	0.071	354459	368811

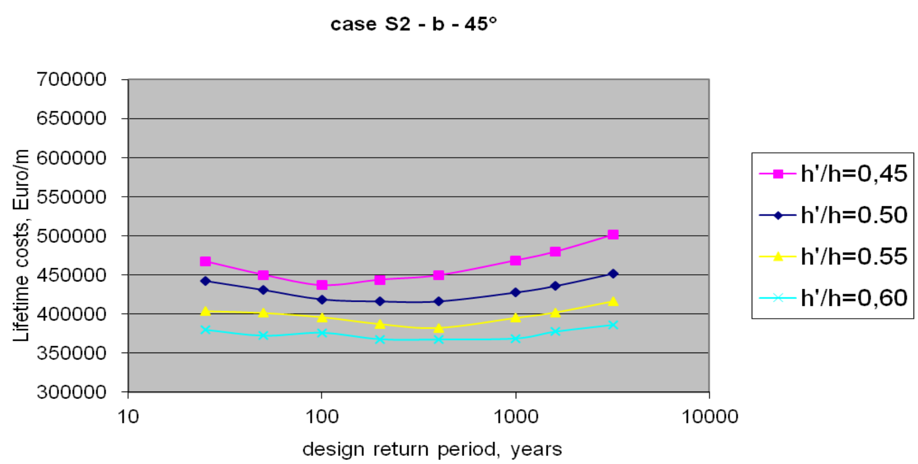

Fig. 6.20. Case S2-b-45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.19 and Fig. 6.21 show the optimum safety levels for caisson breakwaters in 40 m water depth exposed to relatively small waves.

Table 6.19. Case FD-b-40. Optimum safety levels for outer breakwater in 40 m water depth. RLS repair with mound behind caisson.

Case: FD-**b-40**	Structure lifetime T_L = 100 years, Water depth h = 40 m, Wave steepness s_o = 0.04 Rear berm width = 14 m
Seabed : Hard	Waves: Follonica , $H_S^{T_L}$ = 5.64 m, $H_S^{T_L} / h$ = 0.14 Freeboard $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Rubble foundation friction angle φ = 40°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs				Costs	
Caisson draft, h' (m)	Toe level, d below SWL (-m)	Return period (years)	H_s (m)	Caisson width, B (m)	Effective width, b (m)	Aver. normal stress, σ (KN/m²)	Sliding SLS	Sliding RLS	Sliding ULS	Slip failure	Construction (€/m)	Lifetime (€/m)
16.5	15.0	3200	7.01	20.6	12	389	0.000	0.000	0.000	0.166	167493	197393
18.5	17.0	3200	7.01	20.8	12	427	0.001	0.000	0.000	0.122	163989	183355
20.5	19.0	3200	7.01	21.1	12	465	0.000	0.000	0.000	0.053	161438	171242
22.5	21.0	1000	6.56	20.4	12	506	0.000	0.000	0.000	0.045	154630	162742
24.5	23.0	1000	6.56	20.8	12	544	0.000	0.000	0.000	0.022	153448	157602

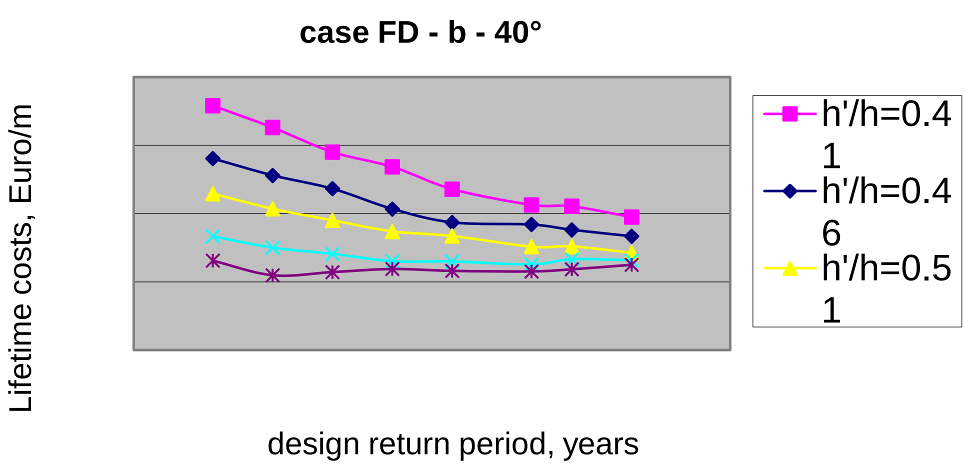

Fig. 6.21. Case FD-b-40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

6.4.2 Caissons on sand seabed

The simulations are performed with sand friction angles φ = 30° and 35°. The friction angles of the rubble foundation are φ = 37°, 40° and 45°. A deep water wave steepness of 0.04 and an interest rate of 5% p.a. are used in all cases. No downtime costs are included. Table 6.20 gives and overview of the cases.

Table 6.20. Case studies. Caissons on sand sea beds. Structure lifetime T_L = 100 years

Case	Water depth, h (m)	Wave climate		Friction angle φ (degrees)		Dimensions of berms and armour (Fig. B4.1), (m)				Sliding eq. No. cf. Fig. 14.4	RLS repair
		Location	$H_s^{100\,y}$	Rubble	Sand	b_f	t_f	b_r	t_r		
F1-s30-r37	15	Follonica	5.64	37	30	8.00	1.50	10.00	1.50	3	Mound behind
F1-s35-r37	-	-	-	-	35	-	-	-	-	-	-
F1-s35-r40	-	-	-	40	-	-	-	-	-	-	-
F1-s35-r45	-	-	-	45	-	-	-	-	-	-	-
F2-s35-r45				-	-	-	-	20.00	-	3	-
B1-s30-r37	25	Bilbao	8.76	37	30	10.00	2.00	12.00	1.5	5	-
B1-s35-r37	-	-	-	-	35	-	-	-	-	-	-
B2-s35-r37	-	.	-	-	-	-	-	24.00	-	-	-
B1-s35-r40	-	-	-	40	-	-	-	12.00	-	-	-
B1-s35-r45	-	-	-	45	-	-	-	-	-	-	-
B2-s35-r45	-	-	-	-	-	-	-	24	-	-	-
S1-s35-r45	40	Sines	13.2	45	35	12.00	3.00	14.00	2.00	5	-
S2-s35-r45	-	-	-	-	-	-	-	28.00	-	-	-

Tables 6.21 – 6.26 and Figs. 6.22 – 6.27 show the results of Cases F, i.e. caissons in 15 m water depth exposed to Follonica waves.

Table 6.21. Case F1-s30-r37. Optimum safety level for outer caisson breakwater in 15 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: **F1-s30-r37°**	Structure lifetime T_L = 100 years. Water depth h = 15 m. Wave steepness s_o = 0.04. Rear berm width = 10 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L} = 5.64$ m $\quad H_S^{T_L} / h = 0.38 \quad h_c = 0.6 H_S^{T_L} = 3.38$ m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 37° Friction angle sand φ = 30°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
10.5	9.0	1000	6.56	20.9	12	258	0.027	0.023	0.013	0.077	0.007	0.079	64157	68151
11.5	10.0	400	6.20	18.7	10	293	0.038	0.030	0.022	0.076	0.044	0.083	58993	62921
12.5	11.0	100	5.64	17.1	10	317	0.035	0.025	0.015	0.052	0.132	0.132	54787	62129
13.5	12.0	400	6.20	18.8	11	331	0.015	0.009	0.004	0.008	0.122	0.122	58949	65803
14.5	13.0	400	6.20	18.9	11	351	0.004	0.003	0.001	0.000	0.162	0.162	59396	70451

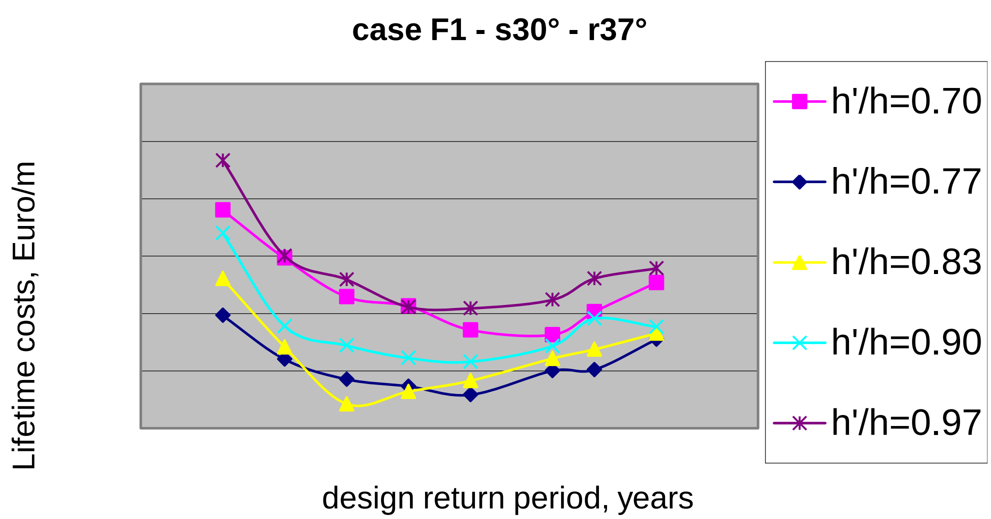

Fig. 6.22. Case F1-s30-r37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.22. Case F1-s35-r37. Optimum safety level for outer caisson breakwater in 15 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: **F1-s35-r37°**	Structure lifetime T_L = 100 years. Water depth h = 15 m. Wave steepness s_o = 0.04. Rear berm width = 10 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L}$ = 5.64 m $H_S^{T_L}/h = 0.38$ $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 37° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m^2)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
10.5	9.0	1000	6.56	20.9	12	258	0.028	0.021	0.011	0.082	0.003	0.083	64157	68185
11.5	10.0	400	6.20	18.7	10	293	0.036	0.025	0.014	0.065	0.013	0.066	58993	62280
12.5	11.0	200	5.92	17.9	10	315	0.029	0.022	0.013	0.044	0.065	0.071	56791	59960
13.5	12.0	400	6.20	18.8	11	331	0.010	0.008	0.003	0.002	0.036	0.036	58949	61067
14.5	13.0	200	5.92	18.2	10	353	0.009	0.008	0.003	0.000	0.070	0.070	57354	60737

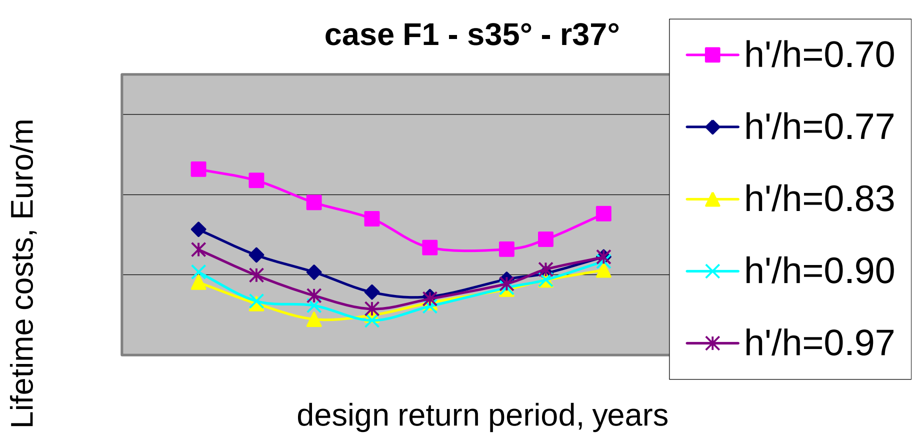

Fig. 6.23. Case F1-s35-r37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.23. Case F1-s35-r40. Optimum safety level for outer caisson breakwater in 15 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: **F1-s35-r40°**	Structure lifetime T_L = 100 years. Water depth h = 15 m. Wave steepness s_o = 0.04. Rear berm width = 10 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L}$ = 5.64 m $H_S^{T_L}/h = 0.38$ $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 40° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
10.5	9.0	400	6.20	18.9	10	274	0.046	0.039	0.029	0.074	0.002	0.075	59596	63454
11.5	10.0	100	5.64	17.0	10	298	0.064	0.056	0.048	0.064	0.027	0.070	54965	59359
12.5	11.0	100	5.64	17.1	10	317	0.035	0.025	0.015	0.022	0.039	0.040	54787	57341
13.5	12.0	100	5.64	17.2	10	336	0.023	0.019	0.015	0.004	0.053	0.053	54911	57936
14.5	13.0	100	5.64	17.4	10	356	0.017	0.016	0.009	0.000	0.073	0.073	55297	59422

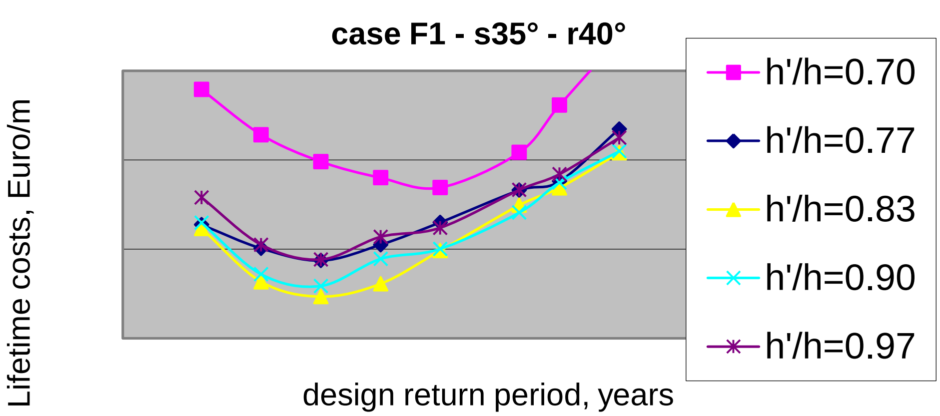

Fig. 6.24. Case F1-s35-r40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.24. Case F1-s35-r45. Optimum safety level for outer caisson breakwater in 15 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: **F1-s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 15 m. Wave steepness s_o = 0.04. Rear berm width = 10 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L}$ = 5.64 m $H_S^{T_L}/h = 0.38$ $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
10.5	9.0	200	5.92	18.0	10	276	0.072	0.059	0.047	0.043	0.002	0.047	57531	60185
11.5	10.0	100	5.64	17.0	10	298	0.052	0.038	0.032	0.017	0.007	0.019	54965	56735
12.5	11.0	100	5.64	17.1	10	317	0.035	0.027	0.020	0.004	0.012	0.012	54787	55812
13.5	12.0	50	5.36	16.4	9	339	0.036	0.028	0.019	0.002	0.041	0.041	52876	55584
14.5	13.0	100	5.64	17.4	10	356	0.008	0.008	0.005	0.000	0.043	0.043	55297	57267

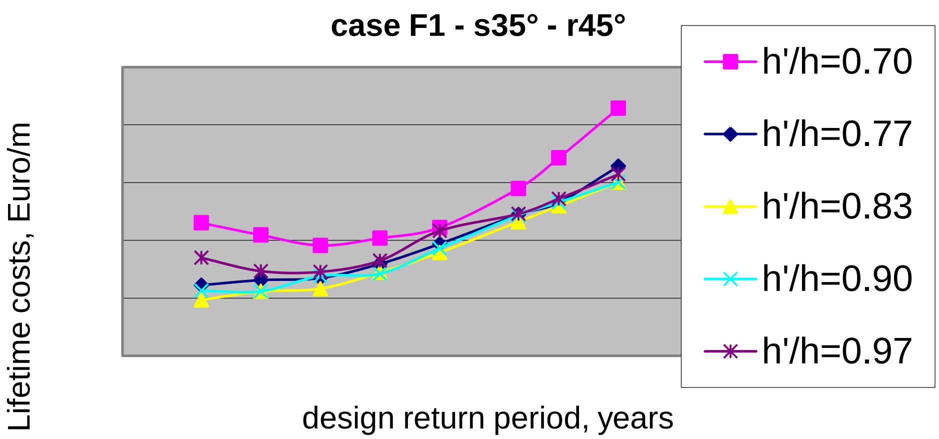

Fig. 6.25. Case F1-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.25. Case F2-s35-r45. Optimum safety level for outer caisson breakwater in 15 m water depth. 100 years lifetime. RLS repair with mound behind caisson. Wide rear berm.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: **F2-s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 15 m. Wave steepness s_o = 0.04. Rear berm width = 20 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Follonica , $H_S^{T_L}$ = 5.64 m $H_S^{T_L}/h = 0.38$ $h_c = 0.6 H_S^{T_L}$ = 3.38 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
10.5	9.0	25	5.07	15.3	9	284	0.197	0.170	0.142	0.040	0.000	0.040	56663	61219
11.5	10.0	25	5.07	15.3	9	303	0.131	0.115	0.085	0.016	0.000	0.016	55785	58688
12.5	11.0	25	5.07	15.5	9	322	0.090	0.080	0.065	0.005	0.005	0.009	55209	57437
13.5	12.0	50	5.36	16.4	9	339	0.039	0.031	0.024	0.000	0.014	0.014	56956	57914
14.5	13.0	100	5.64	17.4	10	356	0.017	0.016	0.009	0.000	0.020	0.020	59007	60113

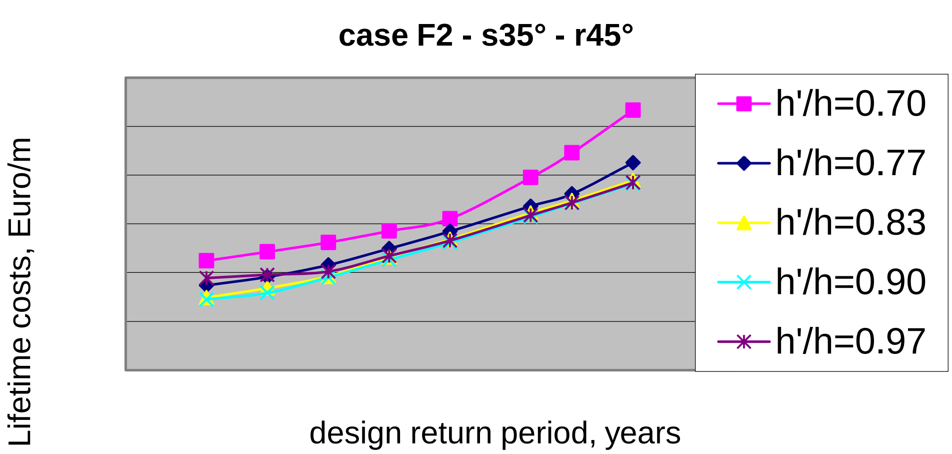

Fig. 6.26. Case F2-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Tables 6.26 – 6.31 and Figs. 6.27 – 6.32 show the results of Cases B, i.e. caissons in 25 m water depth exposed to Bilbao waves.

Table 6.26. Case B1-s30-r37. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B1-**s30-r37°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 12 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.75 m $H_S^{T_L}/h = 0.35$ $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 37° Friction angle sand φ = 30°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.015	0.010	0.005	0.159	0.041	0.161	149001	169944
18.0	16.0	3200	10.25	30.9	17	453	0.010	0.010	0.001	0.109	0.088	0.138	148065	165370
20.0	18.0	3200	10.25	30.8	17	492	0.005	0.004	0.002	0.036	0.213	0.213	148351	188688
22.0	20.0	200	9.08	27.7	16	540	0.009	0.002	0.000	0.026	0.351	0.351	136177	212630
24.0	22.0	400	9.38	28.9	17	576	0.002	0.001	0.000	0.003	0.402	0.402	142365	240882

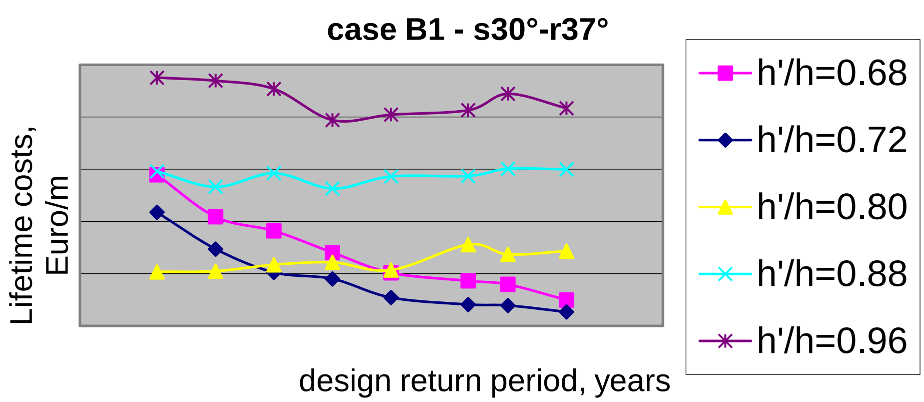

Fig.6.27. Case B1-s30-r37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.27. Case B1-s35-r37. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B1-**s35-r37°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 12 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.75 m $H_S^{T_L}/h = 0.35$ $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 37° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.015	0.011	0.003	0.189	0.020	0.189	149001	173726
18.0	16.0	1600	9.97	30.0	17	456	0.012	0.009	0.004	0.151	0.050	0.154	144655	163210
20.0	18.0	400	9.38	28.2	16	499	0.009	0.006	0.001	0.079	0.137	0.142	138140	159807
22.0	20.0	1600	9.97	30.2	17	533	0.003	0.002	0.000	0.006	0.176	0.176	146643	179973
24.0	22.0	1000	9.77	29.9	17	573	0.000	0.000	0.000	0.000	0.166	0.166	147063	178394

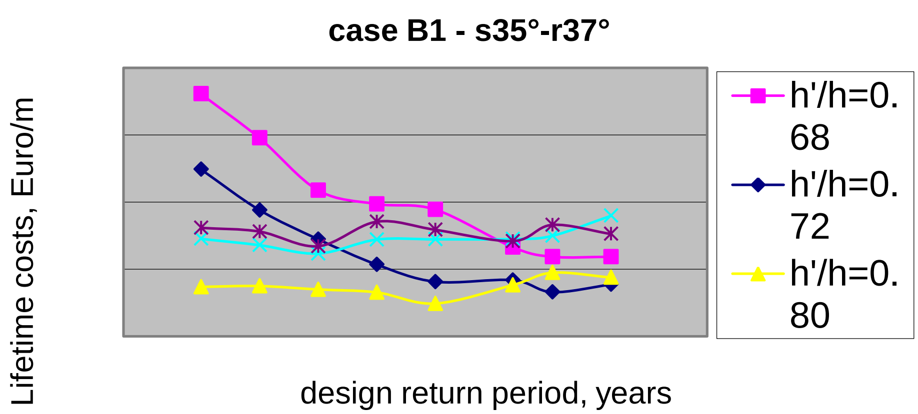

Fig. 6.28. Case B1-s35-r37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.28. Case B2-s35-r37. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson. Wide rear berm.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B2-**s35-r37°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 24 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.75 m $H_S^{T_L}/h = 0.35$ $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 37° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.022	0.012	0.004	0.101	0.000	0.101	158193	171192
18.0	16.0	1600	9.97	30.0	17	456	0.021	0.011	0.004	0.085	0.000	0.085	153403	162911
20.0	18.0	400	9.38	28.2	16	499	0.012	0.007	0.001	0.037	0.006	0.037	146000	150852
22.0	20.0	100	8.76	26.8	15	543	0.010	0.003	0.001	0.026	0.049	0.052	139509	146401
24.0	22.0	200	9.08	28,1	16	579	0.004	0.001	0.000	0.003	0.048	0.048	144799	151800

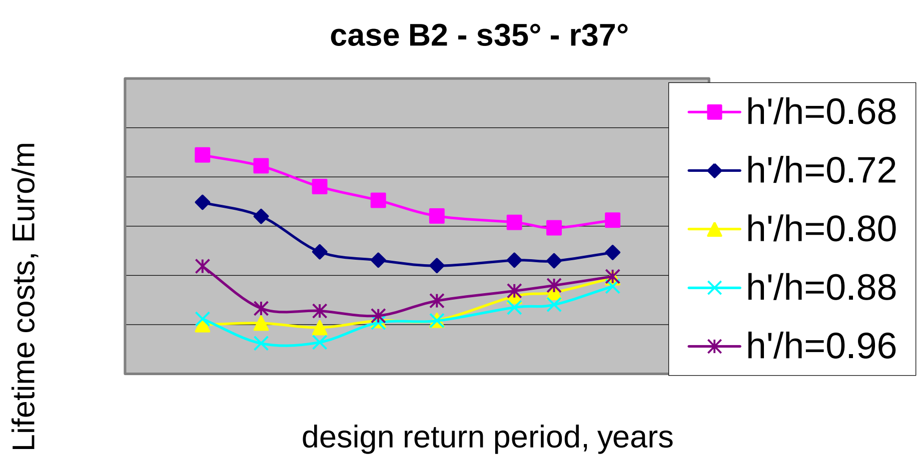

Fig.6.29. Case B2-s35-r37. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.29. Case B1-s35-r40. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B1-**s35-r40°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 12 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.75 m $H_S^{T_L}/h = 0.35$ $h_c = 0.6H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 40° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	3200	10.25	31.2	17	432	0.015	0.010	0.005	0.091	0.011	0.092	149001	159714
18.0	16.0	1600	9.97	30.0	17	456	0.007	0.004	0.001	0.079	0.022	0.080	144655	153715
20.0	18.0	50	8.43	25.5	15	507	0.023	0.011	0.004	0.089	0.104	0.112	127410	144188
22.0	20.0	50	8.43	25.9	15	546	0.014	0.009	0.004	0.022	0.139	0.139	128790	153247
24.0	22.0	200	9.08	28,1	16	579	0.001	0.001	0.000	0.001	0.103	0.103	138715	157727

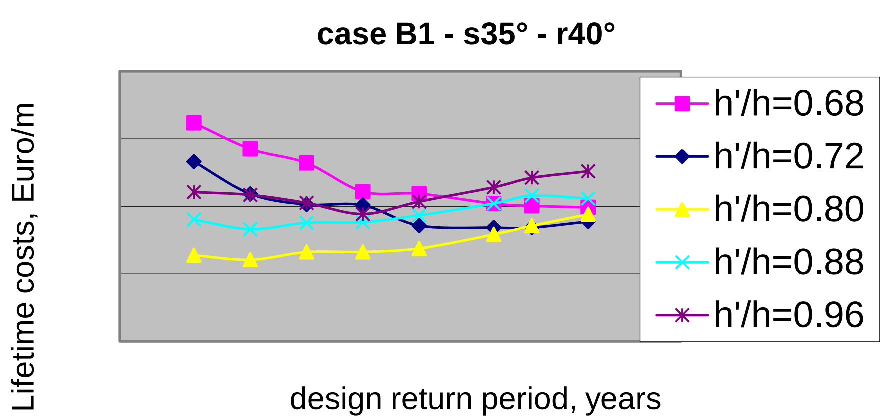

Fig.6.30. Case B1-s35-r40. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.30. Case B1-s35-r45. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B1-**s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 12 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_s^{T_L}$ = 8.75 m $\quad H_s^{T_L}/h = 0.35 \quad h_c = 0.6 H_s^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m^2)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	400	9.38	28.2	16	442	0.039	0.025	0.011	0.073	0.001	0.073	138112	146206
18.0	16.0	200	9.08	27.2	15	464	0.026	0.015	0.004	0.044	0.005	0.044	134265	139385
20.0	18.0	50	8.43	25.5	15	507	0.018	0.015	0.006	0.028	0.036	0.039	127410	131662
22.0	20.0	50	8.43	25.9	15	546	0.017	0.007	0.002	0.007	0.056	0.056	128790	137870
24.0	22.0	100	8.76	27.2	16	581	0.004	0.003	0.001	0.000	0.065	0.065	134965	144975

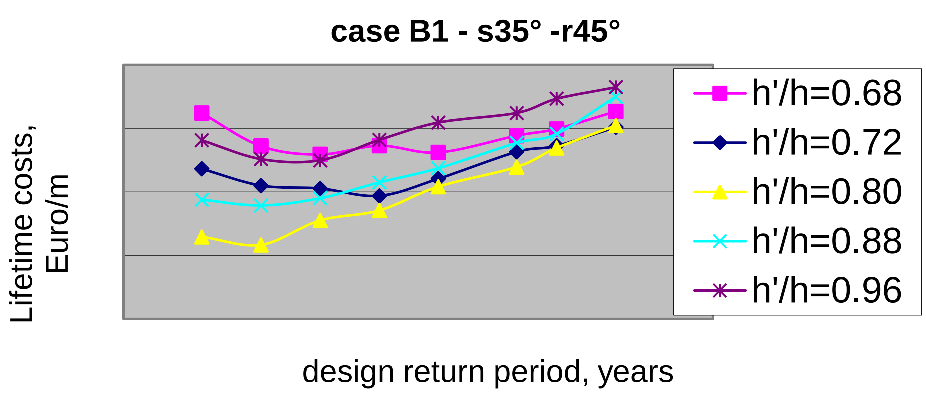

Fig.6.31. Case B1-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.31. Case B2-s35-r45. Optimum safety level for outer caisson breakwater in 25 m water depth. 100 years lifetime. RLS repair with mound behind caisson. Wide rear berm.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: B2-**s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 25 m. Wave steepness s_o = 0.04. Rear berm width = 24 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Bilbao , $H_S^{T_L}$ = 8.75 m $H_S^{T_L}/h = 0.35$ $h_c = 0.6 H_S^{T_L}$ = 5.26 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
17.0	15.0	50	8.43	25.3	14	450	0.080	0.061	0.026	0.055	0.000	0.055	136580	143811
18.0	16.0	100	8.76	26.3	15	466	0.046	0.025	0.018	0.048	0.000	0.048	135837	142785
20.0	18.0	100	8.76	26.5	15	505	0.040	0.027	0.010	0.013	0.004	0.014	131489	133646
22.0	20.0	50	8.43	25.9	15	546	0.019	0.015	0.004	0.002	0.035	0.035	131874	137024
24.0	22.0	25	8.09	25.4	15	587	0.003	0.001	0.000	0.000	0.063	0.063	133143	140599

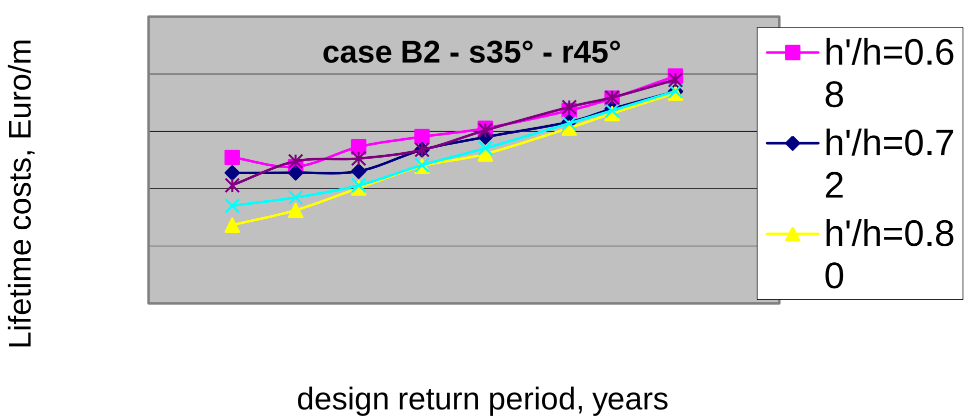

Fig. 6.32. Case B2-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation

Tables 6.32- 6.33 and Figs. 6.33-6.34 show the results of Case S, i.e. caissons in 40 m water depth exposed to Sines waves.

Table 6.32. Case S1-s35-r45. Optimum safety level for outer caisson breakwater in 40 m water depth. 100 years lifetime. RLS repair with mound behind caisson.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: S1-**s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 40 m. Wave steepness s_o = 0.04. Rear berm width = 14 m.
Seabed : Sand	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L}/h = 0.33$ $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Unit prices: Japanese	
Interest rate: , 5 % p.a.	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°
Downtime costs: 0 €	

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
18.0	15.0	400	14.16	59.1	40	415	0.076	0.055	0.030	0.196	0.000	0.196	403859	456613
20.0	17.0	1000	14.73	56.1	37	461	0.046	0.033	0.017	0.135	0.000	0.135	391078	428098
22.0	19.0	1600	15.01	52.0	32	522	0.030	0.017	0.009	0.103	0.00	0.103	371290	396347
24.0	21.0	1600	15.01	47.2	27	604	0.029	0.021	0.008	0.113	0.000	0.113	346387	379174

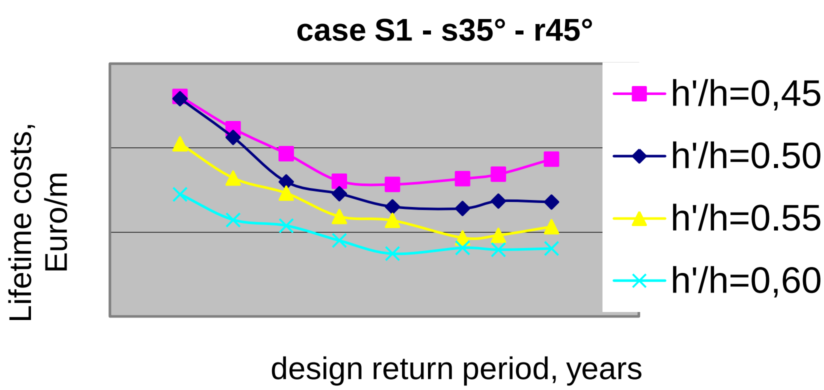

Fig. 6.33. Case S1-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

Table 6.33. Case S2-s35-r45. Optimum safety level for outer caisson breakwater in 40 m water depth. 100 years lifetime. RLS repair with mound behind caisson. Wide rear berm.

Caisson breakwater optimization	**Initial repair with mound behind**
Case: S2-**s35-r45°**	Structure lifetime T_L = 100 years. Water depth h = 40 m. Wave steepness s_o = 0.04. Rear berm width = 28 m.
Seabed : Sand	
Unit prices: Japanese	Waves: Sines , $H_S^{T_L}$ = 13.2 m $H_S^{T_L}/h = 0.33$ $h_c = 0.6 H_S^{T_L}$ = 7.92 m
Interest rate: , 5 % p.a.	
Downtime costs: 0 €	Friction factor f = 0.6 Friction angle rubble φ = 45° Friction angle sand φ = 35°

Data for deterministic design $S_{sliding}$ = 1.2, $S_{tilting}$ = 2.5							Failure probability in structure lifetime corresponding to minimum lifetime costs						Costs	
Caisson draft, h'	Toe level, d below SWL	Return period	H_s	Caisson width, B	Effective width, b	Aver. normal stress, σ	Sliding			Slip failure			Construction	Lifetime
(m)	(-m)	(years)	(m)	(m)	(m)	(KN/m²)	SLS	RLS	ULS	Rubble	Sand	Total	(€/m)	(€/m)
18.0	15.0	100	13.23	49.8	32	445	0.199	0.155	0.111	0.256	0.000	0.256	381542	434737
20.0	17.0	200	13.71	47.3	29	501	0.115	0.093	0.060	0.190	0.000	0.190	367769	410532
22.0	19.0	400	14.16	45.6	27	562	0.081	0.052	0.030	0.132	0.00	0.132	357438	386213
24.0	21.0	200	13.71	41.2	23	640	0.072	0.050	0.027	0.117	0.000	0.117	332461	359984

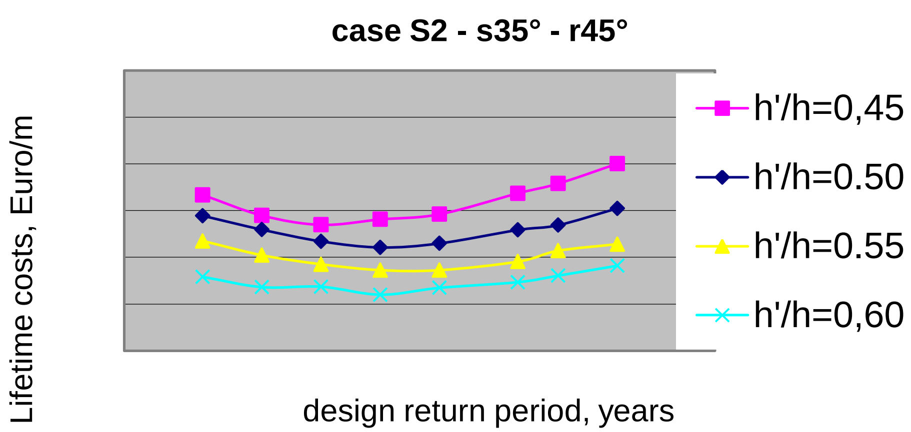

Fig. 6.34. Case S2-s35-r45. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on wave return period applied in deterministic design.

6.5 Conclusions on optimum safety levels

6.5.1 Main results related to individual cases of caissons on hard seabed

Water depths, h = app. 15 m. H_s^{100y} *= 5 – 6 m. Interests rate 5% p.a.*

- Caisson width B = app. 1.05 h = 16 m.
- No high rubble foundation, only a rock material bedding layer, h'/h = 0.96
- Friction angle of bedding layer material not critical if not less than 37^o
- Optimum safety level to be used in design as shown in Table 6.34

Table 6.34 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in bedding layer
0.04	0.03	0.02	≤ 0.01

The corresponding wave return period applied in the deterministic design is 25 years. The reason for this small return period is the large conventional safety factor of S = 1.2 applied for sliding in deterministic design.

Water depths, h = app. 25 m. H_s^{100y} *= 8 – 9 m. Interests rate 5% p.a.*

- Caisson width B = app. 1.02 h = 25.5 m
- No high rubble foundation, only a rock material bedding layer, h'/h = 0.96
- Friction angle of bedding layer material not critical if not less than 37^o
- Optimum safety level to be used in design as shown in Table 6.35

Table 6.35 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in bedding layer
0.01	0.01	0.005	< 0.005

The corresponding wave return period applied in the deterministic design is 25 years. The reason for this small return period is the large conventional safety factor of S = 1.2 applied for sliding in deterministic design.

Large water depth, h = 40 m. Very large waves, H_s^{100y} *= 12 – 13 m. Interests rate 5% p.a.*

- The maximum height of the caisson under water is set to 24 m for construction reasons.

- The rubble foundation should be as low as possible, i.e. h'/h = 0.60, because slip failure in the mound is the most critical failure mode. The friction angle of the rubble should be as large as possible and no less than 45°. A wide rear berm should be arranged in order to ensure maximum resistance to slip failure in the rubble foundation.

- Caisson width B = app. 1.25 h = 50 m

- Optimum safety level to be used in design as shown in Table 6.36

Table 6.36 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in rubble foundation
0.03	0.02	0.01	0.07

The corresponding wave return period applied in the deterministic design is 3200 years. The reason for this large return period is the critical slip failure mode the probability of which reduces with increase in caisson width. Nevertheless, even with a caisson width of 50 m the slip failure probability is high.

It can be concluded that a conventional caisson solution is not feasible in very large water depths with very large design waves.

Large water depth, h = 40 m, small to moderate waves, H_s^{100y} = 5 – 6 m. Interest rate 5%.

- The maximum height of the caisson under water is set to 24 m for construction reasons.

- The rubble foundation should be as low as possible, i.e. h'/h = 0.60, because slip failure in the mound is the most critical failure mode. The friction angle of the rubble should be as large as possible and no less than 40 - 45°.

- Caisson width B = app. 0.53 h = 21 m

- The optimum safety levels to be used in design are given in Table 6.37

Table 6.37 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in rubble foundation
0.001	0.001	0.001	0.03

The corresponding wave return period applied in the deterministic design is 1000 years.

6.5.2 Main results related to individual cases of caissons on sand seabed

Water depth, h = 15 m. H_s^{100y} *= 5 – 6 m. Interest rate 5% p.a.*

- Caisson width B = 1.13 h = 17 m

- The rubble foundation should be medium high, i.e. h'/h = 0.83 and the friction angles of the sand and the rubble must be no less than 35^o and 45^o respectively in order to keep slip failures at a reasonable level.

- The optimum safety levels to be used in design are shown in Table 6.38

Table 6.38 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in sand/rubble foundation
0.035	0.03	0.02	0.01

The corresponding wave return period applied in the deterministic design is 100 years.

Large water depth, h = 25 m. Large waves, H_s^{100y} *= 8 – 9 m. Interest rate 5% p.a.*

- Caisson width, B = 1.06 h = 26.5 m

- The friction angles of sand and rubble must be min. 35^o and 45^o respectively in order to keep the slip failure probability at a reasonable level. Moreover, a wide rear berm must be arranged.

- The height of the mound corresponds to h'/h = 0.80.

- The optimum safety levels to be used in design are shown in Table 6.39

Table 6.39 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in sand/rubble foundation
0.02	0.015	0.01	0.04

The corresponding wave return period applied in the deterministic design is 100 years. An increase from 12 m to 24 m of the width of the rear berm reduces the slip failure probability to 0.01.

Water depth, h = 40 m. H_s^{100y} = 13 – 14 m. Interest rate 5% p.a.

- The maximum height of the caisson is set to 24 m for construction reasons.

- Even with friction angles of 45° for the rubble and 35° for the sand it is not possible to get a reasonably low slip failure probability unless a caisson width exceeding 50 m is used.

- Table 6.40 shows the optimum safety levels for a caisson with width B = 47.2 m. Note that the slip failure probability is unacceptable high.

- It is concluded that conventional caisson solutions are not feasible in very large water depths when exposed to very large waves.

Table 6.40 Optimum probability of occurrence of limit states in 100 years lifetime

SLS	RLS	ULS	Slip failure in rubble foundation
0.029	0.021	0.008	0.113

The corresponding wave return period applied in the deterministic design is 1600 years.

6.5.3 Overall conclusions related to caissons on hard seabed and sand seabed

The costs of repair by placing an armour unit mound in front of the caissons or by placing a rubble mound behind the caissons are almost the same when seen over the lifetime of the structure.

In case of a hard seabed it is beneficial to place the caisson on a relatively thin rock bedding layer. The angle of friction of a thin bedding layer is not so important as long as the friction angle is no less than 37°.

If the water depth is larger than app. 30 m it will be necessary to use a high rubble foundation because of construction difficulties if the caissons have more than 24 m draught. If the rock foundation layer is more than 2 m high the friction angle of the bedding layer should be no less than 45°. The most critical failure mode will then be a slip failure in the foundation mound.

In very large water depths of say 40 m and very large waves of say Hs = 13 – 14 m it is not possible to limit the slip failures to acceptable probabilities of occurrence even if the foundation rubble mound has a friction angle of 45°. So a conventional caisson solution is not feasible under such conditions.

However, if in very deep water the waves are smaller than say H_s = 5 – 6 m then a caisson solution on a high foundation is feasible if the friction angle of the mound is no less than 40°.

In water depths smaller than say 30 m the most economical probabilities of occurrence of the limit states are shown in Table 6.41. Interest rate is 5% p.a. and no downtime costs are included as it is not likely that the breakwater crest level will be significantly lowered even in case of slip failures.

Table 6.41 Optimum probabilities of occurrence of limit states for caissons in outer breakwaters. Quarry rock foundation on hard seabed. 100 years lifetime

Water depth (m)	H_s^{100y} (m)	Seabed	SLS	RSL	ULS	Slip failure	Deterministic design return period (y)
15	5 - 6	Rock and bedding layer	0.04	0.03	0.02	0.01	25
25	8 - 9	-	0.01	0.01	0.003	0.005	25

If in deterministic design instead of a 25 years return period significant wave height a 50 years return period significant wave height is used then the SLS, RLS and ULS probabilities of occurrence will be approximately halved and the slip failure probability reduced to 0.002. At the same time the lifetime costs will increase by approximately 3 - 4 %. As this is a very small increase, the simulations actually confirm that the normally applied wave return period of 50 years is a very reasonable choice.

In case of a weak sand seabed is it necessary to apply a higher rock foundation than a bedding layer in order to distribute the loadings on the sand. The friction angle of the rubble foundation should be no less than 45^o and the friction angel of the sand no less than 35^o.

The optimum probabilities of occurrence of the limit states are given in Table 6.42. Interest rate is 5% p.a. and no downtime costs are included.

Table 6.42 Probability of occurrence of limit states for caissons in outer breakwaters. Quarry rock foundation on sand seabed. 100 years lifetime

Water depth (m)	H_s^{100y} (m)	Seabed	Relative height of foundation h’/h	SLS	RSL	ULS	Slip failure	Deterministic design return period (y)
11	5 - 6	Sand and rock foundation	0.83	0.04	0.03	0.02	0.01	100
25	8 - 9	-	0.80	0.02	0.015	0.01	0.04	50 - 100

The identified optimum deterministic design wave return periods of 50 – 100 years correspond quite well to design praxis.

It is seen from Tables 6.41 and 6.42 that the larger the water depth the smaller the optimum probability of occurrence of the limit states will be.

The annual probabilities of occurrence are obtained by dividing the values given in the tables by 100.

An illustrative interpretation of the probability of occurrences given in the tables would be as follows: The value 0.04 related to SLS given in Table 6.42 implies that within the 100 years lifetime in average one out of 25 caissons (or four out of 100 caissons) will slide the SLS- defined distance of 0.20 m given in Table 6.1.

The identified optimum safety levels given in Tables 6.41 and 6.42 are much higher for both SLS and ULS than the Table 1.1 safety levels given in the Italian Guidelines and the Spanish ROM.

6.6 Partial safety factors

Reference is given to **THE PIANC SAFETY FACTOR SYSTEM FOR BREAKWATERS**. Proc. Coastal Structures `99 (ed. I.J. Losada), A.A. Balkema, Rotterdam, pp.3 -20. Burcharth, H.F. and Sorensen, J.D. (2000).

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