# On Optimum Safety Levels of Breakwaters

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# ON OPTIMUM SAFETY LEVELS OF BREAKWATERS

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**ABSTRACT**

The paper presents results from numerical simulations performed with the objective of identifying optimum design safety levels of conventional rubble mound and caisson breakwaters, corresponding to the lowest costs over the service life of the structures. The work is related to the PIANC Working Group 47 on "Selection of type of breakwater structures". The paper summaries results given in Burcharth and Sorensen (2005) related to outer rubble mound breakwaters but focus on optimum safety levels for outer caisson breakwaters on low and high rubble foundations placed on sea beds strong enough to resist geotechnical slip failures. Optimum safety levels formulated for use both in deterministic and probabilistic design procedures are given. Results obtained so far indicate that the optimum safety levels for caisson breakwaters are much higher than for rubble mound breakwaters.

KEYWORDS: Breakwaters, rubble mound breakwaters, caisson breakwaters, optimum safety levels, probabilistic design.

## 1. INTRODUCTION

### 1.1 Economic optimization

Fig. 1 shows the principle of identifying the most economical design safety level when taking into account construction, repair and downtime costs over the service life of the structure, discounted to present value.

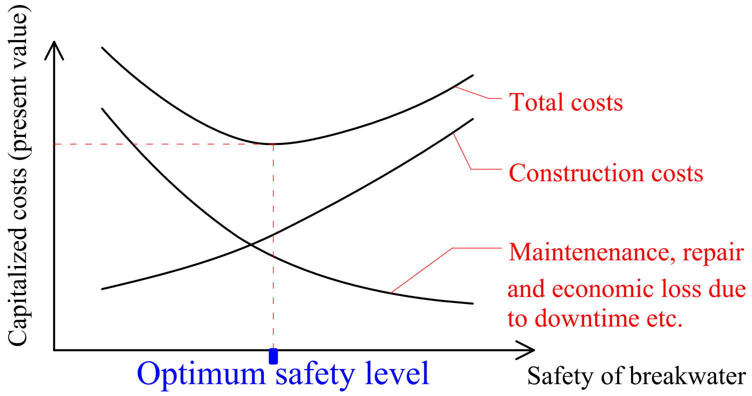



Fig. 1. Illustration of optimum safety level based on economic optimization.

### 1.2. Format for safety implementation

Most national standards and recommendations for design of breakwaters introduce overall safety on loads or resistance related to a specific return period sea state in cases where loadings can be calculated, as for caisson breakwaters. For rubble mound structure where no loadings can be calculated safety is implemented in terms of constraints to damage corresponding to exposure to specific return period sea states. In both cases are the actual safety levels unknown in terms of probability of predefined damage within service life.

The ISO-Standard 2394 (1998) on "Reliability of Structures" prescribes a format for safety implementation where safety-classification is based on the importance of the structure and the consequences of malfunction, and for design both a "Serviceability Limit State" (SLS) and an "Ultimate Limit State" (ULS) must be considered with damage criteria assigned to these limit states. Moreover, uncertainties on all parameters and models must be taken into account. The Spanish recommendations for Maritime Structures, ROM 0.0, Part I (2002), follows this format, however, with what must regarded tentative values of safety levels as they are not based on more systematic investigations.

In the present work is introduced also a "Repairable Limit State" (RLS) defined as the maximum damage level which allows planned maintenance and repair methods to be used.

### 1.3. Functional classification and performance criteria

The following summary of the applied functional classification, assigned performance criteria, procedure in numerical simulations and formulation of total cost function is an extract from Burcharth & Sorensen (2005).

So far only outer breakwaters with no berths just behind or near the breakwater have been analysed. Fig. 2 shows this functional class and the applied tentatively defined limit state performance criteria. $H_{S,T}$ is the transmitted significant wave height corresponding to return period equal to design life time *T*. *D* is the relative number of displaced armour units.

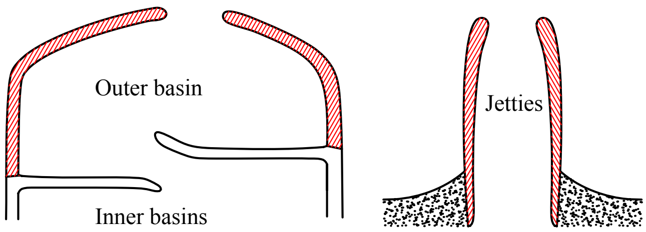
| | | |
|---|---|---|
| | Functional classification<br>Outer breakwaters | Tentative performance criteria |
| |   | Wave transmission<br>SLS: $H_{S,T}$ = 0.5 – 1.8 m<br><br>Damage to main armour<br>SLS: D = 5 %, RLS: D = 15 %<br>ULS: D = 30 %<br><br>Sliding distance of caissons<br>SLS: 0.2 m, ULS: 2 m |

Fig. 2. Functional classification: Outer breakwaters and related limit state performance criteria.

Fig. 3. shows another functional class where moorings are arranged just behind the breakwater and performance criteria therefore are more restrictive. *q* is the average overtopping discharge in $m^3/s$ per metre of breakwater.

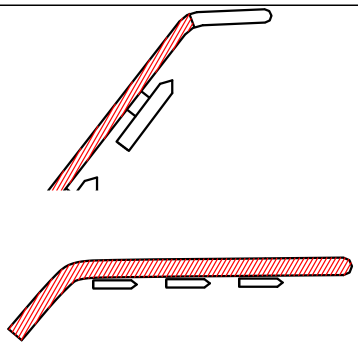
| | | |
|---|---|---|
| | Functional classification<br>Rear side moorings | Tentative performance criteria |
| |  | Wave overtopping<br>SLS: *q* = 10-5 - 10-4 m3/ms<br>ULS: *q* = 10-3 - 10-2 m3/ms<br><br>Damage to main armour<br>SLS: D = 5 %, RLS: D = 15 %<br>ULS: D = 30 %<br><br>Sliding distance of caissons<br>SLS: 0.0 m, ULS: 0.5 m |

Fig. 3. Example of functional class with restrictive limit state performance criteria.

### 1.4. Cost function

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

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

where

| | |
|---|---|
| $T$ | return period used for deterministic design |
| $T_L$ | design life time |
| $C_I(T)$ | initial costs (building costs) with design based on return period *T* |
| $C_{R1}(T)$ | cost of repair for minor damage with design based on return period *T* |
| $P_{R1}(t)$ | probability of minor damage in year *t* with design based on return period *T* |
| $C_{R2}(T)$ | cost of repair for major damage |

| | |
|---|---|
| $P_{R2}(t)$ | probability of major damage in year *t* with design based on return period *T* |
| $C_F(T)$ | cost of failure including downtime costs |
| $P_F(t)$ | probability of failure in year *t* with design based on return period *T* |
| *r* | real rate of interest |

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

The breakwater is designed corresponding to a design wave height with return period *T*. The reliability level corresponding to the optimal return period *T* from (1) is then the optimal reliability level.

**1.5. Wave statistics**

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 Biscaya) and Sines (Atlantic Ocean). Storms are assumed to be modelled by a Poisson process with occurrence rate corresponding to the average number of storms per year.

Characteristics of these wave climates are indicated in Table 1 which provides the deepwater significant wave heights corresponding to 100 years and 400 years return periods. More details are given in PIANC (1992).

Table 1. Example of return period significant wave heights in deep water of applied long-term wave climates.

| **Location** | $H_{s,o}^{100y}$ | $H_{s,o}^{400y}$ |
|---|---|---|
| Follonica | 5.64 m | 6.20 m |
| Bilbao | 8.76 m | 9.38 m |
| Sines | 13.2 m | 14.2 m |

**1.6. Downtime costs**

In case of failure of the breakwater, waves might penetrate into the harbour and cause stop of some port operations, for example loading/unloading of container vessels. The affected parties are the vessel owner/charter, the stevedoring company, the port authority, and the related service industries. For a large container vessel berth out of action the total direct loss could be in the order of 200,000 Euro per day. Assuming 90 days stop in the period of breakwater repair, the total costs would be 18,000,000 Euro. This amount is used in the simulation as a possible upper limit for downtime costs related to a breakwater length of 1km. Downtime costs related to bulk terminals would be significantly lower.

**1.7. Procedure in simulations**

The optimization problem (1) is 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 are determined by conventional deterministic design for a selected range of water depths and long-term wave statistics applying design waves corresponding to different return periods as described above. Damages as they occur are identified and accumulated, and repairs are performed in accordance with defined repair policy. The related costs of repairs are calculated. Failures (large damages), which introduce downtime costs due to stop of port operations are identified and related downtime costs are calculated. Further, the construction cost of each breakwater is calculated. All costs are added to obtain the total lifetime cost. Among each type of structure and environmental conditions is identified the structure with the lowest life time costs, and for this structure is extracted the related probabilities of reaching SLS, RLS and ULS in the structure life time. These values then represent the optimum design safety levels. The simulations comprise the influence on the optimum safety level of interest rate (inflation included), structure service life and downtime costs.

## 2. RUBBLE MOUND BREAKWATERS

### 2.1. Cross Sections

The following is an extract of some of the results given in Burcharth and Sorensen (2005) related to outer rubble mound breakwaters with concrete cube and rock armour with cross sections as shown in Fig. 4. *Dn* = (armour unit volume)$^{1/3.}$ $H_s$ is the significant wave height used in the design

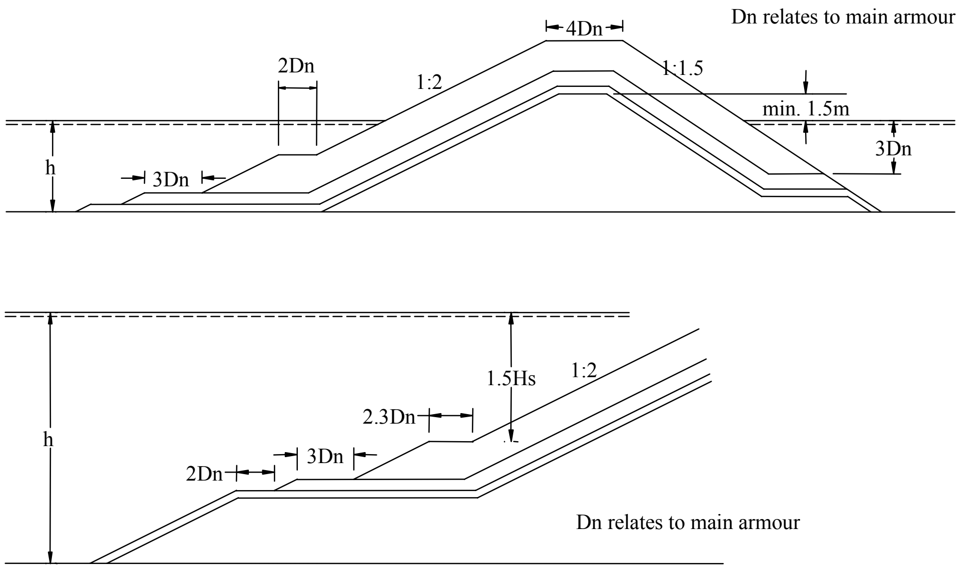



Fig.4. Shallow and deep water cross sections of rubble mound breakwaters used in the simulations.

The crest levels are determined from criteria of maximum transmitted significant wave height of 0.5m by overtopping in sea states with return period equal to structure service life.

### 2.2. Case studies

Table 2 shows the data for the case studies.

Table 2. Data for case studies

| Case | Water depth | Armour mass density | Waves Location Cf. Table 1 | Armour stability formula | Built-in unit prices core/filter 1/filter 2/armour Euro/$m^3$ |
|---|---|---|---|---|---|
| 1.2 | 10 | Rock 2.65 t/$m^3$ | Follonica | Van der Meer (1988) | 10/16/20/40 |
| 1.3 | 15 | Cube 2.40 t/$m^3$ | Bilbao | Van der Meer (1988) Modified to slope 1:2 | 10/16/20/40 |
| 2.3 | 30 | Cube 2.40 t/$m^3$ | Sines | Van der Meer (1988) Modified to slope 1:2 | 5/10/25/35 |

The built-in unit prices are average prices for medium to very large size European projects, collected by the PIANC WG-47 members.

Toe stability is not included as a failure mode in the simulations as small differences in toe berm armour sizes will not influence the results of the optimizations.

### 2.3. Repair policy and costs

The adopted repair policy is given in Table 3. D is the relative number of displaced armour units.

Table 3. Repair policy

| **Damage levels reached** | **D** | **Repair** |
|---|---|---|
| Initial | 2% | No repair |
| Serviceability (SLS) | 5% | Repair of armour |
| Repairable (RLS) | 15% | Repair of armour + filter 1 |
| Failure (ULS) | 30% | Repair of armour + filters 1 and 2 |

For repair the built-in unit prices are increased by 50% compared to prices for initial construction given in Table 2. Moreover, mobilization and demobilization costs are included as 30% of the initial armour layer construction costs.

The downtime costs of 18,000,000 EURO apply when RLS occur.

### 2.4. Damage accumulation model

Each storm is set to 1,000 waves. Damages occur and are accumulated only when the damage levels S=1 and $N_{od}$=0.002 for 1000 waves are exceeded. $S$ and $N_{od}$ are damage parameters used in the Van der Meer armour stability formulae. Damage accumulation takes place only when the next storm has a higher $H_s$-value than the preceding value. The relative decrease in damage with the number of waves inherent in the stability formulae (Van der Meer, 1988a, 1988b) is taken care of by keeping track of the number of waves which contributes to damage.

### 2.5. Example of results

Table 4 and Fig. 5 show the outcome of some of the optimization simulations for cases 1.3 and 2.3.

Table 4. Case 1.3. Optimum safety levels for concrete cube armoured outer breakwater. 50 years service lifetime. 15 m water depth. Damage accumulation included. (Burcharth and Sorensen, 2005).

| Real Interest Rate | Optimum data for deterministic design | | | | Optimum limit state average number of events within service lifetime | | | Construction costs for 1 km length | Total lifetime costs for 1 km length |
|---|---|---|---|---|---|---|---|---|---|
| (%) | Optimized design return period, T (years) | $H_s^T$ (m) | Optimum armour unit mass W (t) | Free-board $R_c$ (m) | SLS | RLS | ULS | (1,000 EURO) | (1,000 EURO) |
| 2 | 400 | 6.20 | 12.5 | 6.3 | 1.11 | 0.008 | 0.001 | 17,494 | 19,268 |
| 5 | 200 | 5.92 | 10.9 | 6.0 | 1.84 | 0.015 | 0.003 | 16,763 | 18,318 |
| 8 | 100 | 5.64 | 9.5 | 5.8 | 2.98 | 0.031 | 0.008 | 16,038 | 17,625 |
| 2 | 400 | 6.20 | 12.5 | 6.3 | 1.11 | 0.008 | 0.002 | 17,494 | 19,391 |
| 5 | 200 | 5.92 | 10.9 | 6.0 | 1.82 | 0.015 | 0.004 | 16,763 | 18,453 |
| 8 | 100 | 5.64 | 9.5 | 5.8 | 2.98 | 0.031 | 0.008 | 16,038 | 17,821 |

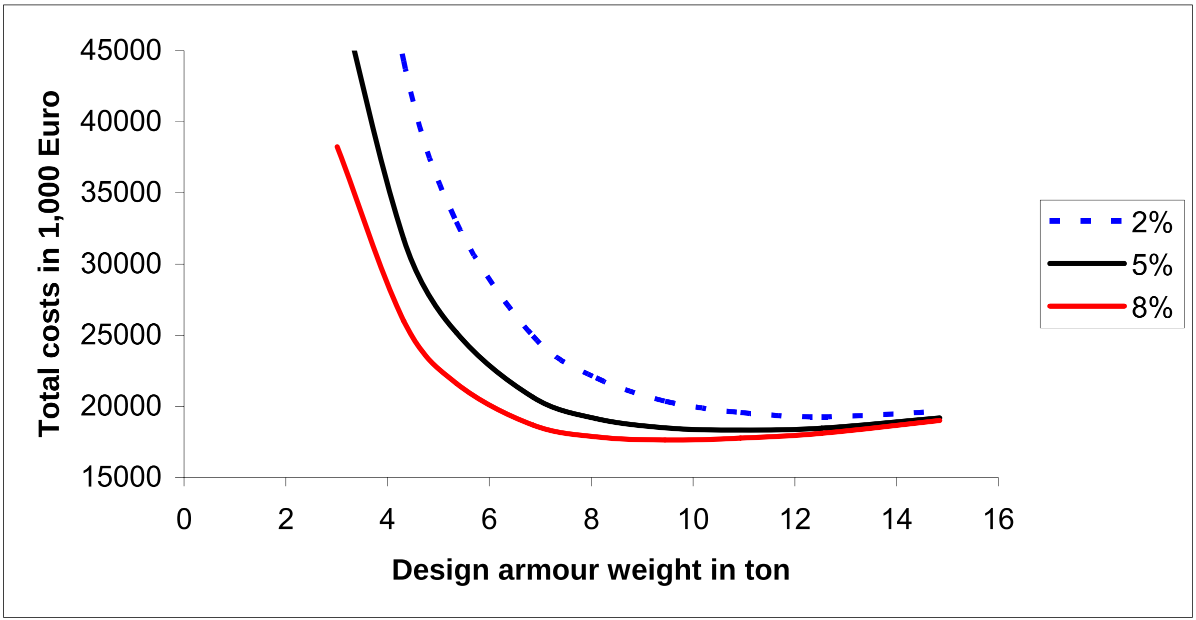



Fig. 5. Case 1.3. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation included. (Burcharth and Sorensen, 2005).

Table 4 and Fig. 5 are valid for simulations with and without the downtime cost as no significant difference was found.

Table 5. Case 2.3. Optimum safety levels for concrete cube armoured outer breakwater. 30 m water depth. 50 years lifetime. Damage accumulation included. Downtime costs 18 million EURO for damage D > 15%. (Burcharth and Sorensen, 2005).

| Lifetime (years) | Real Interest Rate (%) | Optimum data for deterministic design: Optimized design return period, T (years) | $H_s^T$ (m) | Optimum armour unit mass W (t) | Free-board $R_c$ (m) | Optimum limit state average number of events within structure lifetime: SLS | RLS | ULS | Construction costs for 1 km length (1,000 EURO) | Total Lifetime costs for 1 km length (1,000 EURO) |
|---|---|---|---|---|---|---|---|---|---|---|
| | 2 | 1000 | 14.7 | 168 | 14.8 | 1.21 | 0.008 | 0.001 | 76,907 | 86,971 |
| 50 | 5 | 400 | 14.2 | 150 | 14.8 | 1.84 | 0.016 | 0.003 | 73,722 | 81,875 |
| | 8 | 100 | 13.2 | 122 | 14.8 | 3.39 | 0.052 | 0.012 | 68,635 | 78,095 |

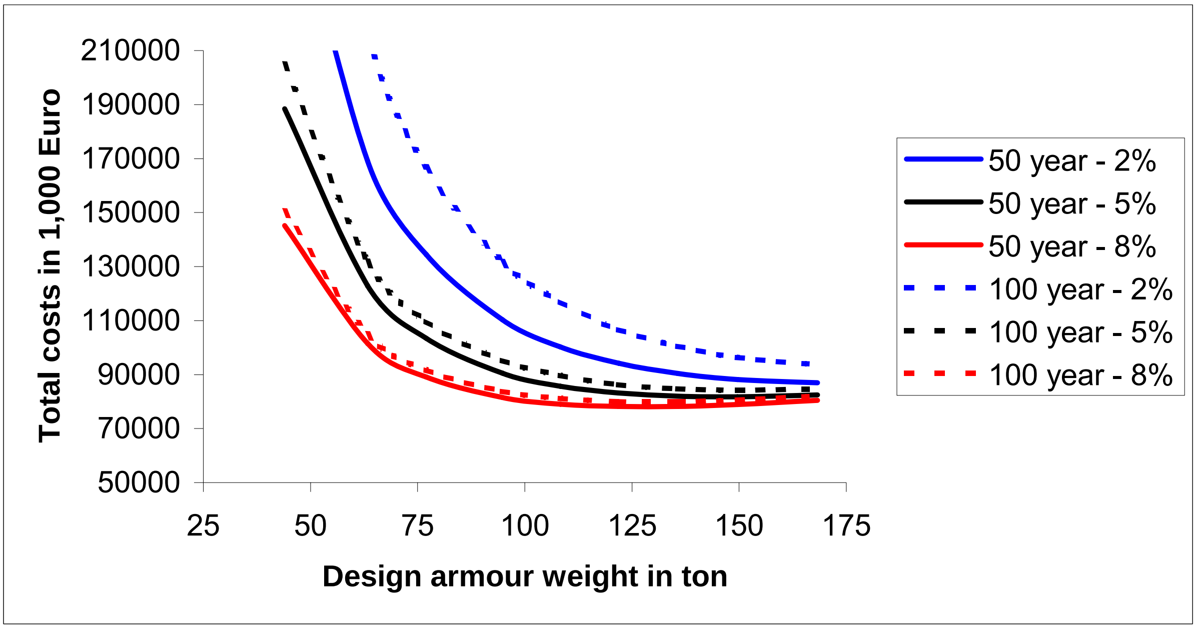



Fig. 6. Case 2.3. Total costs in 50 years lifetime as function of real interest rate and armour unit mass used in deterministic design. Damage accumulation and downtime costs included. (Burcharth and Sorensen, 2005).

**2.6. Conclusions related to outer rubble mound breakwaters**

- All simulations 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 results show that optimum safety levels are higher than the safety levels inherent in conventional deterministic designs, especially in the case of depth limited wave height conditions and/or low interest rates.

- Further, the results show that for the investigated type of breakwater the critical design limit state corresponds to Serviceability Limit State (SLS) defined by moderate damage to the armour layer. Designing for SLS and performing repair when the SLS-damage is reached, imply that the probability of very severe damage or failure is almost negligible, and so will be the related cost of repair and downtime costs. This is typical for structures with ductile damage development.

- The identified optimum safety levels correspond to exceedence of the SLS-moderate damage level in average once to twice within a service life of 50 years, given the yearly interest rates is 2-5 %. For higher interest rates the optimum number of exceedences will increase corresponding to less safe structures.

- The simulations show that for optimum designs the lifetime costs and the optimum safety levels decrease rather significantly with increasing interest rates! Thus it is more economic to design for more frequent repairs in case of high interest rates. This however might be practically and politically unacceptable.

- The ratio of optimum design failure probability to service lifetime is almost constant for each design limit state. This means that if for SLS the optimum number of exceedences 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.

- Downtime costs within realistic ranges seem to have only marginal influence on the optimum safety level.

- Damage accumulation has to be considered in the design of armour layers having a significant influence on the optimum safety level.

- The obtained results indicate that optimum safety levels for rubble mound breakwaters belonging to functional classes with more restrictive performance criteria than outer breakwaters, cf. Fig. 3, will be almost the same as for outer breakwaters. This is because of the marginal influence of downtime costs.

## 3. CAISSON BREAKWATER

### 3.1.Cross sections

Fig. 7 shows the cross sections dealt with in the simulations. The ratio between the draft of the caisson, h' and the water depth *h* is varied in order to identify the most economical height of the rubble foundation. 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.

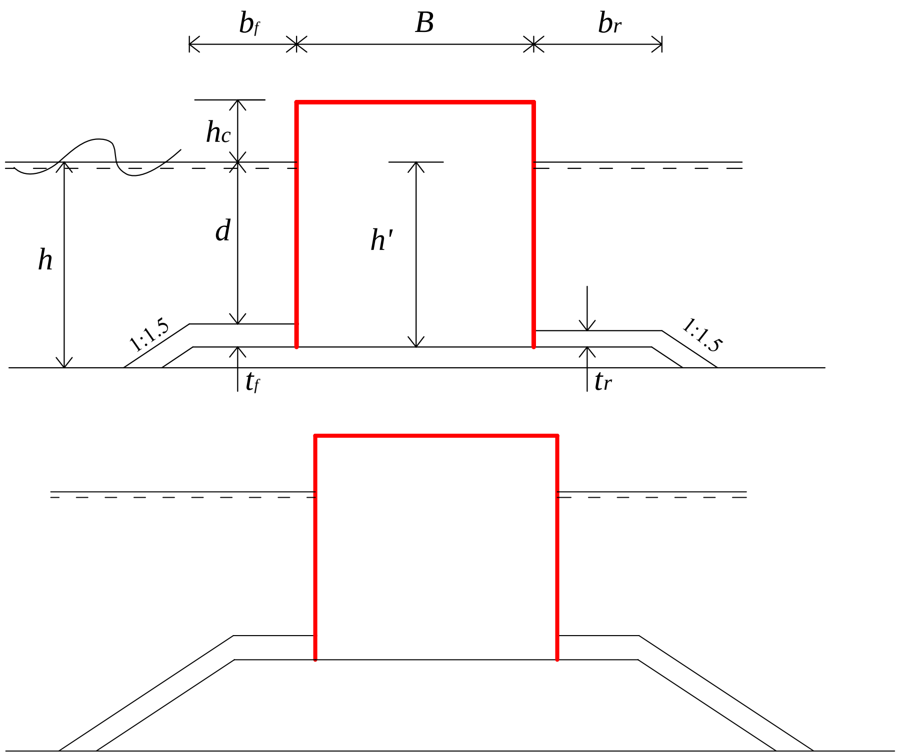



Fig. 7. Cross sections of outer caisson breakwaters on bedding layer (top) and high mound foundation (bottom).

### 3.2. Failure modes

So far only conditions with sea bed materials strong enough to resist slip failures have been analysed. The studied failure modes are shown in Fig. 8. For the slip failure the angle $\Phi$ giving the lowest resistance has been identified.

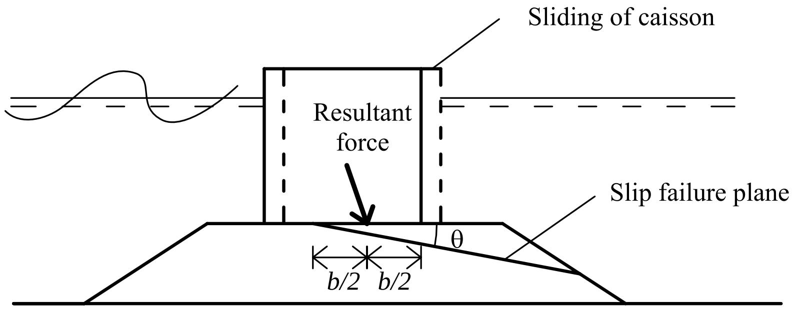



Fig. 8. Failure modes included in the optimization.

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

### 3.3. Repair policy and limit state performances.

Two methods of repair/stabilization are considered as shown in Fig. 9.

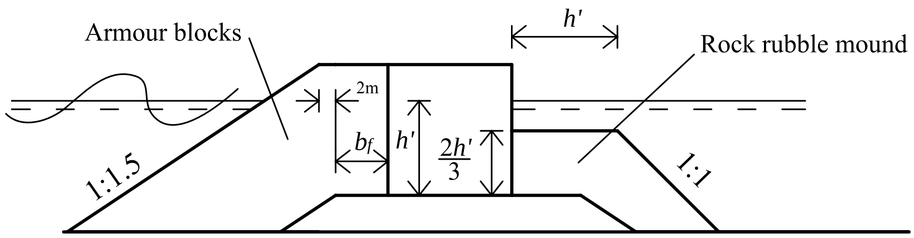



Fig. 9. Armour blocks in front of caisson and rubble mound behind caisson as means of repair.

The used limit state performances and related method of repair are given in table 6.

Table 6. 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<br>Slip failure | Both<br>Both, doubled unit price |

### 3.4. Bulk unit prices

Table 7 provides the average built-in bulk unit prices collected by the Working Group members. The Japanese prices are used in the present analyses.

Table 7. Average built-in bulk unit prices in Euro/$m^3$.

| 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 | 25 | 50 |

**3.5. 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 is more gentle than 1:50, and $d/h \geq 0.6$, see Fig. 7.

*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 $h$.

The design equation for $B$ reads

$$B = \frac{S \cdot F_H}{f\left[(\rho_c - \rho_w)h' g + \rho_c \, h_c \cdot g - \frac{1}{2} p_u\right]} \tag{2}$$

Where $F_H$ is the horizontal wave load corresponding to $H_{design}$, calculated by the Goda formula.
$S$ = 1.2 is a safety factor
$f$ = 0.6 is the friction coefficient of the base plate
$\rho_c$ = 2150 kg/$m^3$, bulk mass density of caisson
$\rho_w$ = 1025 kg/$m^3$, mass density of water
$\rho_u$ = wave induced uplift pressure at base plate front edge calculated by the Goda formula.

Tilting of the caisson around the heel applying a safety factor of $S$ = 2.5 is included in the deterministic determination of $B$, but was never critical.

The average normal stress σ over the effective foundation width $b$, see Fig. 8, is 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 are used the actual time series of Rayleigh distributed wave heights obtained from sample simulations in accordance with the long-term statistics, see Table 1 and PIANC (1992), including uncertainties on the distribution parameters. In order to avoid unrealistic wave heights was used double truncated Weibull distributions (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 (assuming truncated Normal-distribution) as follows:

Horizontal force, $0.5 < F_H < 1.4$, $\mu_{FH} = 0.90$, $\frac{\sigma_{FH}}{\mu_{FH}} = 0.20$

(3)

Uplift force, $0.5 < F_U < 1.4$, $\mu_{FH} = 0.80$, $\frac{\sigma_{FU}}{\mu_{FU}} = 0.30$

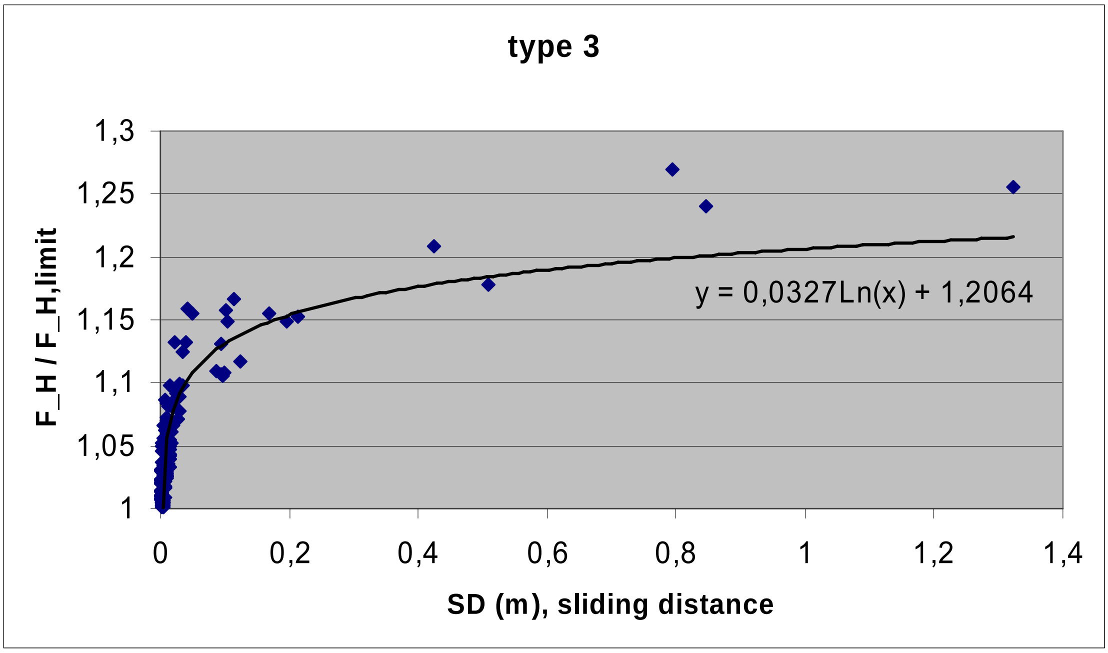
The friction factor *f* is modelled by a double truncated normal distribution with mean value $\mu_f = 0.6$, $\sigma_f / \mu_f = 0.1$, and cut-off limits 0.7 < *f* < 1.4.

In the slip failure calculations are used the reduced effective friction angle $\varphi_d$ based on normal distributed friction angle φ with $\mu_\varphi = 38^o$ and COV = 10%, and a normal distributed dilation angle $\psi$ with $\mu_\psi = 25^o$ and COV = 10%, cf. eq. 4

$$\tan \varphi_d = \frac{\sin \varphi \cos \psi}{1 - \sin \varphi \sin \psi} \quad (4)$$

For the equations related to the slip failure see Sorensen and Burcharth (2000).

The sliding distance SD of the caisson should in principle 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 is used the diagrams shown in Fig. 10. 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 calculated from eq. (2) with S = 1.



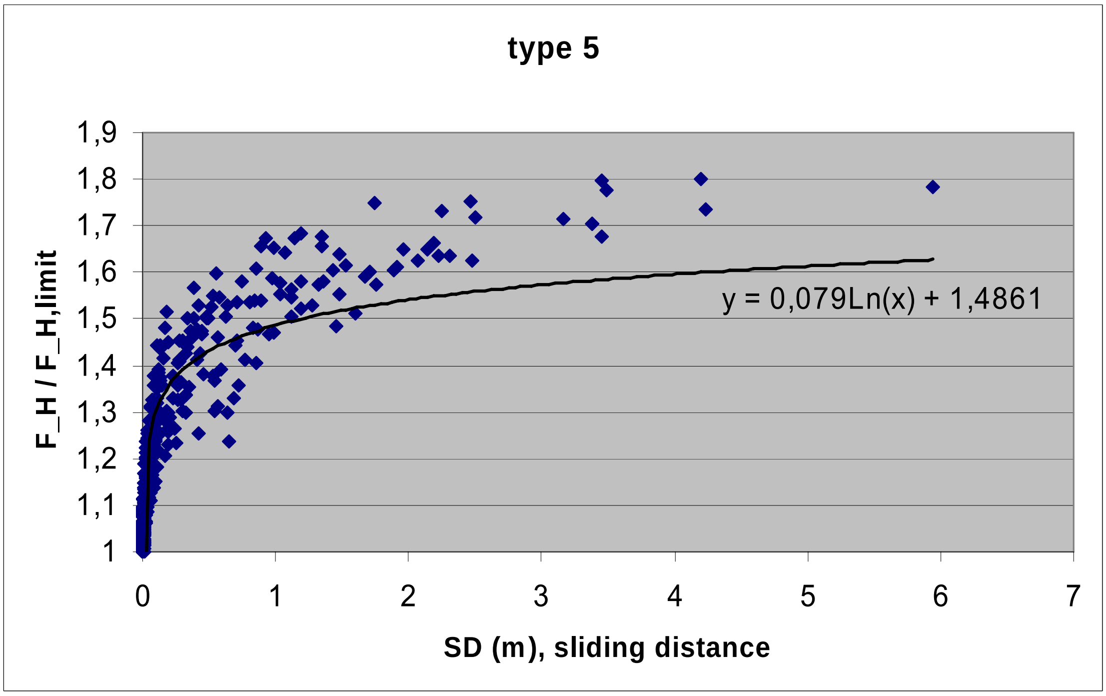




Fig. 10. Diagrams for the estimation of caisson sliding distance.

The data points in Fig. 10 was provided by Tae-Min Kim (2005), based on his earlier calculations for a caisson (type 3) in water depth $h$ = 16 m, of dimensions $h' \times h_c \times B \times d = 13 \times 5 \times 25.3 \times 11.5$ m, and a caisson (type 5) in water depth
$h = 24$ m, of dimensions $h' \times h_c \times B \times d = 14 \times 5 \times 26.8 \times 12.5$ m

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:

$$\lambda_1 = \lambda_3 = \begin{cases} 1.0 & \text{for } H_{max}/h < 0.3 \\ 1.2 - 0.67\,H_{max}/h & \text{for } 0.3 \le H_{max}/h < 0.6 \\ 0.8 & \text{for } H_{max}/h \ge 0.6 \end{cases} \quad (5)$$

$\lambda_2 = 0$

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. 11.

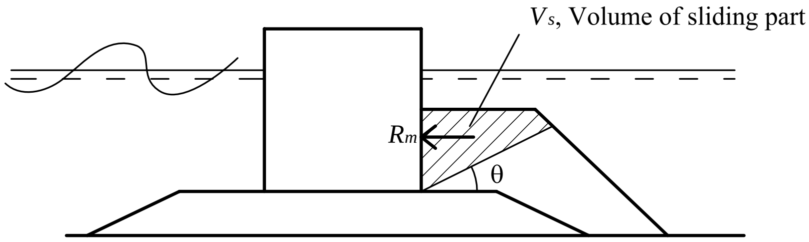



Fig. 11. Illustration of resistance of mound to sliding.

$$R_m = V_s \cdot \gamma' \tan(\theta + \varphi) \quad (6)$$

where $V_s$ is the volume of the sliding part of the mound

$\gamma' = 9810$ N/m$^3$ submerged unit weight of mound

$\varphi = 38^o$, angle of friction of mound material

$\theta$ is the slip plane angle with horizontal, to be identified related to min. $R_m$.

**3.6. Case studies**

Table 8 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 8. Case studies. Caissons on hard bottom.

| Case | Water depth, h (m) | Structure lifetime $T_L$ (years) | Wave climate | | Sliding equation type cf. Fig. 10 | RLS repair |
|---|---|---|---|---|---|---|
| | | | Location | $H_s^{T_L}$ (m | | |
| F1a | 15 | 100 | Follonica | 5.64 | 3 | Armour blocks in front |
| F1b | - | - | - | - | - | Mound behind |
| | | | | | | |
| B1a | 25 | 100 | Bilbao | 8.76 | 5 | Armour blocks in front |
| B1b | - | - | - | - | - | Mound behind |
| | | | | | | |
| S1a | 40 | 100 | Sines | 13.2 | 5 | Armour blocks in front |
| S1b | - | - | - | - | - | Mound behind |
| S2 | | 100 | Follonica | 5.64 | 5 | Mound behind |

The simulations show that there is hardly any difference in optimum safety levels whether initial repairs are made with armour blocks in front of the caisson or a mound behind the caisson. In the following are only shown cases with repair made by armour blocks in the front.

Table 9 and Fig. 12 show the results of Case F1a.

Table 9. Case F1a. Optimum safety levels for outer caisson breakwater in 15 m water depth. 100 years service lifetime.

| **Caisson breakwater optimization** | **Initial repair with blocks in front** | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Case: **F1a** | Structure lifetime $T_L$ = 100 years, | | | Water depth h = 15 m, | | | | Wave steepness $s_o$ = 0.04 | | | | |
| Seabed : Hard | | | | | | | | | | | | |
| Unit prices: Japanese | Waves: Follonica , $H_s^{T_L}$ = 5.64 m, | | | $H_s^{T_L}/h = 0.38$ , | | | | Freeboard $h_c = 0.6 H_s^{T_L}$ = 3.38 m | | | | |
| Interest rate: , 5 % p.a. | | | | | | | | | | | | |
| Downtime costs: 0 € | Friction factor f = 0.6 | | | , Friction angle φ = 38° | | | | , Dilation angle ψ = 25° | | | | |
| 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) | SLS | RLS | ULS | Slip failure | Construction (€/m) | Lifetime (€/m) |
| 10.5 | 9.0 | 1000 | 6.56 | 20.9 | 12 | 258 | 0.035 | 0.031 | 0.019 | 0.094 | 64157 | 68739 |
| 11.5 | 10.0 | 1000 | 6.56 | 19.9 | 11 | 290 | 0.011 | 0.011 | 0.004 | 0.047 | 61701 | 63954 |
| 12.5 | 11.0 | 50 | 5.36 | 16.3 | 9 | 320 | 0.053 | 0.045 | 0.034 | 0.090 | 52781 | 58972 |
| 13.5 | 12.0 | 50 | 5.36 | 16.4 | 9 | 339 | 0.030 | 0.024 | 0.013 | 0.035 | 52876 | 55141 |
| 14.5 | 13.0 | 25 | 5.07 | 15.9 | 9 | 360 | 0.039 | 0.035 | 0.027 | 0.007 | 51104 | 53162 |

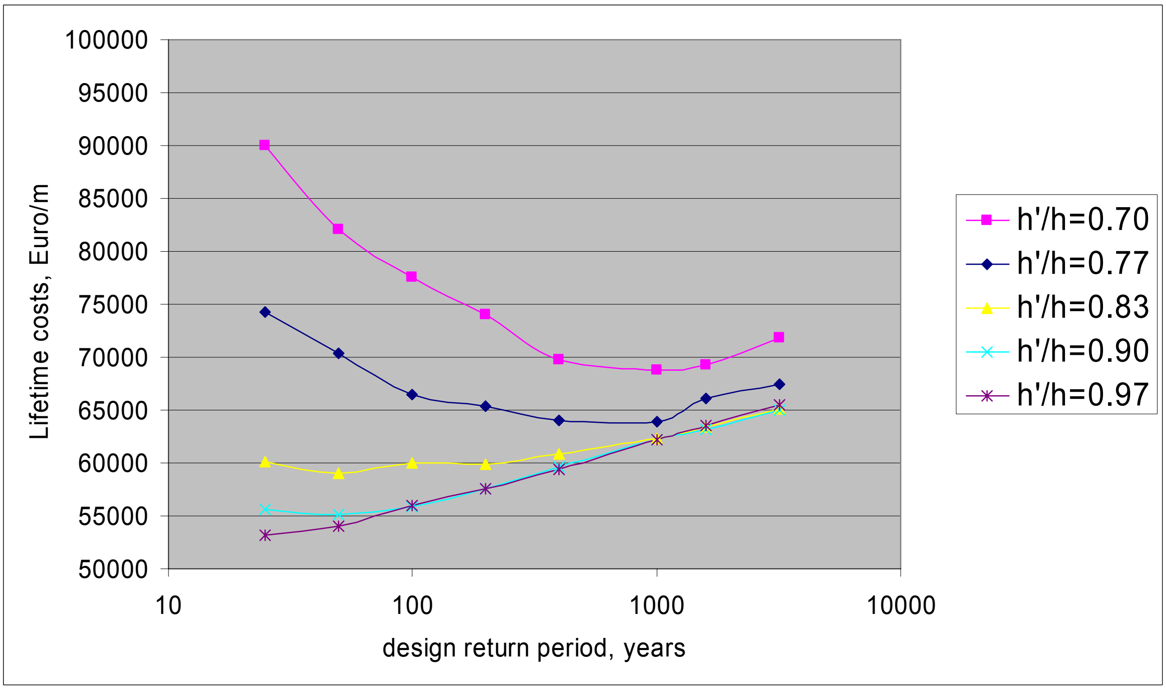



Fig. 12. Case F1a. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on return period applied in deterministic design.

Table 10 and Fig. 13 show the results of case B1a.

Table 10. Case B1a. Optimum safety levels for outer breakwater in 25 m water depth. 100 years service lifetime.

| **Caisson breakwater optimization** **Initial repair with blocks in front** | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Case: **B1a** | Structure lifetime $T_L$ = 100 years, | | | | Water depth h = 25 m, | | | | Wave steepness $s_o$ = 0.04 | | | |
| Seabed : Hard | | | | | | | | | | | | |
| Unit prices: Japanese | Waves: Bilbao , $H_s^{T_L}$ = 8.76 m, | | | | $H_s^{T_L}/h = 0.35$ , | | | | Freeboard $h_c = 0.6H_s^{T_L}$ = 5.26 m | | | |
| Interest rate: , 5 % p.a. | | | | | | | | | | | | |
| Downtime costs: 0 € | Friction factor f = 0.6 | | | | , Friction angle φ = 38° | | | | , Dilation angle ψ = 25° | | | |
| 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) | SLS | RLS | ULS | Slip failure | Construction (€/m) | Lifetime (€/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 |

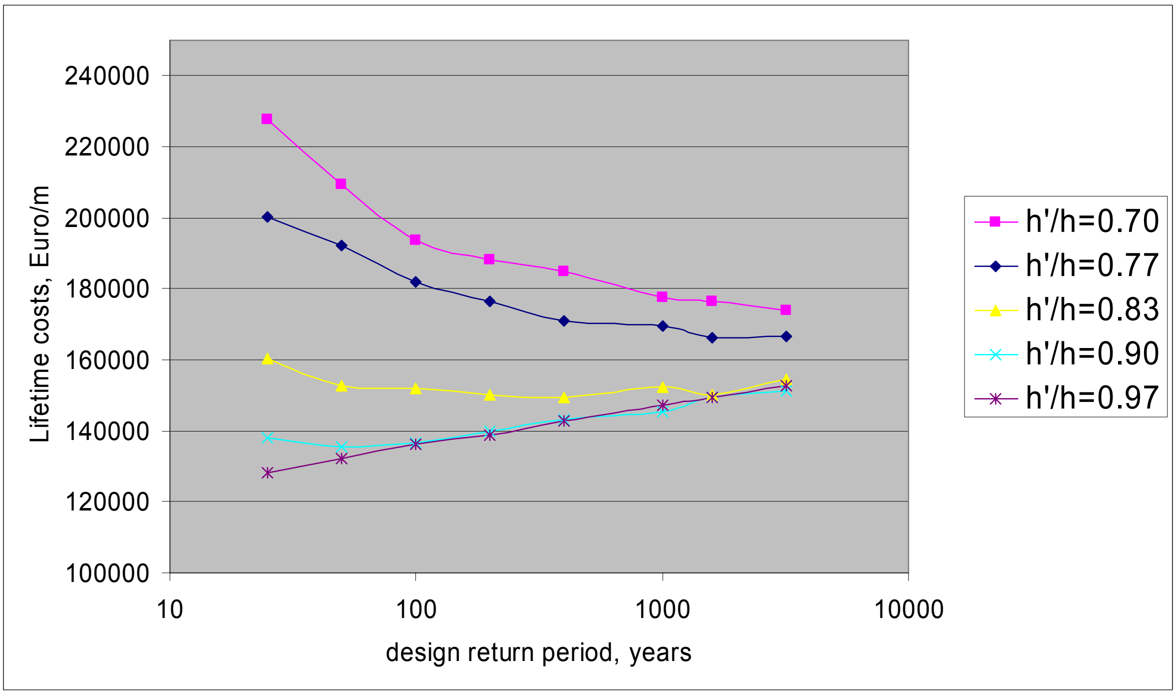



Fig. 13. Case B1a. Dependence of lifetime costs on relative height of caisson rubble mound foundation and on return period applied in deterministic design.

### 3.7. Preliminary conclusions related to outer caisson breakwaters on hard bottom.

For the investigated hard bottom cases where slip failures in the sea bed do not occur, the most economical designs, seen over the structure lifetime, are caissons placed on a bedding layer although the construction costs are almost independent on the relative height of the rubble foundation.

From the two cases it is seen that the optimum safety level in terms of optimum return period in deterministic design is much higher for caissons on a high mound than for a caisson on bedding layer. This is because of the higher probability of a geotechnical slip failure in case of high mounds.

It is also seen that the optimum limit state failure probabilities to be applied in probabilistic designs are significantly higher for the structure in 25 m water depth than for the structure in 15 m water depth. Roughly speaking, the SLS and RLS optimum failure probabilities within 100 years service lifetime are approximately 3 % in case of 15 m water depth and approximately 1 % in case of 25 m water depth.

Compared to the SLS optimum failure probabilities for outer rubble mound structures, the caisson values are two orders of magnitude smaller. This shows the fundamental difference in failure sensitivity between a rubble structure and a monolithic structure.

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