Design of Vertical Wall Caisson Breakwaters Using Partial Safety Factors

Hans F. Burcharth* and John Dalsgaard Sørensen**

1. Abstract

The paper presents a new system for implementation of target reliability in caisson breakwater designs by means of partial safety factors. The development of the system is explained, and tables of partial safety factors are presented for important overall stability failure modes related to caisson structures placed on bedding layers and high rubble mound foundations with underlaying sand and clay subsoils.

2. Introduction

Given the stochastic nature of wave loads it is important to deal with the involved uncertainties in a rational way when designing breakwaters. Application of the partial safety factor concept is generally accepted as a rational solution to implementation of safety in designs, and is adopted in many national codes as well as in the EUROCODE. The partial safety factors are in existing codes calibrated against experience with the performance of numerous civil engineering structures in which way it is assured that a conventional structure, such as a house, will obtain the usual safety when designed using the prescribed safety factors. However, the actual safety in terms of probability of a certain damage within a certain span of years is unknown. For breakwaters this seems not to be a suitable concept because extensive experience with existing structures is not available. Moreover, it is deciable to know the actual safety of a design also because rational comparisons of alternative designs have to be performed on the basis of equal safety levels. Reliability analysis, f.ex. using a level 2 First Order Reliability Method (FORM), can of course be done for any structure by means of computer programs. However, it is regarded a help to the designers to make a partial safety factor system available which in an easy way makes it possible to design a breakwater to any target reliability level.

Such a system was introduced and developed by the PIANC PTC II Working Group 12 on Rubble Mound Breakwaters (Burcharth 1991 and 1993) and has now been further developed to cover caisson breakwaters by the PIANC PTC II Working Group 28 on Vertical Wall Breakwaters.

The partial safety factor system is developed on the basis of the validity of the Goda (1985) wave load formula. This formula is not valid for design cases where frequent wave breaking directly on the caisson wall takes place. This causes very large short-duration impulsive loads for which design tools are hardly developed. Steep seabed slopes or semi-high rubble slopes in front of the structure can trigger such unfavorable wave conditions. Goda (1985 pp 132-138) provides advice as how to avoid such excessive impact loadings.

---

*Prof. Dr.Techn. Hydraulics and Coastal Engineering Laboratory, Inst. of Civil Eng., Aalborg University
**Associate prof. Ph.D. Inst. of Building Technology and Structural Engineering, Aalborg University

The overall procedure in development of the partial safety factor system comprises the following steps:

- Identification of failure modes
- Formulation of limit state equations for the failure modes
- Modelling of uncertainties related to loads (waves), strengths (soils, concrete) and limit state equations
- Selection of format for the partial safety factor system
- Calibration of the partial safety factors
- Verification

3. Failure modes

All possible failure modes must be considered in the design. The present paper deals with the overall stability failure modes illustrated in Fig.1. A more complete discussion of failure modes is given in Burcharth (1998).

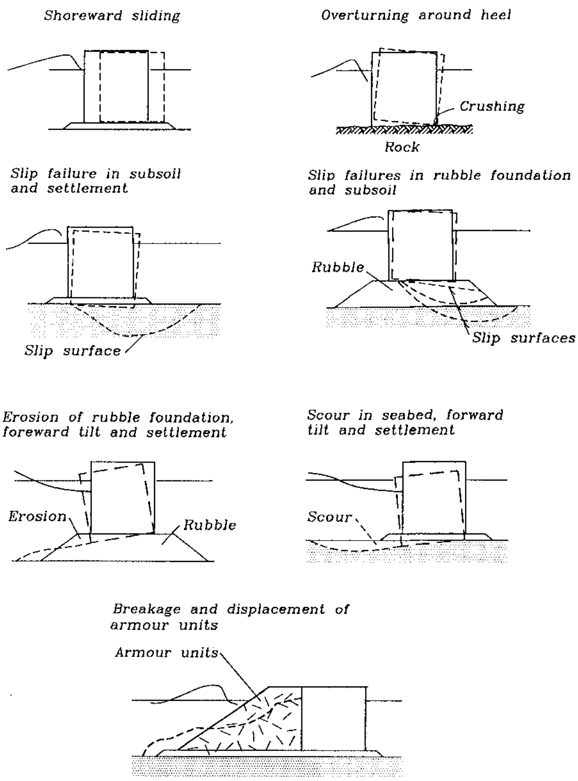

Fig.1. Important overall stability failure modes.

4. Wave modelling

For calibration of partial safety factors the maximum significant wave height in T years is denoted H_S^T and is modelled by the extreme Weibull distribution function:

$$F_{H_S^T}(h_S) = \left[1 - exp\left(-\left(\frac{h_S - H_S'}{\beta}\right)^{\alpha}\right)\right]^{\lambda T} \quad (1)$$

Wave data from 4 quite different geographical locations are selected, see table 1 where h_s is the water depth, N is number of samples and λ is the number of observations per year.

	N	λ	α	β	H_S'	h_s
Bilbao	50	4.17	1.39	1.06	4.9	29
Sines	15	1.25	1.78	2.53	7.1	35
Tripoli	15	0.75	1.83	3.24	2.9	27
Follonica	46	5.94	1.14	0.58	2.69	10

Table 1. Wave data from different locations fitted to a Weibull distribution. β, H_S' and h_s are in meters.

The wave data from Bilbao, Sines and Tripoli correspond to deep water waves while the wave data from Follonica correspond to shallow water waves. In order to model the statistical uncertainty α and β are modelled as Normal distributed variables.

The model uncertainty related to the quality of the measured wave data is modelled by a multiplicative stochastic variable Z_{H_S} which is assumed to be normal distributed with expected value 1 and standard deviation $\sigma'_{Z_{H_S}}$. Good and poor wave data could be represented by $\sigma'_{Z_{H_S}} = 0.05$ and 0.2, corresponding to accelerometer bouy and fetch diagram estimates, respectively.

5. Soil strength modelling

The undrained shear strength of clay is modelled by a log-Gaussian distributed stochastic field $\{c_u(x,z)\}$ where z and x are vertical and horisontal coordinates, respectively. The expected value function $E[c_u(x,z)]$ and the covariance function $Cov[c_u(x_1,z_1), c_u(x_2,z_2)]$ is written

$$E[c_u(x,z)] = E[c_u(z)] \quad (2)$$

$$Cov[c_u(x_1,z_1), c_u(x_2,z_2)] = Cov[c_u(x_1 - x_2, z_1 - z_2)] \quad (3)$$

where (x_1,z_1) and (x_2,z_2) are two points in the soil. $E[c_u(x,z)]$ gives the expected value in depth z of the undrained shear strength of clay. $Cov[c_u(x_1,z_1), c_u(x_2,z_2)]$ gives the covariance between c_u at position (x_1,z_1) and c_u at position (x_2,z_2). $Var[c_u(x_1,z_1)] = Cov[c_u(x_1,z_1), c_u(x_1,z_1)]$ is the variance of c_u at position (x_1,z_1).

It is seen that the expected value depends on the depth and the covariance depends on the vertical and horisontal distances. Generally the correlation lengths in horisontal and vertical direction will be different due to the soil stratification.

The mean value function and covariance function are assumed to be modelled by

$$E[c_u(x,z)] = c_{u0} + c_{u1}z$$
$$Cov[c_u(x_1,z_1), c_u(x_2,z_2)] = \sigma_{c_u}^2 \exp\Big(-|\alpha(z_1 - z_2)|\Big) \exp\Big(-(\beta(x_1 - x_2))^2\Big)$$

where c_{u0} and c_{u1} model the expected value, σ_{c_u} is the standard deviation and α and β model the correlation.

Since the breakwater foundation is made of friction material and it is assumed that foundation failure modes can develope both in the rubble mound and in sand subsoil, statistical models for the effective friction angle and the angle of dilation are needed for the rubble material and the sand subsoil. These angles are modelled by Lognormal stochastic variables.

6. Estimation of partial safety factors for one failure mode

In code calibration based on first order reliability methods (FORM) it is assumed that the limit state function can be written

$$g(\mathbf{x}, \mathbf{z}) = 0 \tag{4}$$

where $\mathbf{x} = (x_1, \ldots, x_n)$ is a realization of $\mathbf{X} = (X_1, \ldots, X_n)$. External loads (e.g. wave), strength parameters and model uncertainty variables are examples of uncertain quantities. $\mathbf{z} = (z_1, \ldots, z_N)$ are N design variables which are used to design the actual structure. Realizations $\mathbf{x}$ of $\mathbf{X}$ where $g(\mathbf{x}, \mathbf{z}) \leq 0$ corresponds to failure states, while $g(\mathbf{x}, \mathbf{z}) > 0$ corresponds to safe states.

If the number of design variables is $N = 1$ then the design (modelled by z) can be determined from the **design equation**

$$G(\mathbf{x}^c, z, \gamma) \geq 0 \tag{5}$$

$\mathbf{x}^c = (x_1^c, \ldots, x_n^c)$ are characteristic values corresponding to the stochastic variables $\mathbf{X}$. $\gamma = (\gamma_1, \ldots, \gamma_m)$ are m partial safety factors. The partial safety factors γ are usually defined such that $\gamma_i \geq 1, i = 1, ..., m$. In the most simple case $m = n$.

The design equation is closely connected to the limit state function (4). In most cases the only difference is that the variables $\mathbf{x}$ are exchanged by design values $\mathbf{x}^d$ obtained from the characteristic values $\mathbf{x}^c$ and the partial safety factors γ.

The characteristic values are for variable load variables usually the 98 % fractile of the distribution function of the stochastic variables. For the significant wave height the characteristic value $\hat{H}_S^{T_L}$ is chosen as the central estimate of the significant wave height which in average is exceeded once every T_L years. The design values for load variables are then obtained from

$$x_i^d = x_i^c \gamma_i \tag{6}$$

The characteristic values are for strength variables usually the 5 % or 50 % fractiles of the distribution function of the stochastic variables. Here the 50 % fractile is used in order to obtain partial safety factors larger than or equal to 1. The design values are then obtained from

$$x_i^d = \frac{x_i^c}{\gamma_i} \tag{7}$$

The limit state / design equations are formulated either as a force balance or, in case of foundation failure modes, as work equations using the upper bound theory of plasticity related to kinematically admissible rupture failures. Figure 2 illustrates two typical cases.

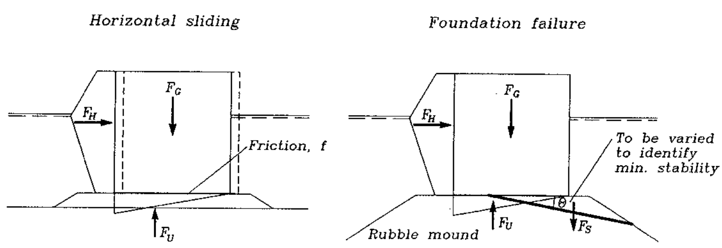

Figure 2. Illustration of failure modes for formulation of limit state design equations.

For sliding failure the limit state function can be written

$$g = (F_G - Z_{F_V} F_U(Z_{H_S} H_D))f - Z_{F_H} F_H(Z_{H_S} H_D) \tag{8}$$

where

F_G	reduced weight of caisson under water
F_U	wave induced uplift force
F_H	horisontal wave force
H_D	design wave height
Z_{H_S}	model uncertainty related to the significant wave height H_S
ρ_c	density of the caisson
Z_{F_H}	model uncertainty on horisontal wave load
Z_{F_V}	model uncertainty on vertical wave load
f	friction coefficient

The design equation corresponding to (8) is written

$$G = \frac{1}{\gamma_Z}(F_G^c - 0.77F_U^c)f^c - 0.90F_H^c \tag{9}$$

where

f^c mean of base friction coefficient

γ_Z partial safety factor on f^c

F_G^c mean value of reduced weight of caisson under water

F_U^c and F_H^c wave induced uplift force and horisontal wave force calculated by Goda formulae using $\gamma_H \hat{H}_D^{T_L}$ as wave height, where γ_H is the partial safety factor and $\hat{H}_D^{T_L}$ is the expected maximum wave height in a storm with T_L-years return period (often taken as $1.8\hat{H}_S^{T_L}$). The factors $Z_{F_V}^c$=0.77 and $Z_{F_H}^c$=0.90 compensates for the bias (safety) implemented in the Goda formulae.

For foundation failure the limit state function can be written

$$g = (F_S + F_G - Z_{F_V} F_U(Z_{H_S} H_D))\omega_V - Z_{F_H} F_H(Z_{H_S} H_D)\omega_H \tag{10}$$

where

F_S boyancy reduced gravitational force on the sliding soil element

ω_V displacement vector, $\omega_V = sin(\varphi_d - \theta)/cos\varphi_d$

ω_H displacement vector. $\omega_H = cos(\varphi_d - \theta)/cos\varphi_d$

The reduced effective angle of friction is calculated from

$$\tan\varphi_d = \frac{\sin\varphi_r' \cos\psi_r}{1 - \sin\varphi_r' \sin\psi_r}$$

where φ_r' is the effective angle of friction and ψ_r is the dilation.

The design equation is written

$$G = (F_S^c + F_G^c - 0.77F_U^c)\omega_V^c - 0.90F_H^c\omega_H^c \tag{11}$$

where F_S^c is the mean value of F_S. ω_V^c and ω_V^c are obtained using the the design value of $\tan\varphi_d^c$ determined from

$$\gamma_Z \tan\varphi_d^c = \frac{\sin\varphi_r^c \cos\psi_r^c}{1 - \sin\varphi_r^c \sin\psi_r^c} \tag{12}$$

where γ_Z is the partial safety factor on $\tan\varphi_d^c$.

The application area for the code is described by a number, L of different typical structures. The partial safety factors γ are calibrated such that the reliability indices corresponding to the L structures are as close as possible to the target reliability index $\beta_t = -\Phi^{-1}(P_f^t)$, where P_f^t is the target probability of failure. This is formulated by the following optimization problem

$$\min_{\gamma} \quad W(\gamma) = \sum_{j=1}^{L} w_j(\beta_j(\gamma) - \beta_t)^2 \tag{13}$$

where $w_j, j = 1, \ldots, L$ are weighting factors ($\sum_{j=1}^{L} w_j = 1$) indicating the relative frequency of appearance of the different design situations. $\beta_j(\gamma)$ is the reliability index for structure no. j

7. Format for partial safety factors

Partial safety factors are calibrated with the following code entries:

- the design lifetime T_L (= 20, 50 or 100 years)
- the acceptable probability of failure P_f (= 0.01, 0.05, 0.10, 0.20 or 0,40) corresponding to the target reliability indices β_T (= 2.33, 1.65, 1.28, 0.84 or 0.25)
- the coefficient of variation $\sigma'_{Z_{H_S}}$ = (0.05 and 0.20).
- Deep or shallow water conditions.
- Hydraulic model test or not.

The partial safety factors are:

- a load partial safety factor $\gamma_P = 1$ to be multiplied to the permanent load.
- a load partial safety factor γ_H to be multiplied to $\hat{H}_S^{T_L}$ (the central estimate of the significant wave height which in average is exceeded once every T_L years). The design wave height is to be taken as a multiplum of $\hat{H}_S^{T_L}$.
- a safety factor γ_Z to be used with friction materials in rubble mound and/or subsoil (tangent to the mean value of the friction angle is divided by γ_Z).
- a safety factor γ_C to be used with the undrained shear strength of clay materials in the subsoil (the mean value of the undrained shear strength is divided by γ_C).

8. Limit state functions and design equations

For calibration of the partial safety factors the parameters for the stochastic variables shown in table 2 are used. The correlation coefficient between Z_{F_H} and Z_{M_H} and between Z_{F_V} and Z_{M_V} are estimated roughly to 0.9. In table 2 D denotes a deterministic variable, N(μ, σ) denotes a normal distribution with expected value μ and standard deviation σ and LN(μ, σ) denotes a lognormal distribution.

The tidal elevation ζ is modelled as a stochastic variable with distribution function $F_\zeta(\zeta) = \frac{1}{\pi} \arccos\left(-\frac{\zeta}{\zeta_0}\right)$ where ζ_0 is the maximum tidal height. $\zeta_0 = 0.75$ m is used.

	distribution	reference
ρ_c	N(2.1, 0.1075)	Burcharth (1992)
Z_{F_H}	N(0.90, 0.25)	Bruining (1994)
Z_{F_V}	N(0.77, 0.25)	Bruining (1994)
Z_{M_H}	N(0.81, 0.40)	Bruining (1994)
Z_{M_V}	N(0.72, 0.37)	Bruining (1994)
ψ_r	LN(0.43, 0.043)	
φ'_r	LN(0.61, 0.061)	
ψ_s	LN(0.35, 0.035)	
φ'_s	LN(0.52, 0.052)	
U_{c_u}	N(0, 1)	
c_{u0}	150 kPa	
c_{u1}	0	
σ_{c_u}	D(37.5 kPa)	
α	D(0.33)	
β	D(0.033)	
Z	N(1, 0.1)	
f	N(0.636, 0.0954)	Takayama (1992)
ζ	see eq. (14)	Takayama (1992)
H_S	ex Weibull	
Z_{H_S}	$N(1, \sigma'_{Z_{H_S}})$	

Table 2. Statistical parameters for calibration of partial safety factors for foundation failure with sand subsoil.

If model tests have been performed to estimate the wave forces the model uncertainties shown in table 3 can be used.

	distribution	reference
Z_{F_H}	N(0.90, 0.05)	Van der Meer et al. (1994)
Z_{F_V}	N(0.77, 0.05)	Van der Meer et al. (1994)
Z_{M_H}	N(0.81, 0.10)	Van der Meer et al. (1994)
Z_{M_V}	N(0.72, 0.10)	Van der Meer et al. (1994)

Table 3. Statistical parameters for model uncertainties when wave forces are determined on the basis of model tests.

8.1 Horizontal sliding

Equations are given in section 6.

8.2 Scour failure for circular roundheads on sand

The design equation is written, see Sumer et al. (1996) (no rubble foundation):

$$G = \frac{1}{\gamma_Z}\frac{S^c}{B^c} - 0.5(1 - \exp(-0.175(KC(\gamma_H \hat{H}_S^{T_L}) - 1)))$$

where s_p is the wave steepness and

$$KC = \frac{U_m T_p}{B_r} \qquad U_m = \frac{\pi Z_{H_S} H_S}{T_p} \frac{1}{\sinh(2\pi h'_s / L_p)}$$

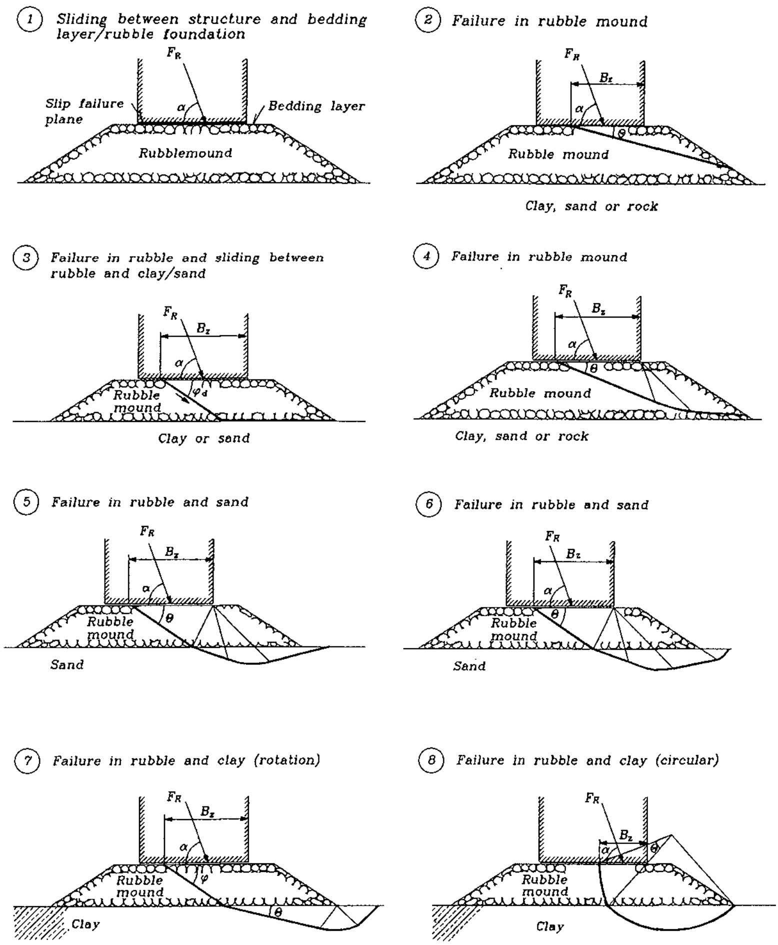

Figure 3. Foundation failure modes.

$$T'_p = \sqrt{\frac{Z_{H_S} H_S}{s_p} \frac{2\pi}{g}}$$

and the wave length L_p is determined from

$$L_p = g\frac{T_p^2}{2\pi} \tanh(2\pi h'_s / L_p)$$

8.3 Hydraulic instability of foundation rubble mound armour layer

The design equation is written, see Madrigal et al. (1995) :

$$G = \frac{1}{\gamma_Z} \Delta^c D_n^c (5.8 \frac{h'}{h_s} - 0.60) N_{od}^{0.19} - \gamma_H \hat{H}_S^{T_L}$$

8.4 Foundation failure modes

Figure 3 shows the investigated foundation failure modes. The complete sets of design equations can be found in Burcharth (1998) and J.D. Sørensen et al. (1998).

9. Partial Safety Factors

Below is shown the results of the probabilistic calibration of partial safety factors. In deterministic design of the breakwater the following bias values for the forces and moments are to be used:

	value
$\hat{Z}_{F_H}$	0.90
$\hat{Z}_{F_V}$	0.77
$\hat{Z}_{M_H}$	0.81
$\hat{Z}_{M_V}$	0.72

Table 4. Values of model uncertainties to be used in deterministic design.

Foundation failure - sand subsoil:

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
$P_f(\beta_t)$	γ_H	γ_Z	γ_H	γ_Z
0.01	1.4	1.4	1.4	1.4
0.05	1.3	1.3	1.3	1.4
0.10	1.2	1.3	1.2	1.3
0.20	1.1	1.2	1.1	1.2
0.40	1.1	1.1	1.1	1.1

Table 5. Partial safety factors for *foundation failure - sand subsoil - deep water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	1.3	1.4	1.3
0.05	1.3	1.2	1.4	1.2
0.10	1.2	1.2	1.3	1.2
0.20	1.1	1.2	1.1	1.2
0.40	1.1	1.1	1.1	1.1

Table 6. Partial safety factors for *foundation failure - sand subsoil - deep water - model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.5	1.4	1.3	1.5
0.05	1.4	1.3	1.3	1.4
0.10	1.3	1.2	1.2	1.3
0.20	1.2	1.1	1.1	1.3
0.40	1.1	1.0	1.1	1.1

Table 7. Partial safety factors for *foundation failure - sand subsoil - shallow water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	1.3	1.4	1.3
0.05	1.3	1.3	1.4	1.3
0.10	1.2	1.2	1.3	1.2
0.20	1.1	1.1	1.1	1.1
0.40	1.1	1.1	1.1	1.1

Table 8. Partial safety factors for *foundation failure - sand subsoil - shallow water - model tests performed.*

Foundation failure - clay subsoil:

	$\sigma'_{Z_{H_S}} = 0.05$			$\sigma'_{Z_{H_S}} = 0.2$		
P_f	γ_H	γ_Z	γ_C	γ_H	γ_Z	γ_C
0.01	1.3	1.4	1.4	1.4	1.4	1.4
0.05	1.2	1.3	1.3	1.3	1.3	1.3
0.10	1.1	1.2	1.3	1.2	1.2	1.3
0.20	1.0	1.1	1.2	1.0	1.1	1.2
0.40	1.0	1.1	1.1	1.0	1.0	1.1

Table 9. Partial safety factors for *foundation failure - clay subsoil - deep water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$			$\sigma'_{Z_{H_S}} = 0.2$		
P_f	γ_H	γ_Z	γ_C	γ_H	γ_Z	γ_C
0.01	1.2	1.4	1.4	1.3	1.4	1.4
0.05	1.1	1.3	1.3	1.2	1.3	1.3
0.10	1.0	1.2	1.2	1.1	1.2	1.3
0.20	1.0	1.1	1.1	1.0	1.1	1.2
0.40	1.0	1.0	1.1	1.0	1.0	1.1

Table 10. Partial safety factors for *foundation failure - clay subsoil - deep water - model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$			$\sigma'_{Z_{H_S}} = 0.2$		
P_f	γ_H	γ_Z	γ_C	γ_H	γ_Z	γ_C
0.01	1.2	1.7	1.5	1.3	1.7	1.5
0.05	1.1	1.5	1.4	1.2	1.5	1.4
0.10	1.1	1.4	1.3	1.2	1.4	1.4
0.20	1.0	1.3	1.2	1.1	1.3	1.3
0.40	1.0	1.2	1.2	1.1	1.2	1.2

Table 11. Partial safety factors for *foundation failure - clay subsoil - shallow water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$			$\sigma'_{Z_{H_S}} = 0.2$		
P_f	γ_H	γ_Z	γ_C	γ_H	γ_Z	γ_C
0.01	1.2	1.5	1.4	1.3	1.5	1.4
0.05	1.1	1.4	1.3	1.2	1.4	1.3
0.10	1.1	1.3	1.3	1.1	1.3	1.3
0.20	1.0	1.2	1.2	1.1	1.2	1.2
0.40	1.0	1.1	1.1	1.1	1.1	1.1

Table 12. Partial safety factors for *foundation failure - clay subsoil - shallow water - model tests performed.*

Sliding failure:

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.4	2.0	1.5	2.0
0.05	1.3	1.7	1.4	1.7
0.10	1.3	1.5	1.4	1.5
0.20	1.3	1.2	1.3	1.2
0.40	1.1	1.1	1.1	1.1

Table 13. Partial safety factors for *sliding failure - deep water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	1.7	1.4	1.7
0.05	1.2	1.6	1.3	1.6
0.10	1.2	1.4	1.3	1.4
0.20	1.2	1.2	1.2	1.2
0.40	1.1	1.2	1.1	1.1

Table 14. Partial safety factors for *sliding failure - deep water - model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.3	2.2	1.4	2.2
0.05	1.2	1.9	1.3	1.9
0.10	1.2	1.7	1.3	1.7
0.20	1.2	1.3	1.2	1.3
0.40	1.0	1.2	1.0	1.2

Table 15. Partial safety factors for *sliding failure - shallow water - no model tests performed.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.2	1.7	1.3	1.6
0.05	1.1	1.5	1.2	1.5
0.10	1.1	1.3	1.2	1.3
0.20	1.1	1.2	1.1	1.2
0.40	1.0	1.1	1.0	1.1

Table 16. Partial safety factors for *sliding failure - shallow water - model tests performed.*

Scour failure:

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	2.0	2.4	2.0	2.4
0.05	2.0	2.0	2.0	2.0
0.10	2.0	1.8	2.0	1.8
0.20	2.0	1.5	2.0	1.5
0.40	2.0	1.2	2.0	1.2

Table 17. Partial safety factors for *scour failure for circular roundheads - deep water.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	2.0	2.4	2.0	2.4
0.05	2.0	2.0	2.0	2.0
0.10	2.0	1.8	2.0	1.8
0.20	2.0	1.5	2.0	1.5
0.40	2.0	1.2	2.0	1.2

Table 18. Partial safety factors for *scour failure for circular roundheads - shallow water.*

Armour layer failure:

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.6	1.3	1.7	1.3
0.05	1.4	1.2	1.5	1.2
0.10	1.3	1.2	1.4	1.2
0.20	1.2	1.1	1.3	1.1
0.40	1.1	1.0	1.2	1.0

Table 19. Partial safety factors for *armour layer failure - deep water.*

	$\sigma'_{Z_{H_S}} = 0.05$		$\sigma'_{Z_{H_S}} = 0.2$	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.5	1.5	1.6	1.5
0.05	1.3	1.3	1.4	1.3
0.10	1.2	1.2	1.3	1.2
0.20	1.1	1.2	1.2	1.2
0.40	1.1	1.0	1.2	1.0

Table 20. Partial safety factors for *armour layer failure - shallow water.*

10. References

Burcharth H.F.: Introduction of partial coefficient in the design of rubble mound breakwaters. Conference on Coastal Structures and Breakwaters, ICE (Inst. of Civil Engineers), London, Nov. (1991).

Burcharth H.F.: Reliability-based design of coastal structures. ICCE92, Short course, Venice, Italy, (1992).

Burcharth H.F.: Development of a partial coefficient system for the design of rubble mound breakwaters. PIANC PTC II Working Group 12, Subgroup-F, final report, December 1991. Published by PIANC in (1993).

Bruining J.W. : Wave forces on vertical breakwaters. Reliability of design formula. Delft Hydraulics Report H 1903, MAST II contract MAS2-CT92-0047, (1994).

Burcharth H.F.: Identification of design tools for vertical wall breakwaters. PIANC PTC II Working Group 28, Report by Sub-Group A. (1998).

Madrigal B.G. and J.M. Valds: Results on stability tests for rubble foundation of a composite vertical breakwater. MAST II/MCS, CEPYC-CEDEX, Madrid, (1995).

Sumer B.M. & J. Fredsøe : Scour at the head of a vertical-wall breakwater. Coastal Engineering, (1996).

Sørensen J.D. and Burcharth H.F.: Implementation of safety in the design. PIANC PTC II Working Group 28, Report by Sub-Group D. (1998).

Takayama T.: Estimation of sliding failure probability of present breakwaters for probabilistic design. Report of Port and Harbour Research Institute, Yokosuka, Japan, Vol. 31, No. 5, (1992).

Van der Meer J., K. d'Angremond and J. Juhl: Probabilistic calculations of wave forces on vertical structures. Proc. Coatal Engineering, Kobe, Japan, pp. 1754-1767 (1994).