


Definitiestudie uitbreiding CRESS

CUR

9 februari 2006
Definitief rapport
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A COMPANY OF

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INHOUDSOPGAVE

1 INLEIDING

1.1 Achtergrond en doel

Het huidige computerprogramma CRESS (Coastal and River Engineering Support System) zal worden uitgebreid met in totaal 30 nieuwe of aangepaste rekenregels. Deze rekenregels zijn afkomstig uit de nieuwste versie van de 'Rock Manual' (CUR 169, conceptversie). In onderstaande tabel zijn de 30 rekenregels weergegeven:

Nr.	Naam rekenregel
1a 1b	Wave run up on sloping structures Wave run up on rough slopes – explicit formulae
2	Golf run down
3	Reshaping berm breakwaters
4	Overtopping per wave
5	Velocities and layer thickness in overtopping waves
6	Small waves and relatively large freeboards
7	Smooth low crested structures
8	Rubble mound low crested structures
9	Reflection
10	Seepage flow through rock
11	Damage to near bed structures
12a 12b	Submerged structures statically stable Submerged structures dynamically stable
13	Low crested structures
14	Rock Armour
15	Cubes
16	Tetrapods
17	Dolosse
18	Accropode
19	Berm breakwater
20	Reshaping berm breakwater
21	Toe stability
22	Toe in front of a caisson breakwater
23	Rear side stability of a breakwater
24	Wave load on crown wall
25	Impact pressure on a crown wall
26	Forces on a crown wall acc. to Martin
27	Escarameia and May
28	Maynord
29	Stability ship induced currents
30	Stability of near bed structures

Dit rapport betreft de definitiestudie ten behoeve van de uitbreiding van het huidige computerprogramma CRESS. Op basis van de definitiestudie wordt de programmeur in staat geacht de rekenregels in het bestaande programma te implementeren.

De werkzaamheden zijn uitgevoerd door Ir. K.A.J. van Gerven en Ir. W. de Jong. Projectleider was M.A. van Heereveld M. Sc. (Eng). Opdrachtgever was RWS Bouwdienst via CUR; het project is van de opdrachtgeverszijde begeleid door Ir. K. Dorst (RWS Bouwdienst), Ir. J. Koenis (CUR) en Ir. H.J. Verhagen (TU Delft).

1.2 Uitgangspunten en randvoorwaarden

Als basis voor het opstellen van de definitiestudie is uitgegaan van de door de opdrachtgever aangeleverde conceptversie van de nieuwe Rock Manual:

- CUR 169, Manual on the use of Rock in Hydraulic Engineering, versie 1995 en 2004 (conceptversie).

De indicatieve waarden voor de geldigheidsgrenzen van de invoer parameters zijn waar mogelijk ontleend aan één van onderstaande bronnen:

- Bouwdienst Rijkswaterstaat, Dienst Weg en Waterbouwkunde, Definitierapport ten behoeve van de programmeerwerkzaamheden van de vernieuwde rekenregels, WB 975, REKEN-R-97030, oktober 1997;
- Het huidige computerprogramma CRESS (versie 2.3).

Voor de invoerparameters waarvoor geen indicatieve waarden kunnen worden herleid uit voorgenoemde bronnen zijn de indicatieve grenzen dusdanig vastgesteld dat met voldoende zekerheid kan worden gesteld dat de desbetreffende parameter binnen deze grenzen valt. De indicatieve grenzen liggen in dit geval dus ver uit elkaar.

De beschrijving van de rekenregels is uitgevoerd in het Engels opdat de teksten kunnen worden overgenomen als helptekst in het computerprogramma. In de beschrijving is, naast de informatie ten behoeve van het programmeren, in kaders achtergrondinformatie bij de rekenregels weergegeven ten behoeve van de eindgebruiker.

2 OPBOUW EN RUBRICERING VAN DE REKENREGELS

2.1 Structuur van de rekenregels

Voor ieder van de in het volgende hoofdstuk uitgewerkte 30 rekenregels is een gelijke opbouw gehanteerd. Deze opbouw ziet er als volgt uit:

- algemene beschrijving van de rekenregel;
- de te hanteren formules met bijbehorende parameters;
- overzicht van de in- en uitvoer parameters;
- geldigheidsgrenzen en mathematische grenzen voor de invoerparameters;
- overzicht van de achtergrondinformatie (bronnen) bij de desbetreffende rekenregel.

De hierboven genoemde onderdelen zijn per rekenregel in afzonderlijke paragraven beschreven. De nummering van de rekenregels loopt van 1 tot 30.

De gegeven geldigheidsgrenzen worden opgelegd door het geldigheidsgebied van de parameters in de gebruikte formule, vaak afkomstig uit experimenten (aangegeven met (f) of zijn slechts indicatief (i). Overschrijding van de (f)-grenzen moet in het programma leiden tot een melding, met de mogelijkheid om te kiezen tussen “hiermee rekenen of waarde aanpassen”. De gebruiker moet bij de invoer van elke parameter de mogelijkheid hebben om de geldigheidsgrenzen in te zien.

De mathematische grenzen worden opgelegd door de gebruikte formule t.b.v. de voortgang van de berekening (bijvoorbeeld delen door nul), waaraan ook altijd moet worden voldaan.

2.2 Relatie tussen de rekenregels en de bestaande menustructuur van CRESS

Onderscheid kan worden gemaakt tussen rekenregels die al bestaan binnen het computerprogramma CRESS en rekenregels die aan CRESS moeten worden toegevoegd. Voor de eerste categorie geldt dat slechts een aanpassing dan wel een aanvulling van de bestaande rekenregel benodigd is en dat de rekenregel dus al gerubriceerd is binnen het huidige computerprogramma CRESS. Voor de tweede categorie geldt dat sprake is van een nieuwe rekenregel. Voor de nieuwe rekenregels is een voorschot gedaan voor een mogelijke inpassing binnen de huidige menustructuur van het programma CRESS.

In Tabel 2-1 is een overzicht gegeven van de inpassing van de 30 rekenregels in de huidige menustructuur van het computerprogramma CRESS (versie 2.3). De eerste 10 rekenregels vallen onder de hoofd directory ‘Water Movement’ en de overige 20 rekenregels onder de hoofd directory ‘Structures’.

Tabel 2-1: Inpassing van de rekenregels in de huidige menustructuur van het computerprogramma CRESS

Water movement			
	Wind waves and swell		
		Wave / structure interaction	
			Wave rundown and wave runup
			Rekenregel 1a: Wave run-up on sloping structures
			Rekenregel 1b: Wave run up on rough slopes – explicit formulae
			Rekenregel 2: Wave run-down
			Overtopping
			Rekenregel 3: Reshaping berm breakwaters
			Rekenregel 4: Overtopping per wave
			Rekenregel 5: Velocities and layer thickness in overtopping waves
			Wave transmission
			Rekenregel 6: Small waves and relatively large freeboards
			Rekenregel 7: Smooth low crested structures
			Rekenregel 8: Rubble mound low crested structures
			Reflection
			Rekenregel 9: Reflection
	Flow		
		Flow and structures	
		Rekenregel 10: Seepage flow through rock	
Structures			
	Protection against waves		
		Rock and stone structures	
		Rekenregel 11: Damage to near bed structures	
		Rekenregel 12a: Submerged structures statically stable	
		Rekenregel 12b: Submerged structures dynamically stable	
		Rekenregel 13: Low crested structures	
		Rekenregel 14: Rock armour layers	
		Rekenregel 15: Cubes	
		Rekenregel 16: Tetrapods	
		Rekenregel 17: Dolosse	
		Rekenregel 18:Accropode and xBlocs	
		Rekenregel 19: Berm breakwater	
		Rekenregel 20: Reshaping berm breakwaters	
		Rekenregel 21: Toe stability	
		Rekenregel 22: Toe in front of Caison breakwater	
		Rekenregel 23: Rear side stability of a breakwater	
		Rekenregel 24: Wave load on crown wall	
		Rekenregel 25: Impact pressure on a crown wall	
		Rekenregel 26: Forces on a crown wall according to martin	
	Protection against currents		
		Stone stability bank and dike revetments	
		Rekenregel 27: Escarameia and May	
		Rekenregel 28: Maynord	
		Rekenregel 29: Stability ship induced currents	
		Rekenregel 30: Stability of near bed structures	

3 REKENREGELS

1a. Wave run-up on sloping structures

In the Netherlands a prediction curve has been developed, reported in the Technical Report on Wave Run-up and Overtopping at Dikes by the TAW (2002), in which the breaker parameter, $\xi_{m-1,0}$, is calculated by using the spectral significant wave height ($H_s = H_{m0}$) and the mean energy wave period, $T_{m-1,0}$, instead of the significant wave height ($H_s = H_{1/3}$) from time-domain analysis and the peak wave period, T_p, as in the methods by Ahrens (1981) and Allsop *et al* (1985). The mean energy wave period $T_{m-1,0}$ accounts for the influence of the spectral shape and shallow foreshores (Van Gent, 2001).

1a.1 Equations

$$R_{u2\%} / H_{m0} = A \gamma_b \gamma_f \gamma_\beta \xi_{m-1,0} \tag{1a.1}$$

with a maximum of:

$$R_{u2\%} / H_{m0} = \gamma_f \gamma_\beta \left(B - C / \sqrt{\xi_{m-1,0}} \right) \tag{1a.2}$$

in which:

$$\xi_{m-1,0} = \frac{\tan \alpha}{\sqrt{\dfrac{2\pi H_s}{g T_{m-1,0}{}^2}}} \tag{1a.1.1}$$

A description of the correction factor to calculate run-up of oblique (short-crested) waves is given by the following equation:

$$\gamma_\beta = 1 - 0.0022 |\beta| \quad \text{for } 0° \le |\beta| \le 80° \tag{1a.1.4}$$

$$\gamma_\beta = 1 - 0.0022 |80°| \quad \text{for } 80° \le |\beta|$$

A correction factor for the influence of berms, γ_b, is proposed in TAW (2002). This correction factor consists of two factors, one for the influence of the berm width, k_B, and one for the position of the middle of the berm in relation to SWL (still water level), k_h:

$$\gamma_b = 1 - k_B (1 - k_h) \quad \text{with } 0.6 \le \gamma_b \le 1.0 \tag{1a.1.5}$$

This method is valid for berms not wider than 1/4 of the wavelength L_o (wavelength in deep water, here in this method based on $T_{m-1,0}$). As this method is also only valid for calculating the influence of angled berms up to 1:15, first angled berms must be drawn to equivalent horizontal berms as is shown in figure 1a.1. If berms are steeper than 1:15, it is suggested that wave run-up (and overtopping) is calculated by interpolation between the steepest berm (1:15) and a plane slope (1:8) or by interpolation between the longest possible berm (L_o/4) and a shallow foreshore.

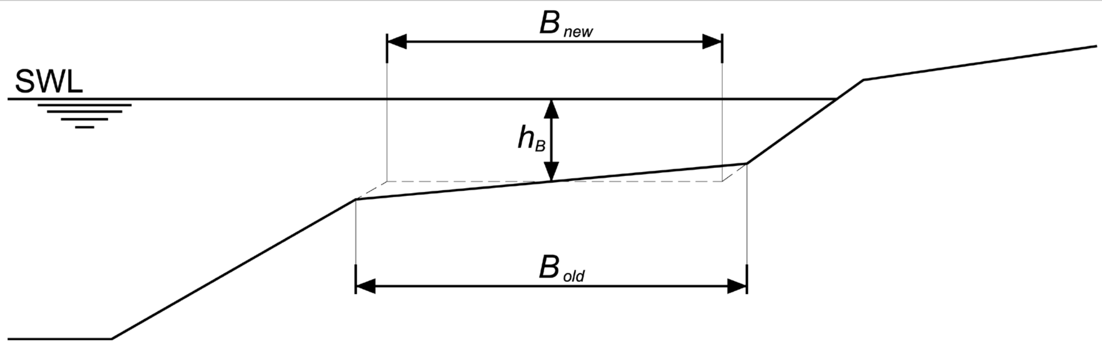

Figure 1a.1 Definition of berm width, B, and depth, h_B

The influence of the berm width factor, k_B, can be found by examining the change in the slope (see Figure 1.1), given by Equation 1a.1.6:

$$k_B = 1 - \frac{2H_{m0} / L_{berm}}{2H_{m0} / (L_{berm} - B_B)} = \frac{B_B}{L_{berm}} \qquad (1a.1.6)$$

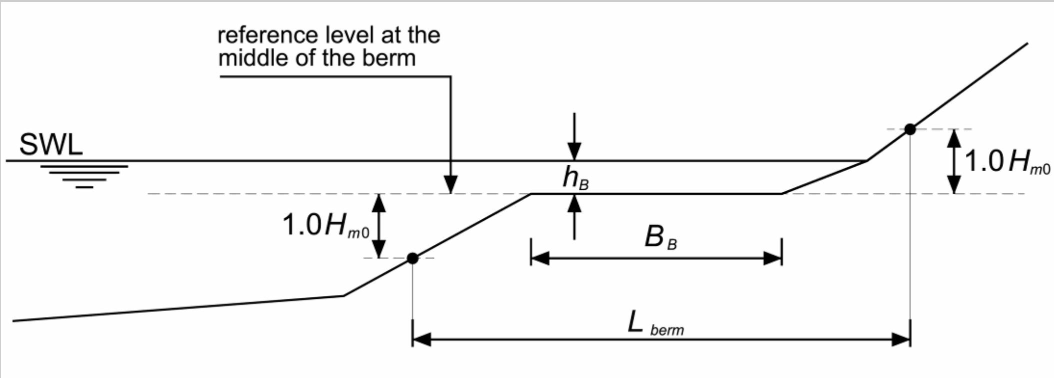

Figure 1a.2 Changes in slope for berms

With the approach from TAW (2002), a berm positioned on the still water line is most effective. The influence of the berm disappears when the berm lies higher than the run-up level, $R_{u2\%}$, on the lower slope or when it lies more than 2 H_{m0} below SWL. The influence of the berm position can be determined using a cosine function, in which the cosine is given in radians by Equation 1a.1.7:

$$k_h = 0.5 - 0.5\cos\left(\pi \frac{h_B}{x}\right) \qquad (1a.1.7)$$

where

x = $R_{u2\%}$ if berm is above still water line, ie $-R_{u2\%} < h_B < 0$

x = $2H_{m0}$ if berm is below still water line, ie $0 \leq h_B < 2H_{m0}$

k_h = 1 if berm is outside influence area, ie $h_B \leq -R_{u2\%}$ or $h_B \geq 2H_{m0}$)

NOTE: In the case of a berm above SWL, an iterative approach should be adopted to calculate the eventual value of the wave run-up, as this parameter is part of Equation 1a.1.6 (via Equation 1a.1.8) to determine the correction factor for the influence of berms, γ_b. Standard procedure is to start with a value of $R_{u2\%} = 1.5H_{m0}$ or $2H_{m0}$, and then check the result of the calculation as to whether the deviation is acceptable or not.
For more details on this method, see TAW (2002).

An overview of the used parameters is given below:

parameter	short description	unit
$R_{u2\%}$	wave run up height for the 2% wave	[m]
H_s (=H_{m0})	Significant wave height calculated from the spectrum, $H_{m0}=4\sqrt{m_0}$	[m]
A	coefficient	[-]
γ_b	berm-factor	[-]
γ_f	roughness factor	[-]
γ_β	correction factor for oblique waves	[-]
$\xi_{m-1,0}$	breaker parameter based on spectral analysis	[-]
α	slope angle	[º]
B	coefficient	[-]
C	coefficient	[-]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
T_p	peak wave period	[s]
β	Angle of wave attack with respect to the structure	[º]

1a.2 input and output parameters

Input:	Output:
H_{m0}, A, γ_b, γ_f, γ_β, α, B, C, T_p	$\mathbf{R_{u2\%}}$, $\xi_{m-1,0}$, $T_{m-1,0}$

1a.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$R_{u2\%}$	wave run up height for the 2% wave		>0
Hs (=H_{m0})	Significant wave height calculated from the spectrum, $H_{m0}=4\sqrt{m_0}$	0,1 – 8 (i)	>0
A	coefficient	see table 1.1	
γ_b	berm-factor	0.6 - 1.0 (f)	> 0
γ_f	roughness factor	see table 1.2	

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
		$\gamma_b\ \xi_{m-1,0}$ < 1,8 (f)	
γ_β	correction factor for oblique waves	0,82 – 1 (f)	> 0
$\xi_{m-1,0}$	breaker parameter based on spectral analysis	0,5 – 8 à 10 (i)	> 0
cot α	slope angle 1: …	1,33 – 5 (f)	> 0
B	coefficient	see table 1.1	
C	coefficient	see table 1.1	
$T_{m-1,0}$	spectral wave period, also called the energy wave period	1 – 10 (i)	> 0
T_p	peak wave period	1 – 10 (i)	> 0
β	Angle of wave attack with respect to the structure	-90º – 90º (f)	-90º - 90º

For the coefficients *A*, *B* and *C* in Equations 1a.1 and 1a.2 values have been derived representing the average trend through the used data set for use in probabilistic calculations. Values that contain a safety margin of one standard deviation are suggested for deterministic use. Both values for these coefficients are presented in Table 1a.1, for more details on this method see TAW (2002).

Table 1a.1 Values for the coefficients A, B and C in Equations 1a.1 and 1a.2

Coefficients (in Eq. 1.1 and 1.2)	Values with safety margin - deterministic calculations	Values without safety margin - average trend / probabilistic calculations
A	1.75	1.65
B	4.3	4.0
C	1.6	1.5

Table 1a.2 Values for roughness reduction factor, γ_f (TAW, 2002)

Structure type	γ_f
Concrete, asphalt and grass	1.0
Stone blocks	0.80 - 0.95
Armourstone - single layer on impermeable base	0.70
Armourstone - two layers on impermeable base	0.55
Armourstone - permeable base	See figure 5.5 Rock Manual and supporting text

Note:
For the TAW method using Equations 1a.1 and 1a.2, the roughness factor, γ_f, is only applicable for $\gamma_b\ \xi_{m-1,0}$ < 1.8. For larger values this factor increases linearly up to 1 for $\gamma_b\ \xi_{m-1,0}$ = 10 and it remains 1 for larger values.

1a.4 References

AHRENS, J. P. (1981) Irregular wave run-up on smooth slopes. Technical Paper 81-17, U.S. Army Corps of Eng., Coastal Eng. Res. Center, Fort Belvoir

ALLSOP, N. W. H. BRADBURY, A. P., POOLE, A. B., DIBB, T. E. and HUGHES, D. W. (1985) Rock durability in a marine environment, HR Wallingford, Report SR 11

VAN GENT, M.R.A. (2001) Wave run-up on dikes with shallow foreshores, ASCE, J. of Waterway, Port, Coastal and Ocean Engineering, Vol.127, No.5, Sept/Oct 2001, pp.254-262, ASCE

TAW (2002) Technical Report Wave Run-up and Overtopping at Dikes, Technical Advisory Committee for Water Defences, The Netherlands

1b Wave run up on rough slopes – explicit formulae

As an alternative to the use of the roughness correction factors, explicit formulae have been derived from tests with rough rubble slopes.
For most wave conditions and structure slope angles, a rubble slope will dissipate significantly more wave energy than the equivalent smooth or non-porous slope. Run-up levels will therefore generally be reduced. This reduction is influenced by the permeability of the armour, filter and underlayers, and by the wave steepness, $s = H/L$ (-). To obtain an alternative for simply using a roughness correction factor, run-up levels on rubble slopes armoured with rock armour or rip-rap have been measured in laboratory tests, using either regular or random waves. In many instances the rubble core has been reproduced as fairly permeable. Test results therefore often span a range within which the designer must interpolate.

1b.1 Equations

Analysis of test data from measurements by Van der Meer and Stam (1992) has given prediction formulae (Equations 1b.1 and 1b.2) for rock armoured slopes with an impermeable core, described by a notional permeability factor $P = 0.1$, and for porous mounds of relatively high permeability, given by $P = 0.5$ and 0.6.

$$R_{u\,n\%}/H_s = a\xi_m \qquad \text{for } \xi_m < 1.5 \qquad (1b.1)$$

$$R_{u\,n\%}/H_s = b\xi_m^{\ c} \qquad \text{for } \xi_m > 1.5 \qquad (1b.2)$$

in which:

$$\xi_m = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{gT_m}}} \qquad (1b.1.1)$$

The run-up for permeable structures ($P > 0.4$) is limited to a maximum:

$$R_{u\,n\%}/H_s = d \qquad (1b.3)$$

An overview of the used parameters is given below:

parameter	short description	unit
$R_{u\,n\%}$	Run-up level exceed by only n% of run-up tongues	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
ξ_m	Surf similarity parameter for the mean period wave	[-]
a	Coefficient	[-]
b	Coefficient	[-]
c	Coefficient	[-]
d	Coefficient	[-]

parameter	short description	unit
α	Slope angle	[º]
g	Gravitational acceleration	[m/s^2]
T_m	Mean wave period	[s]
P	structure permeability	[-]

1b.2 input and output parameters

Input:	Output:
H_s, a, b, c, d, cot α, T_m, P	$\mathbf{R_{u\,n\%}}$, ξ_m

1b.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$R_{u\,n\%}$	Run-up level exceed by only n% of run-up tongues	< d * H_s (formule 1b.3) for permeable structures (*P* > 0.4) (f)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0,1 – 8 (i)	> 0
ξ_m	Surf similarity parameter for the mean period wave	0,5 – 6 (i)	> 0
a	Coefficient	see Table 1b.1	
b	Coefficient	see Table 1b.1	
c	Coefficient	see Table 1b.1	
d	Coefficient	see Table 1b.1	
cot α	Slope angle 1: …	1 – 10 (i)	> 0
T_m	Mean wave period	1 – 10 (i)	> 0
P	structure permeability	0,1 – 0,6 (i)	> 0

Values for the coefficients *a*, *b*, *c* and *d* in the Equations 1b.1 to 1b.3 have been determined for various exceedance levels of the run-up, see Table 1b.1. The experimental scatter of *d* is within 0.07.

Table 1b.1 Coefficients in Equations 1b.1 to 1b.3

Run-up level *n*%	*a*	*b*	*c*	*d*
0.1	1.12	1.34	0.55	2.58
1	1.01	1.24	0.48	2.15
2	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
50 (median)	0.47	0.60	0.34	0.82

1b.4 References

VAN DER MEER J. W., J.W. and STAM, C.J.M. (1992), Wave run-up on smooth and rock slopes of coastal structures, ASCE, Journal of WPC&OE, Vol.118, No.5, 534-550.

2 Wave run-down

The lower extreme water level reached by a wave on a sloping structure is known as wave run-down, R_d. Run-down is defined relative to the still water level (SWL) and will be given as positive if below SWL, as shown in Figure 2.1.

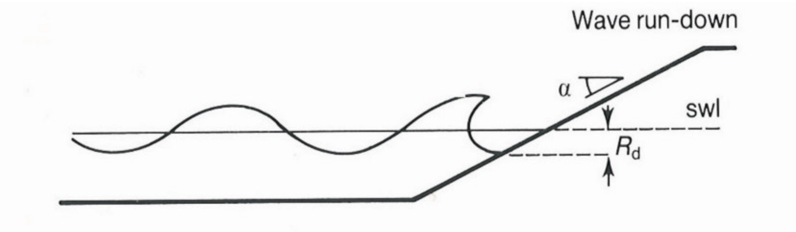

Figure 2.1: Wave run-down on a slope

2.1 Equations

Run-down on **plane smooth slopes** can be calculated with Equations 2.1 and 2.2:

$$R_{d2\%}/H_s = 0.33\xi_p \quad \text{for } 0 < \xi_p < 4 \tag{2.1}$$

$$R_{d2\%}/H_s = 1.5 \quad \text{for } \xi_p > 4 \tag{2.2}$$

Run-down levels on **porous rubble slopes** are influenced by the permeability of the structure and the surf similarity parameter. For wide-graded rock armour or rip-rap on an impermeable slope a simple expression (see Equation 2.3) for a maximum run-down level, taken to be around the 1% level, has been derived from test results by Thompson and Shuttler (1977):

$$R_{d1\%}/H_s = 0.34\xi_p - 0.17 \tag{2.3}$$

$$\xi_p = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{g T_p^{\ 2}}}} \tag{2.3.1}$$

Analysis of run-down by van der Meer (1988) has given a relationship – Equation 2.4 – that includes the effects of structure permeability, *P*, slope angle, α, and wave steepness, s_m:

$$R_{d2\%}/H_s = 2.1\sqrt{\tan\alpha} - 1.2P^{0.15} + 1.5\exp(-60\,s_m) \tag{2.4}$$

$$s_m = \frac{2\pi H_s}{g T_m} \tag{2.4.1}$$

An overview of the used parameters is given below:

parameter	short description	unit
$R_{d2\%}$	wave run down height for the 2% wave	[m]
$R_{d1\%}$	wave run down height for the 1% wave	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
ξ_p	breaker parameter based on the peak wave characteristics	[-]
g	gravitational acceleration	[m/s^2]
α	slope angle	[º]
T_p	peak wave period	[s]
P	structure permeability	[-]
s_m	wave steepness for mean wave period	[-]
T_m	mean wave period	[s]

2.2 Input and output parameters

Input:	Output:
Plane smooth slopes: H_s, cotα, T_p	 $\mathbf{R_{d1\%}}$, ξ_p
Porous rubble slopes: H_s, cotα, T_p	 $\mathbf{R_{d2\%}}$, ξ_p
Van de Meer (1988): H_s, cotα T_m, P	 $\mathbf{R_{d2\%}}$, s_m

2.3 Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$R_{d2\%}$	wave run down height for the 2% wave		> 0
$R_{d1\%}$	wave run down height for the 1% wave		> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0,1 – 8 (i)	> 0
ξ_p	breaker parameter based on the peak wave characteristics	0,5 – 6 (i)	> 0
cot α	slope angle 1: …	1 - 10 (i)	> 0
T_p	peak wave period	1 – 10 (i)	> 0
P	structure permeability	0,1 -0,6 (i)	> 0
T_m	mean wave period	1 – 10 (i)	> 0

2.4 References

THOMPSON, D.M. and SHUTTLER, R.M. (1976) Design of riprap slope protection against wind waves, Report 61, CIRIA, London

VAN DER MEER, J. W. (1988) Rock slopes and gravel beaches under wave attack. Doctoral thesis, Delft University of Technology; also Delft Hydraulics Communication No. 396

3 Reshaping berm breakwaters

There are very few measurements of wave overtopping on berm breakwaters. Lissev (1993) measured time-averaged overtopping on a reshaped berm breakwater and derived Equation 3.1.

3.1 Equations

$$q/\sqrt{gH_s^{\,3}} = 1.5\exp(-2.1\frac{R_c}{H_s}) \qquad (3.1)$$

in which

parameter	short description	unit
H_s	significant wave height	[m]
g	acceleration of gravity	[m/s^2]
R_c	freeboard (height of the crest above still water level)	[m]
q	average specific overtopping discharge	[m^3/s/m]

3.2 Input and output parameters

Input:	Output:
H_s, R_c	**q**

3.3 Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H_s	Significant wave height, average of highest 1/3 of all wave heights	0,1 – 10 (i)	> 0
R_c	crest freeboard, level of crest relative to still water level	0,1 – 5 (i)	> 0
q	average specific overtopping discharge	0,001 – 5 (i)	> 0

3.4 References

LISSEV, N. (1993) Influence of the core configuration on the stability of berm breakwaters, Experimental model investigations. Report No R-6-93, Department of Structural Engineering, University of Trondheim, Norwegian Institute of Technology

4 Overtopping per wave

Overtopping volumes per wave differ substantially from the average wave overtopping discharge.

4.1 Equations

The distribution of the volumes of individual overtopping events can be described by the Weibull probability distribution function, as given in Equation 4.1:

$$P(V) = \Pr(\underline{V} < V) = 1 - \exp\left(-\left(\frac{V}{a}\right)^{b}\right) \qquad (4.1)$$

The maximum expected individual overtopping volume, V_{max} (m^3 per m), in a sequence of N incoming waves is given by Equation 4.2.

$$V_{max} = a\left(\ln\left(N_{ov}\right)\right)^{1/b} \qquad (4.2)$$

In Besley (1999), values for sloping seawalls are suggested for the coefficients *a* and *b* in the Equations 4.1 and 4.2, using the average overtopping discharge calculated with Owen's method. Equations 4.3 and 4.4 give the relation between the coefficient , *a*, and the relevant parameters: wave period, specific discharge and the proportion of waves overtopping a seawall. Equations 4.3 and 4.4 are valid for values of the wave steepness (s_p) between 0.02 and 0.04.

$$a = \left\{0{,}85 + \frac{s_p - 0{,}02}{0{,}02}(0{,}96 - 0{,}85)\right\} T_m \, qN/N_{ov} \qquad (4.3)$$

$$b = 0{,}76 + \frac{s_p - 0{,}02}{0{,}02}(0{,}92 - 0{,}76) \qquad (4.4)$$

$$s_p = \frac{2\pi H_s}{g(1.15 \cdot T_m)} \qquad (4.4.1)$$

$$N = \frac{3600 t_r}{T_m} \qquad (4.5)$$

In Besley (1999) the proportion of waves overtopping a seawall – or the probability of overtopping per wave – is given by Equation 4.6, valid in the range 0.05 < R^* < 0.3:

$$N_{ov}/N = \exp\left(-C\left(R^*/\gamma_f\right)^2\right) \qquad (4.6)$$

with:

$$R^* = R_c / \left(T_m \left(gH_s\right)^{0.5}\right) \qquad (4.7)$$

In TAW (2002), the value $b = 0.75$ is suggested for the shape parameter together with Equation 4.8 as the expression for the scale parameter, using the average overtopping discharge as calculated with the TAW method:

$$a = 0.84 T_m \, q \, N / N_{ov} \tag{4.8}$$

where N_{ov}/N is the proportion of the overtopping waves, given by Equation 4.9:

$$N_{ov}/N = \exp\left(-\left(\sqrt{-\ln 0.02} \frac{R_c}{R_{u2\%}} \right)^2 \right) \tag{4.9}$$

Equation 4.9 is valid for situations in which the wave run-up distribution conforms to the Rayleigh distribution. For this method, the 2% wave run-up, $R_{u2\%}$, can be calculated using Equations 1a.1 and 1a.2.

An overview of the used parameters is given below:

parameter	**short description**	**unit**
$P(V)$	Pr($\underline{V} < V$) = probability of non-exceedance of a given volume, *V*	[-]
a	scale parameter	[m³ /m]
b	shape parameter	[-]
V_{max}	maximum expected individual overtopping volume	[m³ /m]
T_m	mean wave period	[s]
t_r	duration of the storm or wave record	[hrs]
N	number of incoming waves	[-]
N_{ov}	number of overtopping waves out of a total of *N* incoming waves in an examined time period T_r (= $N T_m$)	[-]
q	specific discharge	[m³/s/m]
R^*	dimensionless freeboard	[-]
γ_f	roughness coefficient	[-]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
C	parameter depending on the slope	[-]
R_c	crest freeboard, level of crest relative to still water level	[m]
$R_{u2\%}$	wave run up height for the 2% wave	[m]
s_p	wave steepness for peak wave period	[-]

4.2 Input and output parameters

Input:	Output:
t_r, T_m, $R_{u\,2\%}$, R_c, H_s, γ_f, q, b	**N_{ov}**, **V_{max}**, N, a, R^*

4.3 Boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$P(V)$	Pr($\underline{V} < V$) = probability of non-exceedance of a given volume, V	0 – 1 (f)	0 - 1
b	shape parameter	Owen's method: b = 0,76 for s_p = 0,02 and b = 0,92 for s_p = 0,04 (f) TAW method: b = 0.75 (f)	
V_{max}	maximum expected individual overtopping volume		> 0
T_m	mean wave period	0 – 20 (i)	> 0
t_r	duration of the storm or examined time period	1 - 24 (i)	> 0
N	number of incoming waves	1 – 7500 (i)	> 0
N_{ov}	number of overtopping waves out of a total of N incoming waves in an examined time period t_r (= $N\,T_m$)	1 - N	$0 < N_{ov} < N$
q	specific discharge	0,001 – 5 (i)	> 0
R^*	dimensionless freeboard	0,05 – 0,3 (f)	> 0
γ_f	roughness coefficient	see table 4.1 TAW method: $\gamma_b\ \xi_{m-1,0}$ <≈ 2.0 (f)	
H_s	Significant wave height, average of highest 1/3 of all wave heights	1 – 10 (i)	> 0
C	parameter depending on the slope	C = 38 for 1:2 and C = 110 for 1:4 (f)	> 0
R_c	crest freeboard, level of crest relative to still water level	0,1 – 5 (i)	> 0
$R_{u\,2\%}$	wave run up height for the 2% wave		> 0

Table 4.1 Values for roughness reduction factor, γ_f, Besley (1999) and TAW (2002)

Structure type	γ_f for Owen method	Structure type	γ_f for TAW method
Smooth concrete or asphalt	1.0	Concrete, asphalt and grass	1.0
Stone blocks	0.95	Stone blocks	0.80 - 0.95
Armourstone - single layer on impermeable base	0.80	Armourstone - single layer on impermeable base	0.70
Armourstone - single layer on permeable base	0.55 - 0.60	Armourstone - two layers on impermeable base	0.55
Armourstone - two layers	0.50 - 0.55		

Note:

For the TAW method, the roughness factor γ_f is only applicable for $\gamma_b\ \xi_{m-1,0}$ <≈ 2.0. For larger values this factor increases linearly up to 1 for $\gamma_b\ \xi_{m-1,0}$ = 10 and it remains 1 for larger values.

4.4 References

BESLEY, P. (1999) Overtopping of Seawalls, HR Wallingford, Report W 178, Environment Agency

TAW (2002) Technical Report Wave Run-up and Overtopping at Dikes, Technical Advisory Committee for Water Defences, The Netherlands

5 Velocities and layer thickness in overtopping waves

5.1 Equations

Van Gent (2002) and Schüttrumpf and Van Gent (2003) use the Equations 5.1 and 5.2 for wave run-up, taking into account a smooth transition from plunging to surging breakers.

$$R_{u2\%}/(\gamma H_s) = c_0 \xi_{s-1,0} \quad \text{for } \xi_{s\ 1,0} \leq p \tag{5.1}$$

$$R_{u2\%}/(\gamma H_s) = c_1 - c_2/\xi_{s-1,0} \quad \text{for } \xi_{s\ 1,0} \geq p \tag{5.2}$$

in which:

$$c_2 = 0{,}25 c_1^{\,2}/c_0 \tag{5.2.1}$$

$$p = 0{,}5 c_1/c_0 \tag{5.2.1}$$

The surf-similarity parameter is defined as:

$$\zeta_{s-1,0} = \tan\alpha / \sqrt{(2\pi/g * H_s / T_{m-1,0}{}^{2})} \tag{5.3}$$

Outerslope

Equation 5.4 as derived by Schüttrumpf and Van Gent (2003) gives the relation between the wave run-up velocity and the wave run-up, the wave height and the roughness of the slope. Equation 5.5 gives the relation between the thickness of the water layer and the same wave parameters and roughness.

$$\frac{u_{2\%}}{\sqrt{gH_s}} = c'_{a,u}\left(\frac{R_{u2\%} - z}{\gamma_f H_s}\right)^{0.5} \tag{5.4}$$

$$\frac{h_{2\%}}{H_s} = c'_{a,h}\left(\frac{R_{u2\%} - z}{\gamma_f H_s}\right) \tag{5.5}$$

in which

parameter	short description	unit
H_s	significant wave height of the incident waves at the toe of the structure	[m]
γ	reduction factor γ (= $\gamma_f\,\gamma_\beta$)	[-]
γ_f	roughness factor	[-]
γ_β	correction factor for oblique waves	[-]
$R_{u2\%}$	wave run up height for the 2% wave	[m]
c_0	coefficient	[-]
c_1	coefficient	[-]

parameter	short description	unit
c_2	coefficient	[-]
p	coefficient	[-]
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	[-]
α	slope angle	[º]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
g	Gravitational acceleration	[m/s^2]
$u_{2\%}$	wave run-up velocity on the slope	[m/s]
z	position (vertical height) on the seaward slope relative to SWL	[m]
$h_{2\%}$	thickness of water layer on the slope	[m]
$c'_{a,u}$	coefficient	[-]
$c'_{a,h}$	coefficient	[-]

Crest

The coefficients used in these Equations 5.4 and 5.5 were determined in different model tests; The differences between the results can be explained by different model set-ups and test programmes.
Schüttrumpf *et al* (2002), Van Gent (2002) and Schüttrumpf and Van Gent (2003) use the Equations 5.6 and 5.7 as the expressions to predict the velocities, $u_{2\%}$, and thickness of water layers, $h_{2\%}$, at the crest:

$$\frac{u_{2\%}}{\sqrt{gH_s}} = c'_{c,u}\left(\frac{R_{u2\%}-R_c}{\gamma_f H_s}\right)^{0.5} \cdot \exp(-c''_{c,u}\, x f_c / h_{2\%}) \tag{5.6}$$

$$\frac{h_{2\%}}{H_s} = c'_{c,h}\left(\frac{R_{u2\%}-R_c}{\gamma_f H_s}\right) \cdot \exp(-c''_{c,h}\, x/B) \tag{5.7}$$

in which:

parameter	short description	unit
H_s	significant wave height of the incident waves at the toe of the structure	[m]
γ	reduction factor γ (= $\gamma_f \gamma_\beta$)	[-]
γ_f	roughness factor	[-]
γ_β	correction factor for oblique waves	[-]
$R_{u2\%}$	wave run up height for the 2% wave	[m]
c_0	coefficient	[-]
c_1	coefficient	[-]

parameter	short description	unit
c_2	coefficient	[-]
p	coefficient	[-]
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	[-]
α	slope angle	[º]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
g	Gravitational acceleration	[m/s^2]
$u_{2\%}$	wave run-up velocity at the crest	[m/s]
$h_{2\%}$	thickness of water layer at the crest	[m]
$c'_{c,u}$	coefficient	[-]
$c'_{c,u}$	coefficient	[-]
$c'_{c,h}$	coefficient	[-]
$c'_{c,h}$	coefficient	[-]
R_c	crest freeboard, level of crest relative to still water level	[m]
x	position parameter (with x = 0 at seaward side of the crest)	[m]
B	Structure crest width	[m]

The same coefficients can be used to predict exceedance percentages of 1% or 10% by using the corresponding wave run-up levels in these formulae; the values can be calculated using Equations 5.1 and 5.2 with data provided in Table 5.1. The use of the coefficients proposed by Van Gent (2002) provides in most situations more conservative estimates for velocities at the rear side of the crest than the use of the factors proposed by Schüttrumpf *et al* (2002). The use of the coefficients proposed by Schüttrumpf *et al* (2002) gives in most situations the most conservative estimates for the thickness of water layers.

Innerslope

Van Gent (2002) and Schüttrumpf and Van Gent (2003) proposed the Equations 5.8 and 5.9 as the expressions to be used for the velocities and thickness of water layers at the rear-side:

$$h = h_0 u_0 \,/ (\frac{\alpha'}{\beta} + \mu \exp(-3\,\alpha\beta^2 s)) \tag{5.8}$$

$$u = \frac{\alpha'}{\beta} + \mu \exp(-3\alpha'\beta^2 s) \tag{5.9}$$

where:

$$\alpha' = \sqrt[3]{g \sin \alpha_{rear}} \quad (5.9.1)$$

$$\beta = \sqrt[3]{1/2 f_L / (h_0 u_0)} \quad (5.9.2)$$

$$\mu = u_0 - \alpha / \beta \quad (5.9.3)$$

In Equations 5.8 and 5.9, h_0 and u_0 are obtained from the expressions for $h_{2\%}$ and $u_{2\%}$ at the landward side of the crest as given in Equations 5.6 and 5.7.
With a position on the dike crest (x) that equals B and replacing $h_{2\%}$ and $u_{2\%}$ by h_0 and u_0 the following equations can be obtained:

$$\frac{u_0}{\sqrt{gH_s}} = c'_{c,u} \left(\frac{R_{u2\%} - R_c}{\gamma_f H_s} \right)^{0.5} \cdot \exp(-c''_{c,u} B f_c / h_{2\%}) \quad (5.10)$$

$$\frac{h_0}{H_s} = c'_{c,h} \left(\frac{R_{u2\%} - R_c}{\gamma_f H_s} \right) \cdot \exp(-c''_{c,h}) \quad (5.11)$$

An overview of the used parameters is given below:

parameter	short description	unit
H_s	significant wave height of the incident waves at the toe of the structure	[m]
γ	reduction factor γ (= $\gamma_f \gamma_\beta$)	[-]
γ_f	roughness factor	[-]
γ_β	correction factor for oblique waves	[-]
$R_{u2\%}$	wave run up height for the 2% wave	[m]
c_0	coefficient	[-]
c_1	coefficient	[-]
c_2	coefficient	[-]
p	coefficient	[-]
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	[-]
α	slope angle	[º]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
g	Gravitational acceleration	[m/s^2]
$u_{2\%}$	wave run-up velocity at the rear side	[m/s]
$h_{2\%}$	thickness of water layer at the crest	[m]
$c_{c,u}'$	coefficient	[-]

parameter	short description	unit
$c_{c,u}''$	coefficient	[-]
$c_{c,h}'$	coefficient	[-]
$c_{c,h}''$	coefficient	[-]
R_c	crest freeboard, level of crest relative to still water level	[m]
x	position parameter (with $x = 0$ at seaward side of the crest)	[m]
B	Structure crest width	[m]
s	the coordinate along the landward slope (with $s = 0$ at the landward side of the crest)	[m]
α_{rear}	slope angle at the rear side	[º]
f_L	friction factor at the landward slope	[-]
α'	coefficient	[-]
β	coefficient	[-]
μ	coefficient	[-]
h_0	thickness of water layer at transition of crest and innerslope	[m]
u_0	velocity at the transition of crest and innerslope	[m/s]

5.2 Input and output parameters

Input:	**Output:**
H_s, γ_f, γ_β, c_0, c_1, α, $T_{m-1,0}$, g, z, $c'_{a,u}$, $c'_{a,h}$	$R_{u2\%}$, $\mathbf{h_{2\%}}$, $\mathbf{u_{2\%}}$
H_s, γ_f, γ_β, c_0, c_1, α, $T_{m-1,0}$, g, $c'_{c,u}$, $c_{c,u}''$, $c_{c,h}'$, $c_{c,h}''$, R_c, x, B	$R_{u2\%}$, $\mathbf{h_{2\%}}$, $\mathbf{u_{2\%}}$
H_s, γ_f, γ_β, c_0, c_1, α, $T_{m-1,0}$, g, $c'_{c,u}$, $c_{c,u}''$, $c_{c,h}'$, $c_{c,h}''$, R_c, x, B, s, cot α_{rear}, f_L, α', β, μ	$R_{u2\%}$, u_0, h_0, **u, h**

5.3 Boundary- and default values

Outerslope

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H_s	significant wave height of the incident waves at the toe of the structure	0 – 10 (i)	> 0
γ	reduction factor γ (= $\gamma_f \gamma_\beta$)		> 0
γ_f	roughness factor	see table 1a.2	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
γ_β	correction factor for oblique waves	0,82 – 1 (f)	> 0
$R_{u2\%}$	wave run up height for the 2% wave		> 0
c_0	coefficient	see table 5.1	
c_1	coefficient	see table 5.1	
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	0,5 – 6 (i)	> 0
α	slope angle 1: …	1 - 10 (i)	> 0
$T_{m-1,0}$	spectral wave period, also called the energy wave period	1 – 20 (i)	> 0
z	position (vertical height) on the seaward slope relative to SWL	0,1 – 5 (i)	> 0
$c'_{a,u}$	coefficient	see table 5.2	
$c'_{a,h}$	coefficient	see table 5.2	

Crest

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H_s	significant wave height of the incident waves at the toe of the structure	0 – 10 (i)	> 0
γ	reduction factor γ (= $\gamma_f \gamma_\beta$)		
γ_f	roughness factor	see table 1a.2	> 0
γ_β	correction factor for oblique waves	0,82 – 1 (f)	> 0
$R_{u2\%}$	wave run up height for the 2% wave		> 0
c_0	coefficient	see table 5.1	
c_1	coefficient	see table 5.1	
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	0,5 – 6 (i)	> 0
α	slope angle 1: …	1 - 10 (i)	> 0
$T_{m-1,0}$	spectral wave period, also called the energy wave period	1 – 20 (i)	> 0
$c'_{c,u}$	coefficient	see table 5.2	
$c'_{c,u}$	coefficient	see table 5.2	
$c'_{c,h}$	coefficient	see table 5.2	
$c'_{c,h}$	coefficient	see table 5.2	
R_c	crest freeboard, level of crest relative to still	0,1 – 5 (i)	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
	water level		
x	position parameter (with x = 0 at seaward side of the crest)	0 – B (f)	> 0 ≤ B
B	Structure crest width	0 -20 (i)	> 0

Innerslope

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
H_s	significant wave height of the incident waves at the toe of the structure	0 – 10 (i)	> 0
γ	reduction factor γ (= $\gamma_f \gamma_\beta$)		> 0
γ_f	roughness factor	see table 1a.2	> 0
γ_β	correction factor for oblique waves	0,82 – 1 (f)	> 0
$R_{u2\%}$	wave run up height for the 2% wave		> 0
c_0	coefficient	see table 5.1	
c_1	coefficient	see table 5.1	
$\xi_{s-1,0}$	Surf similarity parameter for mean period wave	0,5 – 6 (i)	> 0
α	slope angle 1: …	1 - 10 (i)	> 0
$T_{m-1,0}$	spectral wave period, also called the energy wave period	0 – 25 (i)	> 0
$c_{c,u}'$	coefficient	see table 5.2	
$c_{c,u}''$	coefficient	see table 5.2	
$c_{c,h}'$	coefficient	see table 5.2	
$c_{c,h}''$	coefficient	see table 5.2	
R_c	crest freeboard, level of crest relative to still water level	0,1 – 5 (i)	> 0
x	position parameter (with x = 0 at seaward side of the crest)	0 – B (f)	> 0 ≤ B
B	structure crest width	0 – 20 (i)	> 0
s	the coordinate along the landward slope (with s = 0 at the landward side of the crest)	0 – 75 (i)	> 0
α_{rear}	slope angle at the rear side 1: …	1 - 10 (i)	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
f_L	friction factor at the landward slope	For smooth slopes the value f_L = 0.02 can be used, for rough slopes the friction factor has a value between 0.1 and 0.6.	> 0

Table 5.1 provides the values of the coefficients c_0 and c_1 for various exceedance levels.

Table 5.1 Coefficients for wave run-up predictions, using H_s and $T_{m-1,0}$ (Equations 5.1 and 5.2)

Run-up level	c_0	c_1
$R_{u1\%}$	1.45	5.1
$R_{u2\%}$	1.35	4.7
$R_{u10\%}$	1.10	4.0

Table 5.2 provides the values of the coefficients $c_{c,u}'$, $c_{a,h}'$, $c_{c,u}'$, $c_{c,u}''$, $c_{c,h}'$ and $c_{c,h}''$ based on both the data by Schüttrumpf *et al* (2002) and on the data by Van Gent (2002).

Table 5.2 Proposed coefficients to be used in the Equations 5.4, 5.5, 5.6 and 5.7

Coefficient	Schüttrumpf	Van Gent
$c_{c,u}'$	1,37	1,30
$c_{a,h}'$	0,33	0,15
$c_{c,u}'$	1,37	1,3
$c_{c,u}''$	0,5	0,5
$c_{c,h}'$	0,33	0,15
$c_{c,h}''$	0,89	0,4

5.4 References

SCHÜTTRUMPF, H. MÖLLER, J. and OUMERACI, H. (2002) Overtopping Flow Parameters on the inner slope of sea dikes, Proc. 28th ICCE, Cardiff, ASCE

SCHÜTTRUMPF, H. and VAN GENT, M. R. A. (2003) Wave overtopping at sea dikes, Proc. Coastal Structures 2003, Portland, ASCE

VAN GENT, M.R.A. (2002) Wave overtopping events at dikes, Proc. 28th ICCE, Cardiff, ASCE

6 Small waves and relatively large freeboards

Structures such as breakwaters constructed with low crest levels will transmit wave energy into the area behind the breakwater. The severity of wave transmission is described by the coefficient of transmission, C_t. The transmission performance of low-crested continuous breakwaters is dependent on the water depth, permeability, wave conditions and the structure geometry, principally the crest freeboard and the crest width. The crest freeboard is defined by the height of the crest above still water level.

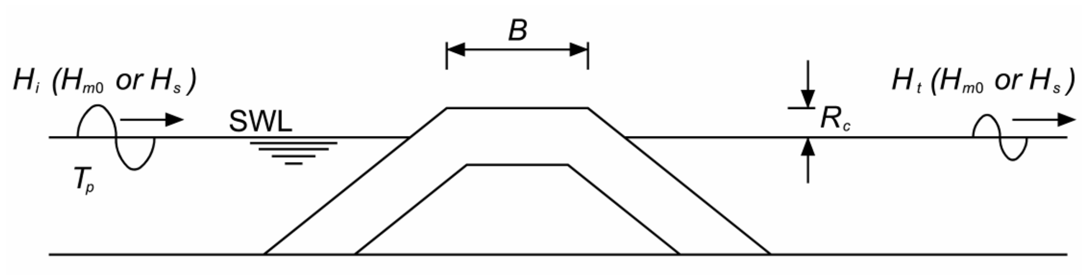

Figure 6.1: Cross-section of breakwater illustrating relevant parameters

6.1 Equations

These formulae yield for transmission of small waves (low values of H_s/D_{n50}) over reef breakwater with relatively large positive freeboards ($R_c/H_s > 1$)

$$C_t = \frac{1.0}{1.0 + X^{0.592}} \tag{6.1}$$

$$C_t = \frac{H_t}{H_i} \tag{6.1.1}$$

$$X = \frac{H_s}{L_P} \cdot \frac{A_t}{D_{n50}^2} \tag{6.1.2}$$

$$L = \frac{gT_m{}^2}{2\pi} \tanh \frac{2\pi h}{L} \tag{6.1.3}$$

$$M_{50} = \rho_s D_{n50}{}^3 \tag{6.1.4}$$

in which:

parameter	short description	unit
A_t	Area of structure cross section	[m^2]
C_t	coefficient of wave transmission	[-]
D_{n50}	nominal mean diameter	[m]
g	gravitational acceleration	[m/s^2]
h	water depth	[m]
H_i	Incident wave height	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
H_t	Transmitted wave height	[m]
L_p	Wave length of peak period	[m]
R_c	crest freeboard, level of crest relative to still water level	[m]
T_m	Mean wave period	[s]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	mass density of stone material	[kg/m^3]

6.2 input and output parameters

Input:	Output:
H_i, H_s, T_m, h, A_t, ρ_s, D_{n50}, M_{50}	**H_t,**

6.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
A_t	Area of structure cross section	10 – 1000 (i)	> 0
C_t	coefficient of wave transmission	0 – 1 (i) > 0.15 (f)	$0 \le C_t \le 1$
D_{n50}	nominal mean diameter	0.02 – 0.05 (i)	> 0
h	water depth	0.5 – 30 (i)	> 0
H_i	Incident wave height	0.1 – 10 (i)	≥ 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i) $R_c/H_s>1$ (f)	≥ 0
H_t	Transmitted wave height	0 – H_s (i)	0 – H_s
R_c	crest freeboard, level of crest relative to still water level	0.1 – 5 (i) $R_c/H_s>1$ (f)	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
ρ_s	mass density of stone material	1500 - 3200	> 0

6.4 References

AHRENS, J. P. (1987) Characteristics of reef breakwaters, Technical Report CERC 87-17 U.S. Army Corps of Eng., Coastal Eng. Res. Center, Vicksburg

7 Smooth low crested structures

These formulae yield for the wave transmission over smooth low crested structures; this also includes the influence of oblique wave attack. The equations are based on a large database on wave transmission and are developed by Van der Meer et al. (2003). The formulae are based on the significant wave height at the toe of the structure and the peak wave period at deep water.

7.1 Equations

For $0.075 \le C_t \le 0.8$ with limitations 1 < ξ_p < 3; 0˚ ≤ β ≤ 70˚; 1 < B/H_s < 4, yields:

$$C_t = \left(\frac{-0.3R_c}{H_s} + 0.75 \cdot \left(1 - e^{-0.5\zeta_p}\right)\right) \cdot \cos^{2/3} \beta \qquad (7.1)$$

$$C_t = \frac{H_t}{H_i} \qquad (7.1.1)$$

$$\xi_p = \frac{\tan \alpha}{\sqrt{\frac{2\pi H_s}{g T_p{}^2}}} \qquad (7.1.2)$$

in which:

parameter	short description	unit
B	structure crest width	[m]
C_t	coefficient of wave transmission	[-]
g	gravitational acceleration	[m/s^2]
H_i	Incident wave height	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
H_t	Transmitted wave height	[m]
R_c	crest freeboard, level of crest relative to still water level	[m]
α	slope angle	[º]
T_p	Peak wave period	[s]
β	Angle of wave attack with respect to the structure	[º]
ξ_p	Surf similarity parameter for peak period wave	[-]

7.2 input and output parameters

Input:	**Output:**
R_c, cot α, T_p, β, H_s, H_i	**H_t**, ξ_p

7.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
B	structure crest width	1 – 100 (i)	> 0
C_t	coefficient of wave transmission	$0.075 \leq C_t \leq 0.8$ (f)	$0 \leq C_t \leq 1$
H_i	Incident wave height	0 – 10 (i)	≥ 0
H_s	significant wave height, average of highest 1/3 of all waves height	0 – 10 (i) $1 > B/H_s > 4$ (f)	≥ 0
H_t	Transmitted wave height	0 – H_s (i)	0 – H_s
R_c	crest freeboard, level of crest relative to still water level	0.1 – 5 (i)	> 0
cot α	slope angle 1 : …	1 – 10 (i)	> 0
T_p	Peak wave period	0.1 – 20 (i)	> 0
β	Angle of wave attack with respect to the structure	$0^\circ \leq \beta \leq 70^\circ$ (f)	$0^\circ \leq \beta \leq 90^\circ$
ξ_p	Surf similarity parameter for peak period wave	$1 < \xi_p < 3$ (f)	> 0

For oblique wave transmission on smooth low-crested structures, the research of Van der Meer et al. (2003) concluded that, for angles up to 45º, the transmitted and incident waves have similar directions. For angles larger than 45º the transmitted wave angle remains 45º, see 7.1.2 and 7.1.3

$$\beta_t = \beta_i \quad \text{for } \beta_i \leq 45^\circ \qquad (7.1.2)$$

$$\beta_t = 45^\circ \quad \text{for } \beta_i > 45^\circ \qquad (7.1.3)$$

In which:

parameter	short description	unit
β_t	Transmitted wave angle	[º]
β_i	Incident wave angle	[º]
β	Angle of wave attack with respect to the structure	[º]

7.4 References

VAN DER MEER J. W., WANG B, WOLTERS A, ZANUTTIGH B and KRAMER M (2003) Wave Transmission behind Low-Crested structures, Proc. Coastal structures 2003, Portland, ASCE

8 Rubble mound low crested structures

Equations 8.1 to 8.3 yield for wave transmission over relatively narrow and over relatively wide submerged rubble mound structures. Briganti et al (2003) used the DELOS database[1] to calibrate a relationship developed by d'Angremond *et al* (1996). This has resulted in two different equations - 8.1 and 8.2. The equations are original developed for perpendicular wave attack, but can also be used for oblique wave attack up to 70º. The performance of these formulae has been evaluated against the database. Equation 8.1 shows a standard deviation of $\sigma = 0.05$, for equation 8.2 and 8.3 the standard deviation is $\sigma = 0.06$.

8.1 Equations

For **narrow** structures, $B/H_i < 10$ and $0.075 \leq C_t \leq 0.8$:

$$C_t = \frac{-0.4R_c}{H_s} + 0.64 \cdot \left(\frac{B}{H_s}\right)^{-0.31} \cdot \left(1 - e^{-0.5\xi_p}\right) \tag{8.1}$$

For **wide** structures, $B/H_i > 10$ and $0.05 \leq C_t \leq C_{t,\max}$:

$$C_t = \frac{-0.35R_c}{H_s} + 0.51 \cdot \left(\frac{B}{H_s}\right)^{-0.65} \left(1 - e^{-0.41\xi_p}\right) \tag{8.2}$$

$$C_{t,max} = -0.006\, B/H_s + 0.93 \tag{8.3}$$

$$C_t = \frac{H_t}{H_i} \tag{8.1.1}$$

$$\xi_p = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{g{T_p}^2}}} \tag{8.1.2}$$

in which:

parameter	short description	unit
C_t	coefficient of wave transmission	[-]
$C_{t,max}$	Maximum value coefficient of wave transmission	[-]
B	structure crest width	[m]
g	gravitational acceleration	[m/s^2]
H_i	Incident wave height	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
H_t	Transmitted wave height	[m]

[1] Acronym for the EU project: Design of low-crested coastal defence structures (2001 – 2004)

parameter	short description	unit
R_c	crest freeboard, level of crest relative to still water level	[m]
α	slope angle	[º]
T_p	Spectral peak period, inverse of peak frequency	[s]
β	Angle of wave attack with respect to the structure	[º]
ξ_p	Surf similarity parameter for peak period wave	[-]

8.2 input and output parameters

Input:	Output:
Narrow structures R_c , B, cot α, T_p, β , H_s , H_i	$\mathbf{H_t}$,, ξ_p
Wide structures R_c , B, cot α, T_p, β , H_s , H_i	$C_{t,\max}$, $\mathbf{H_t}$, ξ_p

8.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
C_t	coefficient of wave transmission	Narrow structure: $0.075 \le C_t \le 0.8$ (f) Wide structure: $0.05 \le C_t \le C_{t,\max}$ (f)	$0 \le C_t \le 1$
B	Crest width of structure	1 – 100 (i) Narrow structure: $B/H_i < 10$ (f) Wide structure: $B/H_i > 10$ (f)	> 0
H_i	Incident wave height	0.1 – 10 (i) Narrow structure: $B/H_i < 10$ (f) Wide structure: $B/H_i > 10$ (f)	≥ 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	≥ 0
H_t	Transmitted wave height	0 – H_s (i)	0 – H_s
R_c	crest freeboard, level of crest relative to	0.1 – 5 (i)	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
	still water level		
T_p	Peak wave period	0.1 – 20 (i)	> 0
cot α	slope angle 1: …	1 - 10 (i)	> 0
β	Angle of wave attack with respect to the structure	0˚ ≤ β ≤ 90˚ (i)	0˚ ≤ β ≤ 90˚
ξ_p	Surf similarity parameter for peak period wave	0.5 – 6 (i)	> 0

The process of wave breaking over low crested structures will tend to reduce the mean wave period (each longer wave breaks to form typically 2-5 shorter waves). With a shorter mean period behind the structure (and possible local refraction effects), the DELOS project suggests (equation 8.3) that the mean obliquity behind the structure, β_t, will be around 0.8 of that in front of the structure, β_i:

$$\beta_t = 0.8\beta_i \tag{8.4}$$

in which:

parameter	short description	unit
β_t	Transmitted wave angle	[º]
β_i	Incident wave angle	[º]

8.4 References

BRIGANTI, R., VAN DER MEER J. W., BUCCINO, M. and CALABRESE, M. (2003) Oblique Wave Transmission over Low-Crested structures, Proc. Coastal structures 2003, Portland, ASCE

d' ANGREMOND, K. VAN DER MEER, J. W. and DE JONG, R. J. (1996) Wave transmission at low-crested structures, Proc. 25th ICCE 1996, Orlando, ASCE

9 Reflection

This equation yields for the wave reflection of regular waves from a smooth non-porous (impermeable) sloping structure. The equation was presented by Seeling and Ahrens (1981).

9.1 Equations

$$C_r = \frac{c \cdot \xi^2}{\left(d + \xi^2\right)} \tag{9.1}$$

$$C_r = \frac{H_{sr}}{H_i} \tag{9.2}$$

$$\xi = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{g T_m^{\,2}}}} \tag{9.3}$$

according to Seeling and Ahrens (1981): c= 1.0 and d=5.5. According to Allsop (1990): *c*=0.96 and *d*=4.80. Allsop did use ξ_m for the breaker parameter.

in which:

parameter	short description	unit
C_r	Coefficient of wave reflection	[-]
ξ	Surf similarity parameter	[-]
c	coefficient	[-]
d	coefficient	[-]
H_i	Incident wave height	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
H_{sr}	Wave height after reflection	[m]
T_m	Mean wave period	[s]
α	slope angle	[º]

9.2 input and output parameters

Input:	Output:
c, d, H_i, H_s, cot α, T_m	C_r, $\mathbf{H_{sr}}$, ξ

9.3 Boundary- and default values

parameter	**short description**	**Indicative (i) or formulae (f) boundary values**	**Mathematical boundary values**
c	coefficient	Seeling and Ahrens (1981): c=1.0 Allsop (1990): c=0.96	> 0
d	coefficient	Seeling and Ahrens (1981): d = 5.5 Allsop (1990): d=4.80	> 0
H_{sr}	Wave height after reflection	0 - H_s (f)	0 - H_s
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
H_i	Incident wave height	0.1 – 10 (i)	> 0
ξ	Surf similarity parameter	0.5 – 6 (i)	> 0
T_m	Mean wave period	0.1 – 20 (i)	> 0
cot α	slope angle 1: …	1 – 10 (i)	> 0

9.4 References

SEELIG, W. N. and AHRENS, J. P. (1981) Estimation of wave reflection and energy coefficients for beaches, revetments and breakwaters, US Army Corps of Eng., Coastal Eng. Res. Center, Fort Belvoir, Technical Paper 81-1

ALLSOP, N.W.H. (1990) Reflection performance of rock armoured slopes in random waves, Proc 22nd ICCE 1990, Delft, ASCE

10 Seepage flow through rock

For estimating the seepage flows through rock structures the use of Darcy's Law (rule 26-1) is no longer appropriate because of the fully developed turbulent seepage that will occur through the rock. These formulae can be used for the determination of the velocity in the voids between the particles of stone. More important, with these formulae the flow rate that can be expected through a rock structure can be computed.

10.1 Equations

The velocity and flow through the rockfill can subsequently be calculated with:

$$U_v = K \cdot C_U^{-0.26} \cdot \left(2g \cdot eD_{50} i\right) \quad (10.1)$$

$$C_u = D_{60} / D_{10} \quad (10.1.1)$$

$$e = n_v / (1 - n_v) \quad (10.1.2)$$

$$Q = U_v n_v A \quad (10.2)$$

in which:

parameter	short description	unit
A	cross-sectional area	[m^2]
C_U	coefficient of uniformity	[-]
D_{10}	Diameter of stone which exceeds the 10% value of sieve curve	[m]
D_{50}	Diameter of stone which exceeds the 50% value of sieve curve	[m]
D_{60}	Diameter of stone which exceeds the 60% value of sieve curve	[m]
e	voids ratio defined as the ratio of volume of the voids and total rockfill volume	[-]
g	gravitational acceleration	[m/s^2]
i	Gradient of (phreatic) water level	[-]
K	coefficient that depends on stone shape	[-]
n_v	Volumetric porosity of the medium	[-]
Q	Discharge through the rockfill	[m^3/s]
U_v	velocity through the voids	[m/s]

10.2 input and output parameters

Input:	Output:
K, D_{50}, D_{60}, D_{10}, i , n_v	$\mathbf{U_v}$, e, C_U
K, A, D_{50}, D_{60}, D_{10}, i , n_v	U_v, e, C_U, $\mathbf{Q}$

10.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
A	cross-sectional area	10 - 1000 (i)	> 0
D_{10}	Diameter of stone which exceeds the 10% value of sieve curve	0.1 – 5 (i)	> 0
D_{50}	Diameter of stone which exceeds the 50% value of sieve curve	0.1 – 5 (i)	> 0
D_{60}	Diameter of stone which exceeds the 60% value of sieve curve	0.1 – 5 (i)	> 0
e	voids ratio defined as the ratio of volume of the voids and total rockfill volume	0 – 6 (i)	> 0
i	Gradient of (phreatic) water level	$1 * 10^{-3} – 1 * 10^{-6}$	> 0
K	coefficient that depends on stone shape	K = 2.48 for crushed stone; (i) K = 3.32 for rounded stones (i)	> 0
n_v	volumetric porosity	0 – 1 (i)	> 0
Q	Discharge through the rockfill	0.1 - 100	> 0

10.4 References

MARTINS, R and ESCARAMEIA, M (1987). Turbulent seepage flow (in Portuguese*), Proc. of 4*^Th^ *Luso-Brazilian Symposium on Hydraulics and Water Resources*, June 1989, Lisbon.

11 Damage to near bed structures

Near-bed structures are submerged structures with a relatively low crest compared with the water depth. The depth of submergence of these structures is enough to assume that wave breaking does not significantly affect the hydrodynamics around the structure.

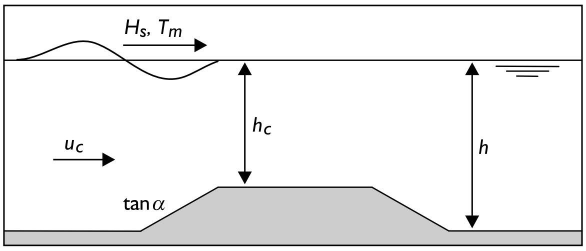

Figure 11.1: Definition sketch of a near-bed rubble mound structure

These formulae focus on the stability of near bed structures under waves, or waves in combination with a flowing current. The parameter predicted in these formulae is one that characterises the amount of material displaced from its original position.

11.1 Equations

To predict the amount of damage a mobility parameter (θ) is used. In these formulae there is no parameter that describes the influence of currents. Although there is an influence of currents on the amount of damage, available data show that this influence can be neglected within the following range: u_c / $\hat{u}_\delta$ < 2.2 for 0.15 < $\hat{u}_\delta^2$ / ($g\Delta D_{n50}$) < 3.5. Neglecting the effects of currents outside this range of currents cannot be justified.

$$\theta = g\left(\frac{1}{0.2}\frac{S}{N^{0.5}}\right)^{\frac{1}{3}} \qquad (11.1)$$

In equation 11.1 the following relations are applicable:

$$u = \hat{u}_\delta = \frac{\pi H_s}{T_m} \cdot \frac{1}{\sinh kh_c} \qquad (11.1.2)$$

$$\theta = \frac{u^2}{g\Delta D_{n50}} \qquad (11.1.3)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (11.1.4)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (11.1.5)$$

$$k = 2\pi / L \qquad (11.1.6)$$

$$L = \frac{gT_m{}^2}{2\pi}\tanh\frac{2\pi h}{L} \qquad (11.1.7)$$

$$S = A_e / D_{n50}^2 \qquad (11.1.8)$$

$$N = 3600 * t_r / T_m \qquad (11.1.9)$$

In which:

parameter	short description	unit
θ	Mobility parameter	[-]
Δ	Relative buoyant density of material of stone material	[-]
A_e	Erosion area on rock profile	[m^2]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s^2}
h	Water depth	[m]
h_c	Armour crest level relative to sea bed	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
k	Wave number	[m^{-1}]
L	Wave length	[m]
N	Number of waves	[-]
S	Stability parameter	[-]
T_m	Mean wave period	[s]
t_r	Duration of storm or wave record	[hrs]
u	Local velocity	[m/s]
$\hat{u}_\delta$	Peak bottom velocity	[m/s]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

11.2 input and output parameters

Input:	Output:
A_e, H_s, h, h_c , T_m, t_r, ρ_w , ρ_s	***D_{n50}***, *M_{50}*, S, N

11.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
θ	Mobility parameter	0 – 10 (i)	> 0
A_e	Erosion area on rock profile	10 – 1000 (i)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
h	Water depth	0.1 – 30 (i)	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
h_c	Armour crest level relative to sea bed	H_s/h_c: 0.2 – 0.9 (f)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i) H_s/h: 0.15 – 0.5 (f)	> 0
N	Number of waves	1 -7500 (i)	> 0
S	Stability parameter	5 – 50 (i)	≥ 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
u	Local velocity	0.1 – 10 (i) $u_c / \hat{u}_\delta < 2.2$ (f)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

11.4 References

Lomónaco, P. (1994), Design of rock cover for underwater pipelines, M.Sc.-thesis International Institute for Infrastructural, Hydraulic and Environmental Engineering, Delft, NL.

Wallast, I. and M.R.A. Van Gent (2002), Stability of near-bed structures under waves and currents, ASCE, Proceedings of 28th International Conference on Coastal Engineering, Cardiff, UK.

12a Submerged structures statically stable

Coastal structures exposed to direct wave attack can be classified by means of the stability number, N_s (=$H/\Delta D_{n50}$).
For submerged structures yields that the crest height lies below the still water level (R_c<0). For the design of submerged structures a distinction is made between statically stable and dynamically stable structures. In the case of submerged structures the waves do not only affect the stability of the front slope but also the stability of the crest and the rear slope. Therefore, the size of the armour units for these segments is more critical in the case of a submerged structure then for a non-overtopped structure.

The armour layer of a submerged can be divided in different segments. Figure 12a.1 shows an example: front slope (I), crest (II) and rear slope (III).

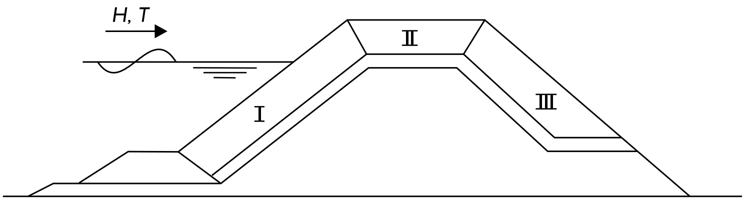

Figure 12a.1 : division of top armour layer in several segments

Statically stable structures are structures where no or minor damage to the armour layer is allowed. The mass of individual units must be large enough to withstand the wave forces during design conditions.

12a.1 Equations

In equation 12a.1 the stability of the armour layer can be determined as a function of the incoming wave height and the wave length.

$$\frac{H_s}{\Delta D_{n50}} = -\frac{1}{0.14}\left(\frac{H_s}{L_p}\right)^{1/3} \ln\left(\frac{d}{h}\frac{1}{\left(2.1+0.1\,S_d\right)}\right) \qquad (12a.1)$$

in which the damage level is approximated by

S=$\frac{H_s}{\Delta D_{n50}}$ (values in table 12a.1) (12a.1.1)

In equation 12a.2 the stability can be determined as a function of the relative crest height based on the ratio R_c/D_{n50}

$$\frac{H_s}{\Delta D_{n50}} = A + B\frac{R_c}{D_{n50}} + C\left(\frac{R_c}{D_{n50}}\right)^2 \qquad (12a.2)$$

in which A, B and C are constants which are shown in table 12a.2

In equation 12a.1 and 12a.2 the following relations are applicable:

$$\Delta = (\rho_s / \rho_w) - 1 \quad (12a.3)$$

$$M_{50} = \rho_s D_{n50}^{\ 3} \quad (12a.4)$$

$$L_p = \frac{gT_p^{\ 2}}{2\pi} \quad (12a.5)$$

$$S_d = A_e / D_{n50}^2 \quad (12a.6)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
A,B,C	coefficients	[-]
A_e	Erosion area on structure	[m^2]
d	Thickness of the armour layer	[m]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s^2}
h	Water depth	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
L_p	Wave length of peak wave period	[m]
R_c	crest freeboard, level of crest relative to still water level	[m]
S	Stability parameter	[-]
S_d	Damage level	[-]
T_p	peak wave period	[s]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

12a.2 input and output parameters

Input:	Output:
Equation 12a.1 H_s, ρ_w, ρ_s, d, h, T_p, A_e	$\mathbf{D_{n50}}$, M_{50}, S, S_d
Equation 12a.1 H_s, ρ_w, ρ_s, R_c, A, B, C	$\mathbf{D_{n50}}$, M_{50}, S

12a.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
A_e	Erosion area on rock profile	10 – 1000 (i)	> 0
d	Thickness of a armour layer	0.05 – 1.0 (i)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
h	Water depth	0.1 – 30 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
R_c	crest freeboard, level of crest relative to still water level	0.1 – 5 (i)	> 0
S	Stability parameter	5 – 50 (i)	$\geq$ 0
S_d	Damage level	0 – 20 (i)	> 0
T_p	peak wave period	1 – 20 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Table 12a.1: approximate S-values for several segments of the breakwater during several damage levels.

Damage level	Front slope	Crest	Back slope	Total section
Initiation of damage	1.0	1.0	0.5	1.5
Iribarren's damage	2.5	2.5	2.0	2.5
Start of destruction	4.0	5.0	3.5	6.5
Destruction	9.0	10.0	-	12.0

The coefficients A, B and C depend on the segment of the breakwater and the damage level. Table 12a.2 shows the coefficients for initiation of damage according to Vidal (1999)

Table 12a.2: fitting coefficients of the stability curves for initiation of damage according to Vidal (1999)

Segment	A	B	C
Front slope	1.831	-0.2450	0.0119
Crest	1.652	0.0182	0.1590
Back slope	2.575	-0.5400	0.1150
Total section	1.544	-0.230	0.053

The coefficients of Vidal (1999) are considered valid for the experimental test conditions done by Vidal (1995) within the range shown in table 12a.3

Table 12a.3: range of validity of the coefficients of Vidal (1999) for equation 12a.2

Parameter	Symbol	Range
Front and rear slope angle	tan α	1:1.5
Relative buoyant density of material	Δ	1.65
Number of waves	N	2600 – 3000
Wave steepness	s_p	0.010 - 0.049
Non-dimensional freeboard	R_c / D_{n50}	-2.01 – 2.41
Non-dimensional crest width	B / D_{n50}	6.0
Non-dimensional structure height	d / D_{n50}	16 - 24
Stability parameter	$H_s/\Delta D_{n50}$	1.1 – 3.7

Kramer and Burcharth (2004) calibrated coefficients from Equation 12a.2 based on 3D physical model tests: A=1.36, B=-0.23 and C=0.06. The range of validity for these values are shown in table 12a.4

Table 12a.4: range of validity of equation 12a.2 with specified values for A, B and C according to Kramer and Burcharth (2004)

Parameter	Symbol	Range
Front and rear slope	tan α	1:1.5
Relative buoyant density of material	Δ	1.65
Number of waves	N	1000
Wave steepness	s_p	0.020 - 0.035
Non-dimensional freeboard	R_c / D_{n50}	-3.1 – 1.5
Non-dimensional crest width	B / D_{n50}	3.1 – 7.7
Non-dimensional structure height	d / D_{n50}	9.1
Angle of wave attack	θ	-20º - 20º
Stability parameter	$H_s/\Delta D_{n50}$	1.2 – 4.8

12a.4 References

Kramer, M. and Burcharth, H. (2004), Design guidelines on low crested structures, 8th draft June 2004, DELOS (in preparation).

Burger, G. (1995), Stability of low-crested breakwaters, Stability of front, crest and rear, Influence of rock shape and gradation, MSc-thesis, WL | Delft Hydraulics (report H1878/H2415) and Delft University of Technology

Vidal, C., Losada, M.A. and Mansard, E.P.D. (1995), Stability of low-crested rubble mound breakwater heads, Journal of Waterway, Port, Coastal and Ocean Engineering, pp 114-122

Vidal, C., Medina, R. and Losada, M.A. (1999), A methodology to assess the armour unit of low-crested and submerged rubble mound breakwaters, Proceedings of the International Conference on Coastal Structures, Vol 2, pp 721-725, Santander, Spain

12b Submerged structures dynamically stable

Coastal structures exposed to direct wave attack can be classified by means of the stability number, N_s (=$H/\Delta D_{n50}$).
For submerged structures yields that the crest height lies below the still water level (R_c<0). For the design of submerged structures a distinction is made between statically stable and dynamically stable structures. In the case of submerged structures the waves do not only affect the stability of the front slope but also the stability of the crest and the rear slope. Therefore, the size of the armour units for these segments is more critical in the case of a submerged structure then for a non-overtopped structure.

The armour layer of a submerged can be divided in different segments. Figure 12a.1 shows an example: front slope (I), crest (II) and rear slope (III).

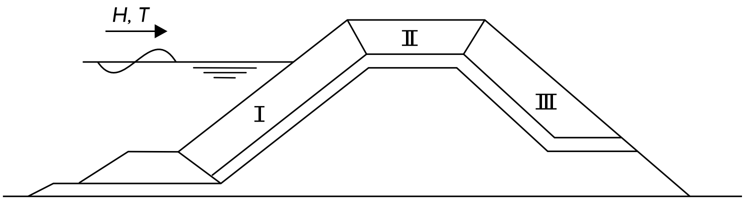

Figure 12a.1 : division of top armour layer in several segments

Dynamically stable structures are reef-type structures consisting of homogeneous piles of stones without a filter layer or core. Some reshaping due to wave action is allowed for these structures. The analysis of stability concentrates on the change in crest height due to the wave attack.

12b.1 Equations

The crest height (*d)* after wave attack can be described by:

$$d = \sqrt{A_t \cdot e^{-aN_s^*}} \qquad (12b.1)$$

if equation 12b.1 lead to d>d_0, then d_0 should be kept equal to d_0.

in which:

$$a = -0.028 + 0.045 C_0 + 0.034 \frac{d_0}{h} - 6 \cdot 10^{-9} B_n^2 \qquad (12b.2)$$

$$N_s^* = N_s \cdot \left(\frac{H_s}{L_p}\right)^{-\frac{1}{3}} = \frac{H_s}{\Delta D_{n50}} \cdot \left(\frac{H_s}{L_p}\right)^{-\frac{1}{3}} \qquad (12b.3)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (12b.4)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (12b.5)$$

$$L_p = \frac{g T_p{}^2}{2\pi} \qquad (12b.6)$$

$$C_0 = A_t / d_0^2 \qquad (12b.7)$$

$$B_n = A_t / D_{n50}{}^2 \qquad (12b.8)$$

$$N_s = H/\Delta D_{n50} \qquad (12b.9)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
A_t	Area of structure cross-section	[m^2]
B_n	Bulk number (cross-section of stones)	[-]
C_0	Response slope as built	[-]
d	Crest height after wave attack	[m]
d_0	Original crest height before wave attack	[m]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s^2]
h	Water depth	[m]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
L_p	Wave length of peak wave period	[m]
N^*_s	Spectral (or modified) stability number	[-]
N_s	Stability number	[-]
T_p	peak wave period	[s]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

12b.2 input and output parameters

Input:	Output:
A_t, H_s, T_p, d_o, h, ρ_s, ρ_w	d, $\mathbf{D_{n50}}$, C_0, B_n, M_{50}, N_s, N^*_s

12b.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
A_t	Area of structure cross-section	10 – 1000 (i)	>0
B_n	Bulk number (cross-section of stones)	200 – 3500	> 0
C_0	Response slope as built	1.5 – 3.0	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
d	Crest height after wave attack	0 – d_0 (i)	< d_0
d_0	Original crest height before wave attack	0.5 – 30 (i) d_0/h = 0.8 – 1.4 (f)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
h	Water depth	0.1 – 30 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
N_s	Stability number	3 – 6	> 0
T_p	peak wave period	1 – 20 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

12b.4 References

Ahrens, J.P. (1987), Characteristics of reef breakwaters. Technical report CERC 87-17, U.S. Army Corps of Engineers, Coastal Eng. Res. Center, Vicksburg

Meer, J.W. van der, (1990), Low-crested and reef breakwaters. WL | Delft Hydraulics, report no. H198/Q638

Meer, J.W. van der, and Pilarczyk, K.W. (1990), Stability of low-crested and reef breakwaters. Proceedings of 22nd ICCE, Vol 2, pp 1375-1388, Delft, The Netherlands

13 Low crested structures

For low-crested structures yields that the crest height lies above the still water level (R_c>0). Due to this a part of the wave energy can pass over the breakwater. Therefore, the size or mass of the material at the front slope of a low-crested structure might be smaller than on a non-overtopped structure. However, because of a larger influence of the waves on the stability of the crest and rear-side of a low-crested structure, the size of the armour units for these segments in the case of overtopped structures is more critical compared to non-overtopped structures.
The stability of a statically stable low-crested structure can be related to the stability of a non-overtopped structure. It is advised to take great care when reducing the armour weight of a low-crested breakwater.

13.1 Equations

General rules are not available yet for low-crested structures. The following rule of thumb can be used to obtain a first estimate of the nominal diameter of the stones in a conceptual design phase:

for $\frac{H_s}{h} < 0.6$ and $\cot\alpha_s > 100$ and Δ = 1.61 yields

$$D_{n50} > 0.33h \qquad (13.1)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (13.1.1)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (13.1.2)$$

in which:

parameter	short description	unit
α_s	Slope angle of the foreshore	[º]
Δ	Relative buoyant density of material	[-]
D_{n50}	Nominal mean diameter	[m]
h	Water depth at toe of structure	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

13.2 input and output parameters

Input:	Output:
h, H_s, ρ_w, ρ_s, *cot* α_s	**D_{n50}**, M_{50}

13.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$\cot \alpha_s$	Foreshore slope angle 1: …	$\cot \alpha_s > 100$ (f)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
h	Water depth at toe of structure	0.1 – 30 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Equation 13.1 is valid for the ranges mentioned in table 13.1.

Table 13.1: Ranges of test conditions used by Burger (1995)

Parameter	Symbol	Range
Front slope angle	$\tan \alpha$	1:1.5
Rear slope angle	$\tan \alpha$	1:2
Relative buoyant density of material	Δ	1.61
Number of waves	N	1000 – 3000
Wave steepness	s_p	0.010 - 0.036
Non-dimensional freeboard	R_c / D_{n50}	-2.9 – 3.0
Non-dimensional crest width	B / D_{n50}	8.0
Non-dimensional structure height	d / D_{n50}	9 - 15
Stability parameter	$H_s / \Delta D_{n50}$	1.4 - 4.0

13.4 References

Burger, G. (1995), Stability of low-crested breakwaters, Stability of front, crest and rear, Influence of rock shape and gradation, MSc-thesis, WL | Delft Hydraulics (report H1878/H2415) and Delft University of Technology

14 Rock armour layers

This section focuses on the stability of rock armour layers on the seaward side of structures under wave attack, such as revetments and breakwaters. These structures have such a crest elevation that the stability of the front slope is not affected by a large amount of wave transmission, wave overtopping, damage to the crest, or damage at the rear-side of the structure. In this section the stability formulae developed by Hudson (1953), van der Meer (1988) and van Gent et al. (2003) will be treated.

14.1 Equations

The original Hudson formula (1953,1959), a stability formula for rock armour structures, was based on model tests with regular waves on non-overtopped rock structures with a permeable core. The formula gives the relationship between the median weight of
Armourstone and the wave height at the toe of the structure and the various relevant structural parameters.

$$W_{50} = \frac{\rho_a g H^3}{K_D \Delta^3 \cot \alpha}$$

in which:
K_D = stability coefficient (-)
ρ_a = armour rock density (kg/m3)
α = angle of armour slope (-).
W_{50}=median weight of armour stone (N)
H = wave height at the toe of the structure (m)
Δ = Relative buoyant density of material (-)
The values given for K_D for rough, angular, randomly placed rock in two layers on a breakwater trunk were K_D = 3.5 for "breaking waves on the foreshore", and K_D = 4.0 for "non-breaking waves on the foreshore".

The original Hudson formula (1953, 1959) did have some limitations:
- the use of regular waves only
- no account of the wave period and the storm duration
- no description of the damage level
- the use of non-overtopped and permeable structures only.

These limitations occurred to generate relatively large differences between predictions and the actual situation. Therefore the original Hudson formula is rewritten in terms of the stability number and using $H_{1/10}$ = 1.27 H_s. The rewritten Hudson formula gives the relationship between this stability parameter and the structure slope and the stability coefficient, K_D. The armour stone size can be calculated by using equation 14.1:

Equation 14.1 yields for damage level, *D*=0-5%, and K_D=2 (breaking waves) or K_D=4 (non-breaking waves).

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D \cot \alpha)^{1/3}}{1.27} \qquad (14.1)$$

in which:

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (14.1.1)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (14.1.2)$$

Equation 14.1 yields for a fixed damage level of 0 to 5%. The armour stone size that yields for higher damage percentages can be determined by making use of table 14.1. It is also possible to derive equation 14.1 such that it can be used for damage levels described by the parameter S_d (van der Meer, 1988).

$$\frac{H_s}{\Delta D_{n50}} = 0.7 \cdot (K_D \cot\alpha)^{1/3} \cdot S_d^{\ 0.15} \quad (14.2)$$

$$S_d = \frac{A_e}{D_{n50}^{\ 2}} \quad (14.2.1)$$

The limits of S_d can be determined by table 14.2. The range of validity of equation 14.2 is given by table 14.3.

The parameters mentioned in equation 14.1 and 14.2 are:

parameter	short description	unit
α	Slope angle	[º]
Δ	Relative buoyant density of material	[-]
A_e	Erosion area on structure	[m²]
D_{n50}	Nominal mean diameter	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
K_D	Stability coefficient	[-]
M_{50}	Mass of a armour unit that is exceeded by 50% of the stones	[kg]
S_d	Damage level	[-]
ρ_s	Mass density of stone material	[kg/m³]
ρ_w	Mass density of water	[kg/m³]

Table 14.1 shows Hs/Hs;D = 0 as a function of the damage percentage D. H_s is the (significant) design wave height corresponding to damage D and H_s;D = 0 is the design wave height corresponding to 0 to 5% damage, generally referred to as “no damage” condition.

Armour type	Relative wave height	Damage D (per cent) [1] with corresponding damage level S_d						
		0-5 (S_d=2)	5-10 (S_d=6)	10-15 (S_d=10)	15-20 (S_d=14)	20-30 (S_d=20)	30-40 (S_d=28)	40-50 (S_d=36)
Quarry stone (smooth)	$H_s/H_{s;D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarry stone (rough)	$H_s/H_{s;D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	1.56 [2]

Table 14.1: *Hs/Hs;D*=0 as a function of armour layer damage and armour type

Notes:
1) All values for breakwater trunk, randomly placed armour in two layers and non-breaking waves on the foreshore
2) Extrapolated value.

The damage levels, Sd , can be characterised as follows:
- start of damage; corresponding to “no damage” (D = 0-5%) in the formula by Hudson (1953, 1959)

- intermediate damage
- failure; corresponding to reshaping of the armour layer such that the filter layer under the armour stone in a double layer is visible.

The limits of S_d depend mainly on the slope angle of the structure. For armour stone in a double layer the values in table 14.2 can be used.

slope	Damage level S_d (-)		
Cot α	**Start of damage**	**Intermediate damage**	**failure**
1.5	2	3 – 5	8
2	2	4 – 6	8
3	2	6- 9	12
4	3	8 – 12	17
6	3	8 – 12	17

Table 14.2: Design values of S_d for armour stone in a double layer

The range of validity of the formula of Van der Meer (1988) is given in table 14.3. The formula yields for deep water conditions with standard single-peaked wave energy spectra at the toe of the structure. 'Deep water' is for the purpose of the validity of these formulae defined as: the water depth at the toe of the structure is larger than 3 times the significant wave height at the toe: htoe > 3Hs-toe (when the water depth at the toe of the structure becomes smaller than 3 times the significant wave height at the toe, the conditions are no longer regarded as deep water).

Parameter	Symbol	Range
Slope angle	$\tan \alpha$	1:6 – 1:1.5
Relative buoyant density of material .	Δ	1 – 2.1 [1)]
Number of waves	N	< 7500
Wave steepness	s_m	0.01 - 0.06
Surf-similarity parameter using *Tm*	ξ_m	0.7 - 7
Relative water depth at the toe	$ht_{oe}/H_{s\text{-}toe}$	> 3 [2)]
Permeability parameter	P	0.1-0.6
Grading of armour rock	D_{n85}/D_{n15}	< 2.5
Stability parameter	$H_s/\Delta D_{n50}$	1 - 4
Damage – storm duration ratio	$S_d/\sqrt{N}$	< 0.9
Damage level	S_d	1 < Sd < 20

Table 14.3: Range of validity of parameters in formulae Van der Meer (1988)

Notes:
1) For higher values of the buoyant density (up to . . 3.5) the validity of the stability formulae is restricted to structures with front slopes with cot á = 2 (see Helgason & Burcharth (2005)
2) This ratio representing the area of research, the range of validity ('for deep water') can also be approximated by: Hs-toe > 0.9Hso (ie hardly any wave breaking)

For deep water conditions Van der Meer (1988) derived formulae to predict the stability of rock armour in uniform rock slopes with crests above the maximum run-up level. These formulae are already described in Cress.
But for shallow water conditions the wave load changes. In shallow water conditions the distribution of the wave heights changes, the shape of the spectrum changes and the wave itself becomes more peaked and skewed.

In order to take the effect of these changes into account, the stability of the armour layer for shallow water conditions would be better described by using the 2% wave height, $H_{2\%}$, than by the significant wave height, H_s. The formulae for deep water conditions of Van der Meer (1988) can be rewritten to formulae yielding for shallow water conditions by making use of the known ratio of $H_{2\%}/H_s = 1.4$.

For plunging conditions $\left(\xi_{s-1,0} < \xi_c\right)$:

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} \cdot P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) \cdot \xi_{s-1,0}^{-0.5} \qquad (14.3)$$

For surging conditions $\left(\xi_{s-1,0} \geq \xi_c\right)$:

$$\frac{H_s}{\Delta D_{n50}} = c_s \cdot P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) \cdot \sqrt{\cot\alpha} \cdot \xi_{s-1,0}^{-0.5} \qquad (14.4)$$

with:

$$\xi_{s-1,0} = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{g\left(T_{m-1,0}\right)^2}}} \qquad (14.4.1)$$

$N = 3600 * t_r / T_m$

(14.4.2)

$$\Delta = \left(\rho_s / \rho_w\right) - 1$$

(14.4.3)

$$M_{50} = \rho_s D_{n50}{}^3$$

(14.4.4)

The transition from plunging to surging waves can be calculated with equation 14.5 using a critical value of the surf similarity parameter.

$$\xi_c = \left[\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} \qquad (14.5)$$

The parameters mentioned in equation 14.3 to 14.5 are:

parameter	**short description**	**Unit**
α	Slope angle	[º]
Δ	Relative buoyant density of material	[-]
A_e	Erosion area on structure	[m²]
C_{pl}	Coefficient for plunging conditions	[-]
C_s	Coefficient for surging conditions	[-]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s²]

parameter	short description	Unit
$H_{2\%}$	wave height exceeded by 2% of the incident waves at the toe	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
K_D	Stability coefficient	[-]
M_{50}	Mass of a armour unit that is exceeded by 50% of the stones	[kg]
N	Number of waves	[-]
P	Permeability parameter	[-]
S_d	Damage level	[-]
T_m	Mean wave period	[s]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
t_r	Duration of storm or wave record	[hrs]
ξ_c	Critical value of the surf-similarity parameter	[-]
$\xi_{s-1,0}$	surf-similarity parameter using the spectral wave period Tm-1,0	[-]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

If no accurate information on the wave period $T_{m\text{-}1,0}$ and the ratio $H_{2\%}/H_s$ is available, the formulae of Van Gent (2003) can be used in stead of the formulae of Van der Meer. The data-set described in Van Gent *et al* (2003) mainly includes conditions with shallow and very foreshores (ie 1.25 < $h_{toe}/H_{s\text{-}toe}$ = 3) and gently-sloping foreshores (1:30 and more gentle). Equation 14.6 leads to almost the same accuracy as equations 14.3 and 14.4. The simple stability formula as derived by Van Gent *et al* (2003):

$$\frac{H_s}{\Delta D_{n50}} = 1.75 \cot\alpha^{0.5} \left(1 + D_{n50-core} / D_{n50}\right) \left(\frac{S}{\sqrt{N}}\right)^{1/5} \qquad (14.6)$$

N =3600*t_r/T_m (14.6.1)

S= A_e / D_{n50}^2 (14.6.2)

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (14.6.3)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (14.6.4)$$

in which:

parameter	short description	Unit
α	Slope angle	[º]
Δ	Relative buoyant density of material	[-]
A_e	Erosion area on structure	[m^2]
D_{n50}	Nominal mean diameter	[m]

parameter	short description	Unit
$D_{n50\text{-}core}$	Nominal mean diameter of core material	[m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
M_{50}	Mass of a armour unit that is exceeded by 50% of the stones	[kg]
N	Number of waves	[-]
S	Stability parameter	[-]
T_m	Mean wave period	[s]
t_r	Duration of storm or wave record	[hrs]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

Table 14.4 shows the range of validity of the various parameters used in the Van der Meer formulae for shallow water conditions (14.3 and 14.4). These ranges of validity yield also for the formula of Van gent (2003).

Parameter	Symbol	Range
Slope angle	tan α	1:4 – 1:2
Number of waves	N	< 3000
Wave steepness based on T_m	s_m	0.01 - 0.06
Surf-similarity parameter using T_m	ξ_m	1 - 5
Surf-similarity parameter using $T_{m-1,0}$	$\xi_{s-1,0}$	1.3 – 6.5
Wave height ratio	$H_{2\%}/H_s$	1.2 – 1.4
Deep water height over depth	H_{50}/h_{toe}	0.25 – 1.5
Grading armour material	D_{n85}/D_{n15}	1.4 – 2.
Core material	$D_{n50\text{-core}}/D_{n50}$	0 – 0.3
Stability parameter	$H_s/\Delta D_{n50}$	0.5 – 4.5
Damage level	S_d	< 30

Table 14.4: Range of validity of Van der Meer formulae for shallow water conditions.

It is not possible to compute cpl and cs, because also the peakedness and skewness of the waves change. Therefore, these coefficients have to be determined using tests with shallow water conditions. On the basis of the tests of Van Gent et al. (2003) one may determine the coefficients cpl and cs by regression analysis. The coefficient for "best fit", 5% and 10% exceedance are given in table 14.5.

Coefficient	Average value, μ	Standard deviation, σ, of the coefficient	Value to determine 5% limit, = μ-1.64σ	Value to determine 10% limit, = μ-1.28σ
c_{pl}	8.4	0.7	7.25	7.5
c_s	1.3	0.15	1.05	1.1

Table 14.5: Coefficients for "best fit, 5% and 10% exceedance limit" for equation 14.3 and 14.4

Equations 14.1 to 14.6 are based on damage occurring during the peak of one, single storm. Especially for maintenance it is sometimes needed to determine the cumulative damage over a number of storms. A method to compute the cumulative damage is presented by Melby (2001).

This formula is based on laboratory tests with a limited range of validity. Important is that the tested range of surf similarity parameters is between 2 and 4 and that the ratio $D_{n50\text{-}armour}/D_{n50\text{-}filter}$ = 2.9.

$$S(t_n) = S(t_0) + a\frac{N_{s0,n}^5}{T_{m,n}^b}\left((3600 \cdot t_n)^b - (3600 \cdot t_0)^b\right) \tag{14.7}$$

$N_s = H_s / \Delta D_{n50}$ (14.7.1)

$$\Delta = (\rho_s / \rho_w) - 1 \tag{14.7.2}$$

$$M_{50} = \rho_s D_{n50}^{\ 3} \tag{14.7.3}$$

In which:

parameter	short description	Unit
Δ	Relative buoyant density of material	[-]
a	coefficient determined in experiments	[-]
b	coefficient determined in experiments	[-]
D_{n50}	Nominal mean diameter	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
M_{50}	Mass of a armour unit that is exceeded by 50% of the stones	[kg]
n	time counter	[-]
N_s	respective stability number based on the significant wave height	[-]
*S(*t0*)*	damage at time *t*0	[-]
S(tn)	damage at time *tn*	[-]
t_0	duration time of storm to reach a damage *S(t*0*)*	[hrs]
T_m	Mean wave period	[s]
t_n	duration time of additional storm	[hrs]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

The influence of **ship-induced waves** on the stability of rock slopes has been investigated by Boeters *et al* (1993). Although wind- and ship-induced waves have much in common, the problem is to define appropriate values for *N*, *H* and *î.* The number of waves (N) is equal to the number of passages of relevant types of ships during the total life time of the structure. The corresponding ship wave is set equivalent to $H_{2\%}$. It is also important to note that damage due to different waves can be superimposed.

$$\frac{H_{2\%}}{\Delta D_{n50}} 8.2 P^{0.18} \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \xi^{-0.5} \tag{14.8}$$

$$\Delta = (\rho_s / \rho_w) - 1 \tag{14.8.1}$$

$$M_{50} = \rho_s D_{n50}{}^3 \tag{14.8.2}$$

$H_{2\%}$ is the maximum of the interference peaks *Hi*

$$H_i = 1.2 \alpha_i h (y_s / h)^{-1/3} V_s{}^4 / (gh) \tag{14.8.1}$$

ξ is based on H_i and L_i,

$$\xi = \frac{\tan \alpha}{\sqrt{\frac{H_i}{L_i}}} \tag{14.8.2}$$

$$L_i = \frac{4 \pi V_s^2}{3g} \tag{14.8.3}$$

Equation 14.9 yields for the influence of **transversal stern waves** on the stability of rock slopes. Equation 14.9 gives the stability relationship between the height of the stern wave, z_{max} (m), and the structural parameters. For design purposes $z_{max} / \Delta D_{n50}$ should be 2 to 3.

$$\frac{z_{\max}}{\Delta D_{50}} = 1.5 (\cot \alpha)^{1/3} \tag{14.9}$$

$D_{n50} = 0.84 D_{50}$ (14.9.1)

In which:

parameter	short description	Unit
α	Slope angle	[º]
α_i	coefficient depending on the type of ship	[-]
Δ	Relative buoyant density of material	[-]
D_{50}	mean diameter	[m]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s^2]
h	water depth	[m]
$H_{2\%}$	wave height exceeded by 2% of the incident waves at the toe	[m]

parameter	short description	Unit
H_i	interference wave height	[m]
L_i	interference wave length	[m]
M_{50}	Mass of a armour unit that is exceeded by 50% of the stones	[kg]
N	the number of passages of relevant types of ships during the total life time of the structure	[-]
P	parameter to take the influence of the permeability of the structure into account	[-]
S_d	Damage level	[-]
V_s	velocity of the ship	[m/s]
y_s	distance to the bank normal to the sailing line	[m]
z_{max}	height of the stern wave	[m]
ξ	surf-similarity parameter	[-]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

14.2 input and output parameters

Equation	Input:	Output:
14.1 & 14.2	$H_s, \rho_w, \rho_s, K_D, \cot\alpha, S_d$	$\mathbf{D_{n50}}$, M_{50}
14.3 & 14.4	$H_s, \rho_w, \rho_s, \cot\alpha, S_d, c_{pl}, c_s, P, t_r, T_m, H_{2\%}, Tm\text{-}1{,}0$	$\mathbf{D_{n50}}$, M_{50}
14.5	α, c_{pl}, c_s, P	$\boldsymbol{\xi_c}$
14.6	$H_s, \rho_w, \rho_s, S, t_r, T_m, \alpha$	$\mathbf{D_{n50}}$, M_{50}
14.7	$S(t_0), H_s, D_{n50}, \rho_w, \rho_s, a, b, t_n, t_o, T_m,$	$\mathbf{S(t_n)}$
14.8	$A_i, h, y_s, V_s, \rho_w, \rho_s, \cot\alpha, S_d, P, N, H_i, \alpha_i$	$\mathbf{D_{n50}}$, M_{50}
14.9	$z_{max}, \rho_w, \rho_s, \cot\alpha$	$\mathbf{D_{n50}}$, M_{50}

14.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
$\cot\alpha$	Armour slope angle 1 : …	1- 10 (i) Table 14.3 or 14.4 (f)	> 0
α_i	coefficient depending on the type of ship	$\alpha_i = 1.0$ for tugs and recreational craft and loaded conventional ships (f)	$\alpha_i > 0$

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
		α_i = 0.35 for unloaded conventional ships (f) α_i = 0.5 for unloaded push units (f)	
a	coefficient determined in experiments	0.025 (f)	*>0*
A_e	Erosion area on structure	10 – 1000 (i)	*>0*
b	coefficient determined in experiments	0.25 (f)	*>0*
C_{pl}	Coefficient for plunging conditions	Table 14.5 (f)	*>0*
C_s	Coefficient for surging conditions	Table 14.5 (f)	*>0*
D_{50}	mean diameter	0.1 - 5 (i)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
$D_{n50\text{-}core}$	Nominal mean diameter of core material	0.1 - 5 (i) Table 14.4 (f)	> 0
h	Water depth	0.1 – 30 (i)	> 0
$H_{2\%}$	wave height exceeded by 2% of the incident waves at the toe	0.1 – 10 (i) deep water: H2% /Hs = 1.4 (f) Table 14.4 (f)	> 0
H_i	interference wave height	0.1 – 10 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
K_D	Stability coefficient	Breaking waves K_D = 2 (f) Non-breaking waves k_D = 4 (f)	*>0*
N	the number of passages of relevant types of ships during the total life time of the structure	N = 2000 (f)	> 0
N	Number of waves	1 – 7500 Table 14.3 or 14.4 (f)	>0
P	Permeability parameter	0.1 – 0.6 (i) Table 14.3	>0
S	Stability parameter	3-12 (i)	>0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
		Table 14.2	
$S(t0)$	damage at time $t0$	1 – 36 (i)	>0
S_d	Damage level	1– 36 (i) Table 14.1, 14.3 or 14.4 (f)	>0
t_0	duration time of storm to reach a damage $S(t0)$	1 – 24 (i)	>0
T_m	Mean wave period	1-20 (i)	>0
$T_{m-1,0}$	a spectral wave period, also called the energy wave period	1-20 (i)	
t_n	duration time of additional storm	1 – 24 (i)	>0
t_r	Duration of storm or wave record	1 – 24 (i)	>0
V_s	velocity of the ship	0.1 – 7.5 (i)	>0
y_s	distance to the bank normal to the sailing line	5 – 500 (i)	>0
z_{max}	height of the stern wave	0.1 – 3 (i)	>0
ξ	surf-similarity parameter	1– 5 (i) Table 14.3 or 14.4 (f)	>0
$\xi_{s-1,0}$	surf-similarity parameter using the spectral wave period Tm-1,0	1– 5 (i) Table 14.3 or 14.4 (f)	>0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

14.4 References

Melby, J. A. (2001): Damage development on stone armoured breakwaters and revetments, ERDC/CHL CHETN-III-64, U.S. Army Engineer Research and Development Center, Vicksburg, MS, USA.

Van Gent, M.R.A. (2005): On the stability of rock slopes, in: Zimmerman *et al* (Ed), Environmentally Friendly Coastal Protection, pp 73-92, NATO Science services, IV – Vol53, Dordrecht, the Netherlands

Van der Meer, J.W., (1988a), Rock slopes and gravel beaches under wave attack. Ph.D. thesis, Delft University of Technology; also Delft Hydraulics Communication No. 396, Delft, The Netherlands

15 Cubes

The design of concrete armour layers generally follows the approach for rock armouring. Various approaches have been developed for concrete armour layer to provide hydraulic stable armour layers. Concrete units obtain their stability in different ways; using their own weight, using interlocking between adjoining units or using friction between the individual units. Cubes obtain their hydraulic stability mainly from their own weight.

15.1 Equations

For wave attack the dimensions of cubes used in a double layer can be calculated from equation 15.1 (Van der Meer, 1988).

For cubes used in a double layer on a 1:1.5 slope with 3 < ξ_m < 6:

$$\frac{H_s}{\Delta D_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_m^{-0.1} \qquad (15.1)$$

For wave attack the hydraulic stability of cubes used in a single layer can be described by equation 15.2 and 15.3 (Van Gent et al., 1999, 2001 and 2003).

$$s_m = 2\pi H_s / g T_m^{\ 2} \qquad (15.1.1)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (15.1.2)$$

$$N = 3600 * t_r / T_m \qquad (15.1.3)$$

$$M = \rho_s D_n^{\ 3} \qquad (15.1.4)$$

Start of damage (N_{od} = 0):

$$\frac{H_s}{\Delta D_n} = 2.0 - 3.0 \qquad (15.2)$$

Failure (N_{od} = 0.2):

$$\frac{H_s}{\Delta D_n} = 3.0 - 3.5 \qquad (15.3)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D_n	Nominal diameter of armour units	[m]
g	Gravitational acceleration	[m/s^2]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
N	Number of waves	[-]

parameter	short description	unit
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
s_m	Wave steepness for mean wave period	[-]
t_r	Duration of storm or wave record	[hrs]
T_m	Mean wave period	[s]
M	Mass of a stone	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

15.2 input and output parameters

Input:	Output:
H_s, ρ_w, ρ_s, N_{od}, t_r, T_m,	**D_n**, M

15.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D_n	Nominal diameter of armour units	0.1 - 5 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
N	Number of waves	1 -7500 (i)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	0.1 – 3 (i) table 15.1 (f)	> 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Typical values of N_{od} and N_d for certain damage levels are listed in table 15.1

Table 15.1: characteristic damage levels for various types of concrete armour units

Armour type	Damage number	Damage level		
		Start of damage	Intermediate damage	Failure
Cube		0.5	–	2
Tetrapod	N_{od}	0.5	–	1.5
Accropode		> 0	–	> 0.5
Cube		–	4%	–
Dolos	N_d	0% – 2%	–	≥ 15%
Accropode		0%	1% – 5%	≥ 10%

15.4 References

Van der Meer, J W (1988) "Stability of Cubes, Tetrapodes and Accropode", *Proceedings of the Breakwaters '88 Conference; Design of Breakwaters*, Institution of Civil Engineers, Thomas Telford, London, UK, pp 71-80

Van Gent, M R A, Spaan, G B H, Plate, S E, Berendsen, E, Van der Meer, J W, D'Angremond, K (1999) "Single-layer rubble mound breakwaters", *Proc International Conference Coastal Structures*, Balkema, Santander, Spain, Vol 1, pp 231-239

Van Gent, M R A, D'Angremond, K and Triemstra, R (2001) "Rubble mound breakwaters; single armour layers and high density units", *Proc Coastlines, Structures and Breakwaters*, ICE, London

Van Gent, M R A (2003) Recent developments in the conceptual design of rubble mound breakwaters, Proc COPEDEC VI, Colombo, Sri Lanka

16 Tetrapods

The design of concrete armour layers generally follows the approach for rock armouring. Various approaches have been developed for concrete armour layer to provide hydraulic stable armour layers. Concrete units obtain their stability in different ways; using their own weight, using interlocking between adjoining units or using friction between the individual units. Tetrapods obtain their hydraulic stability mainly from their own weight and interlocking between adjoining units.

16.1 Equations

For wave attack the dimensions of tetrapods used in a double layer system can be calculated from equation 16.1 (Van der Meer, 1988). The stability decreases with increasing wave steepness.

For tetrapods used in a double layer system on a 1:1.5 slope with 3 < ξ_m < 6 and non-depth limited wave conditions:

$$\frac{H_s}{\Delta D_n} = \left(3.75 \cdot \left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 0.85\right) s_m^{-0.2}$$ (surging waves) (16.1)

$$s_m = 2\pi H_s / g T_m^{\ 2}$$ (16.1.1)

$$\Delta = (\rho_s / \rho_w) - 1$$ (16.1.2)

$N = 3600 * t_r / T_m$ (16.1.3)

$$M = \rho_s D_n^{\ 3}$$ (16.1.4)

For plunging waves yields (De Jong, 1996):

$$\frac{H_s}{\Delta D_n} = \left(8.6 \cdot \left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 3.94\right) s_m^{0.2}$$ (plunging waves) (16.2)

Formulae 15.1, 16.1 and 16.2 regard to almost non-overtopped slopes. De Jong (1996) introduced a factor that increases the stability number with respect to structures with a lower crest height. De Jong (1996) also introduced a coefficient that takes the influence of the packing density on the stability of tetrapods armour layers into account.

$$\frac{H_s}{\Delta D_n} = \left(8.6 \cdot \left(\frac{N_{od}}{\sqrt{N}}\right)^{0.5} + 2.64 k_t + 1.25\right) s_m^{0.2} \left(1 + 1.07 e^{-0.061 \frac{R_c}{D_n}}\right)$$ (plunging waves) (16.3)

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D_n	Nominal diameter of armour units	[m]

parameter	short description	unit
g	Gravitational acceleration	[m/s^2]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
k_t	Layer thickness coefficient	[-]
N	Number of waves	[-]
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
R_c	Crest freeboard, level of crest relative to still water level	[m]
s_m	Wave steepness for mean wave period	[-]
T_m	Mean wave period	[s]
t_r	Duration of storm or wave record	[hrs]
M	Mass of a stone	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

16.2 input and output parameters

Input:	Output:
Plunging waves (Van der Meer, 1988): H_s, ρ_w, ρ_s, N_{od}, t_r, T_m,	 $\mathbf{D_n}$, M
Plunging waves (de Jong, 1996): H_s, ρ_w, ρ_s, N_{od}, t_r, T_m, k_t, R_c	 $\mathbf{D_n}$, M
Surging waves (de Jong, 1996): H_s, Δ, N_{od}, N, H, T_m	 $\mathbf{D_n}$, M

16.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D_n	Nominal diameter of armour units	0.1 - 5 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
k_t	Layer thickness coefficient	Tetrapods: k_t=1.02	
N	Number of waves	1 - 7500 (i)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	0.1 – 3 (i) table 16.1 (f)	> 0
R_c	crest freeboard, level of crest relative to still water level	0.1 – 5 (i)	> 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Typical values of N_{od} and N_d for certain damage levels are listed in table 16.1. More information on the influence of the crest height and packing density for tetrapods is also made by Van der Meer (Pilarczyk ed, 1998).

Table 16.1: characteristic damage levels for various types of concrete armour units

Armour type	Damage number	Damage level		
		Start of damage	Intermediate damage	Failure
Cube		0.5	–	2
Tetrapod	N_{od}	0.5	–	1.5
Accropode		> 0	–	> 0.5
Cube		–	4%	–
Dolos	N_d	0% – 2%	–	≥ 15%
Accropode		0%	1% – 5%	≥ 10%

16.4 References

Van der Meer, J W (1988) "Stability of Cubes, Tetrapodes and Accropode", *Proceedings of the Breakwaters '88 Conference; Design of Breakwaters*, Institution of Civil Engineers, Thomas Telford, London, UK, pp 71-80

De Jong, R J (1996) *Wave transmission at low-crested structure, stability of tetrapods at front, crest and rear of a low crested breakwater*, MSc-thesis, Delft University of Technology

Pilarczyk, K W (ed) (1998) *Dikes and Revetments: Design, Maintenance and Safety Assessment*, AA Balkema, Rotterdam

17 Dolosse

The design of concrete armour layers generally follows the approach for rock armouring. Various approaches have been developed for concrete armour layer to provide hydraulic stable armour layers. Concrete units obtain their stability in different ways; using their own weight, using interlocking between adjoining units or using friction between the individual units. Dolosses obtain their hydraulic stability mainly from interlocking between adjoining units.

17.1 Equations

For wave attack the dimensions of Dolosse used on a non-overtopped slope with $0.32 < r < 0.42$; $0.61 < \phi < 1$ yields (Burchart and Liu (1992):

$$\frac{H_s}{\Delta D_n} = (17 - 26r)\phi^{2/3} N_{od}^{1/3} N^{-0.1} \tag{17.1}$$

$$\Delta = (\rho_s / \rho_w) - 1 \tag{17.1.2}$$

$$N = 3600 * t_r / T_m \tag{17.1.3}$$

$$M = \rho_s D_n^{\ 3} \tag{17.1.4}$$

for Dolosse with packing densities in the range of $0.83 < \phi < 1.15$ (Holtzhausen, 1996):

$$N_{od} = 6.95 \cdot 10^{-5} \left(\frac{H_s}{\Delta^{0.74} D_n} \right)^7 \phi^{1.51} \tag{17.2}$$

Equation 17.2 implies that if the packing ratio decreases, the damage will also decrease. This means that lower packing densities are more stable than higher densities. Holtzhausen (1996) present the following approximation of the damage number for Dolosse at failure (for $\phi < 1.15$):

$$N_{od\_f} = 10.87\phi - 6.2 \tag{17.3}$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
ϕ	packing density	[-]
D_n	Nominal diameter of armour units	[m]
g	Gravitational acceleration	[m/s^2]
H_s	significant wave height, average of highest 1/3 of all waves height	[m]
N	Number of waves	[-]
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
N_{od_f}	damage number at failure.	[-]

parameter	short description	unit
r	Dolos waist ratio	[-]
T_m	Mean wave period	[s]
t_r	Duration of storm or wave record	[hrs]
M	Mass of a stone	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

17.2 input and output parameters

Input:	Output:
H_s, ρ_w, ρ_s, N_{od}, t_r, T_m, r , ϕ	**D$_n$**, M
ϕ	$\boldsymbol{N_{od\_f}}$

17.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
ϕ	packing density		
D_n	Nominal diameter of armour units	0.1 - 5 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
N	Number of waves	1 -7500 (i) If N >3000 then use N = 3000 (f)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	0.1 – 3 (i) table 17.1 (f)	> 0
N_{od_f}	damage number at failure.	0.5 – 3 (i)	> N_{od}
r	Dolos waist ratio		
T_m	Mean wave period	1 – 20 (i)	> 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Typical values of N_{od} and N_d for certain damage levels are listed in table 17.1

Table 17.1: characteristic damage levels for various types of concrete armour units

Armour type	**Damage number**	**Damage level**		
		Start of damage	Intermediate damage	Failure
Cube		0.5	–	2
Tetrapod	N_{od}	0.5	–	1.5
Accropode		> 0	–	> 0.5
Cube		–	4%	–
Dolos	N_d	0% – 2%	–	≥ 15%
Accropode		0%	1% – 5%	≥ 10%

17.4 References

Holtzhausen, A H (1996) *Effective use of concrete for breakwater armour units*, PIANC Bulletin No 90, pp 23–28

Burcharth, H F and Liu, Z (1992) "Design of Dolos Armour Units", *Proc 23rd ICCE*, ASCE, Vol 1, pp 1053-1066

18 Accropode and xBlocs

The design of concrete armour layers generally follows the approach for rock armouring. Various approaches have been developed for concrete armour layer to provide hydraulic stable armour layers. Concrete units obtain their stability in different ways; using their own weight, using interlocking between adjoining units or using friction between the individual units. Accropodes and Xblocs obtain their hydraulic stability mainly from interlocking between adjoining units.

An Xbloc is a simple and robust breakwater armour unit. The units have great hydraulic stability in the armour layer. Results of tests show that the stability coefficients of the Xblocs are the same for non-breaking and breaking waves. For more information about the xbloc, see www.xbloc.com.

18.1 Equations

Accropode and Xbloc are designed for zero damage under design conditions. The Accropode and Xbloc armour should be placed on 1:1.5 or 1:1.33 slopes. If necessary the units can be also applied on 1:2 slopes. The hydraulic stability of accropods and Xblocs are not influenced by storm duration or wave period. The following stability numbers shall be used for concept design :

- Accropode: (18.1)

$H_s/\Delta D_n$ = 2.7 and 2.5 for trunk sections (non-breaking and breaking waves)
$H_s/\Delta D_n$ = 2.5 and 2.3 for roundhead (non-breaking and breaking waves)

- Xbloc: (18.2)

$H_s/\Delta D_n$ = 2.8 for trunk sections (non-breaking and breaking waves)
$H_s/\Delta D_n$ = 2.6 for roundhead (non-breaking and breaking waves).

In which:

$$\Delta = (\rho_s / \rho_w) - 1 \quad (18.2.1)$$

$$M = \rho_s D_n^3 \quad (18.2.2)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D_n	Nominal diameter of armour units	[m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
M	Mass of a stone	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

Note that that the term “breaking waves” does not refer to wave breaking on the breakwater slope but to a depth limited situation at the breakwater toe (i.e. wave breaking on the foreshore and depth limited waves at the structure).
For steep foreshore a 10% reduction of the stability should be considered as a rough guidance.
The stability of Xbloc and Accropode has been tested only for typical concrete densities (about 2300 – 2400 kg/m^3). The hydraulic stability of high density single layer armour units is uncertain and most probably not correctly predicted by the stability number. As the stability depends mainly on interlocking (and not on armour unit weight) the effect of high density concrete might be overestimated

Table 18.1: design values for the stability numbers for accropodes and xblocs

Armour type	**Stability number $H_s/\Delta D_n$**				**References / remarks**
	Trunk		**Head**		
	Non-breaking waves	Breaking waves	Non-breaking waves	Breaking waves	
Accropode	2.7 (15)	2.5 (12)	2.5 (11.5)	2.3 (9.5)	Sogreah (2000) [7,8)]
Xbloc	2.8 (16.0)		2.6 (13.0)		DMC (2003) [7,8)]
[7)] in brackets: corresponding Hudson stability coefficient (K_D) for a 1:1.33 slope					
[8)] stability does not increase on slopes gentler than 1:2, a further 10% reduction of stability numbers is recommended for situations with depth-limited wave heights in combination with steep foreshore slopes					

18.2 input and output parameters

Input:	**Output:**
Accropods and Xblocs:	
H_s, ρ_w, ρ_s	$\mathbf{D_n}$, M

18.3 boundary- and default values

parameter	**short description**	**Indicative (i) or formulae (f) boundary values**	**Mathematical boundary values**
D_n	Nominal diameter of armour units	0.1 - 5 (i)	> 0
H_s	significant wave height, average of highest 1/3 of all waves height	0.1 – 10 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

18.4 References

Van der Meer, J W (1988) "Stability of Cubes, Tetrapodes and Accropode", Proceedings of the Breakwaters '88 Conference; Design of Breakwaters, Institution of Civil Engineers, Thomas Telford, London, UK, pp 71-80

Muttray, M.; Reedijk, J.; Vos-Rover, I.; Bakker, P. (2005): Discussion of paper “Placement and structural strength of Xbloc and other single layer armour units”. Proc. of the Int. Conf. on Coastlines, Structures and Breakwaters 2005, ICE, London, UK.

Vincent, G.; Tourmen, L.; Vara, J.G. (1989): Diques maritimos. Revistas de Obras Publicas, June 1989, pp. 457 – 466.

www.xbloc.com

19 Berm breakwater

Berm breakwater can be characterized by an initial berm that is allowed to reshape, either during storm conditions or only during conditions exceeding the design conditions.

19.1 Equations

The original equation of Hall and Kao (1991) is converted. First the principal design parameter (B=rec) was related to wave climate, stone size, grading and shape. In the newly derived equation Hall and Kao expressed the parameter, Rec, in terms of de nominal diameter in stead of the sieve sizes D.

For slopes 1:1.5 to 1:2 and stability numbers in the range of 2 < $H_s/\Delta D_{n50}$ < 5 and N=3000:

$$\frac{\mathrm{Rec}}{D_{n50}} = 12.4 + 0.39\left(\frac{H_s}{\Delta D_{n50}}\right)^{2.5} + 8.95\left(\frac{D_{n85}}{D_{n15}}\right) - 1.27\left(\frac{D_{n85}}{D_{n15}}\right)^2 - 7.3R_p \qquad (19.1)$$

$$D_n / D = 0.84 \qquad (19.1.1)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (19.1.2)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (19.1.3)$$

The time correction factor for the duration of a storm (number of waves) is defined as a function of the relative number of waves (N/3000):

$$\frac{\mathrm{Rec}_N}{\mathrm{Rec}_{3000}} = 1 + 0.111\ln(N/3000) \qquad (19.1.4)$$

$$N = 3600 * t_r / T_m \qquad (19.1.5)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D	Diameter of armour units	[m]
D_{n50}	Nominal mean diameter	[m]
D_{n15}	Nominal mean diameter which exceeds the 15% value of the sieve curve	[m]
D_{n85}	Nominal mean diameter which exceeds the 85% value of the sieve curve	[m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
N	Number of waves	[-]
Rec	Width of berm eroded (B=Rec)	[m]
Rec_{3000}	Width of berm eroded by 3000 waves	[m]

parameter	short description	unit
Rec_n	Width of berm eroded by N waves	[m]
R_p	Fraction of rounded stones in armour	[-]
T_m	Mean wave period	[s]
t_r	Duration of storm or wave record	[hrs]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

19.2 input and output parameters

Input:	Output:
Rec, t_r, T_m, R_p, ρ_w , ρ_s, H_s, D_{n15}, D_{n85}, D_{n50},	**Rec**, M_{50}, Rec_N

19.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D	Diameter of armour units	0.1 - 5 (i)	> 0
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
D_{n15}	Nominal mean diameter which exceeds the 15% value of the sieve curve	0.1 – 2 (i)	> 0
D_{n85}	Nominal mean diameter which exceeds the 85% value of the sieve curve	1 – 5 (i)	> 0
H_s	Significant wave height	0.1 – 10 (i)	> 0
N	Number of waves	1000 -7500 (i)	> 0
Rec	Width of berm eroded (B=Rec)	1 – 1000 (i)	> 0
Rec_{3000}	Width of berm eroded by 3000 waves	1 – 1000 (i)	> 0
Rec_n	Width of berm eroded by N waves	1 – 1000 (i)	> 0
R_p	Fraction of rounded stones in armour	0 - 1	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

19.4 References

Hall, K and Kao, S (1991): A study of the stability of dynamically stable breakwaters. Canadian Journal of Civil Engineering, Vol. 18, pp 916 – 925.

20 Reshaping berm breakwaters

Tørum noticed that for a given berm breakwater all the reshaped profiles intersected with the original berm at almost a fixed point *A*, at a distance h_f below SWL (figure 20.1)

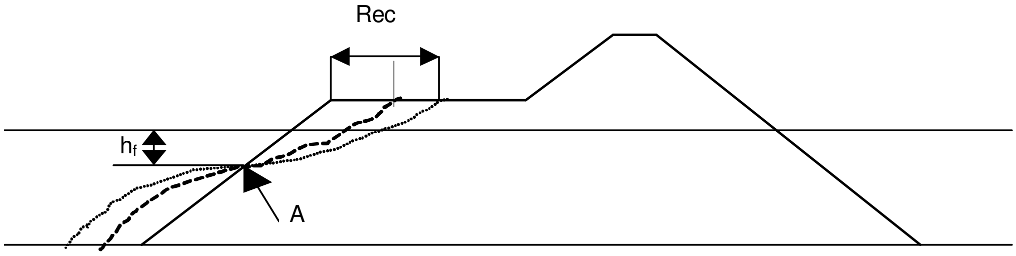

Figure 20.1: Recession on a reshaping berm breakwater

20.1 Equations

For the range: 12.5 < d/D_{n50} < 25, the distance h_f can be obtained from:

$$\frac{h_f}{D_{n50}} = 0.2\frac{d}{D_{n50}} + 0.5 \qquad (20.1)$$

$$M_{50} = \rho_s D_{n50}{}^3 \qquad (20.1.1)$$

in which:

parameter	short description	unit
d	Depth in front of the berm breakwater	[m]
D_{n50}	Nominal mean diameter	[m]
g	Gravitational acceleration	[m/s^2]
h_f	Distance between point A and SWL	[m]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]

Equation 20.2 gives the relationship between the dimensionless recession Rec/D_{n50} and the period stability number *HoTo.*

$$\frac{\mathrm{Re}\,c}{D_{n50}} = 2.7\cdot10^{-6}(H_0T_0)^3 + 9\cdot10^{-6}(H_0T_0)^2 + 0.11(H_0T_0) - f(f_g) - f(\frac{d}{D_{n50}}) \qquad (20.2)$$

$$H_0 = \frac{H_s}{\Delta D_{n50}} \qquad (20.2.1)$$

$$T_0 = T_z\sqrt{\frac{g}{D_{n50}}} \qquad (20.2.2)$$

for $1.3 < f_g < 1.8$:

$$f(f_g) = 9{,}9\ f_g^2 + 23.9\ f_g - 10.5 \qquad (20.2.3)$$

$$f_g = D_{n85}/D_{n15}$$

for $12.5 < d/D_{n50} < 25$

$$f(d/D_{n50}) = 0.16\ (d/D_{n50}) + 4.0 \qquad (20.2.4)$$

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
T_z	zero-cossing period	[s]
H_0	Dimensionless wave height	[-]
T_0	Dimensionless wave period	[-]
H_0T_0	Period stability number	[-]
d	Depth in front of the breakwater	[m]
d/D_{n50}	Depth factor function	[-]
D_{n50}	Nominal mean diameter	[m]
D_{n15}	Nominal mean diameter which exceeds the 15% value of the sieve curve	[m]
D_{n85}	Nominal mean diameter which exceeds the 85% value of the sieve curve	[m]
$f(f_g)$	gradation factor function	[-]
f_g	gradation factor	[-]
g	Gravitational acceleration	[m/s^2]
Rec	Width of berm eroded	[m]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]

20.2 input and output parameters

Input:	Output:
d, g, ρ_s, h_f,	$\mathbf{D_{n50}}$, M_{50}
d, D_{n50}, ρ_s	$\mathbf{h_f}$, M_{50}
H_0, T_0, D_{n50}, D_{n15}, D_{n85}, d	***Rec***

20.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
d	Depth in front of the breakwater	0.5 – 30 (i)	> 0
d/D_{n50}	Depth factor function	$12.5 < d/D_{n50} < 25$ (f)	
D_{n50}	Nominal mean diameter	0.1 - 5 (i)	> 0
D_{n15}	Nominal mean diameter which exceeds the 15% value of the sieve curve	0.1 – 2 (i)	> 0
D_{n85}	Nominal mean diameter which exceeds the 85% value of the sieve curve	1 – 5 (i)	> 0
f_g	gradation factor	$1.3 < f_g < 1.8$ (f)	
h_f	Distance between point A and SWL	0 – d (i)	< d
H_oT_o	Period stability number	10 – 80 (i) Figure 20.2 (f)	> 0
Rec	Width of berm eroded (B=Rec)	1 – 1000 (i)	> 0

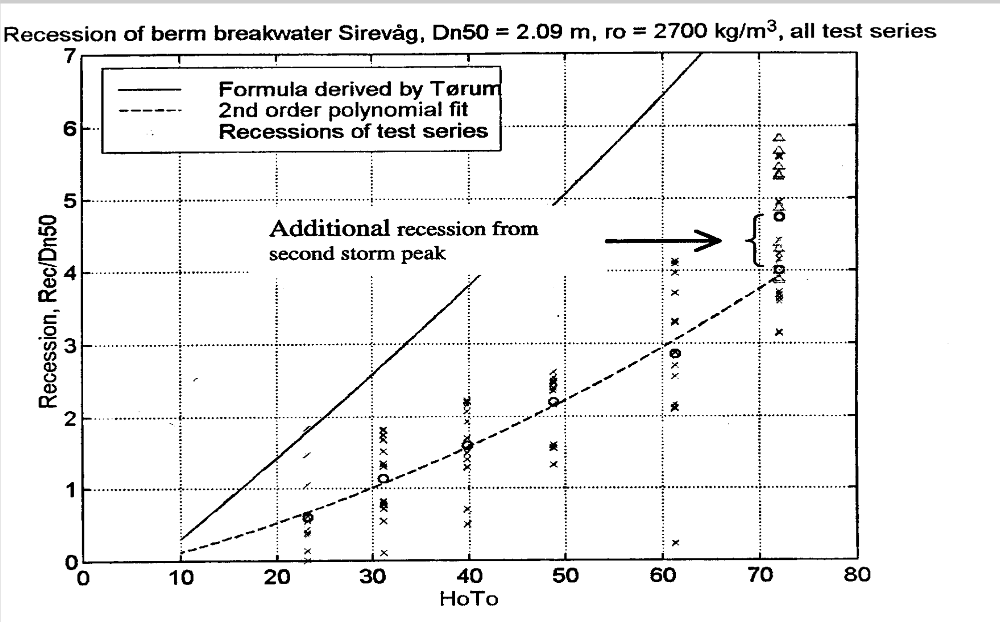

figure 20.2 relation between dimensionless recession and H_oT_o (Tørum, 1998, eq. 20.2) with f_g=1.8

20.4 References

Tørum, A. (1998): On the stability of berm breakwaters in shallow and deep waters. Proc. 26th International Conference on Coastal Engineering, Copenhagen, Denmark. ASCE, 1998.

Tørum, A. , Kuhnen, F. and Menze, A. (2003): On berm breakwaters. Stability, scour, overtopping. Coastal Engineering, 49, (2003), pp 209 – 238.

21 Toe stability

21.1 Equations

The stability of the toe of a structure can be derived in two ways;
with the dimensionless toe depth expressed as h_t/h in the range $0.4 < h_t/h < 0.9$

$$\frac{H_s}{\Delta D_{n50}} = \left(2 + 6.2 \left(\frac{h_t}{h} \right)^{2.7} \right) N_{od}^{0.15} \tag{21.1}$$

or with the dimensionless toe depth expressed as h_t/D_{n50} in the range $3 < h_t/D_{n50} < 25$

$$\frac{H_s}{\Delta D_{n50}} = \left(1.6 + 0.24 \left(\frac{h_t}{D_{n50}} \right) \right) N_{od}^{0.15} \tag{21.2}$$

in which:

$$\Delta = (\rho_s / \rho_w) - 1 \tag{21.2.1}$$

$$M_{50} = \rho_s D_{n50}{}^3 \tag{21.2.2}$$

parameter	**short description**	**unit**
Δ	Relative buoyant density of material	[-]
D_{n50}	nominal mean diameter of armour units	[m]
g	Gravitational acceleration	[m/s^2]
h	Water depth (in front of toe)	[m]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
h_t	Water depth (at structure toe)	[m]
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

21.2 input and output parameters

Input:	**Output:**
$0.4 < h_t/h < 0.9$: ρ_s, ρ_w, N_{od}, h_t, H_s	$\boldsymbol{D_{n50}}$, M_{50}
$3 < h_t/D_{n50} < 25$: ρ_s, ρ_w, N_{od}, h_t, H_s, h	$\boldsymbol{D_{n50}}$, M_{50}

21.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D_{n50}	nominal mean diameter of armour units	0.1 – 5 (i)	> 0
h	Water depth (in front of toe)	0.5 – 30 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
h_t	Water depth at structure toe	0.5 – 30 (i)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	N_{od}=0.5 → start of damage N_{od}=2 → some flattening out N_{od}=4 → complete flattening out of toe	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

21.4 References

Van der Meer, J.W., D'Angremond, K., Gerding, E. (1995): Toe structure stability of rubble mound breakwaters; in: Clifford, J.E. [Ed.]: Advances in coastal structures and breakwaters, ISBN 0-7277-2509-2, Thomas Telford, London, UK.

22 Toe in front of Caison breakwater

22.1 Equations

The design of a toe protection in front of vertical structures requires lower toe stability numbers then needed for toe protection in front of sloping structures. The relation between the stability number and the structural parameters is given by equation 22.1.

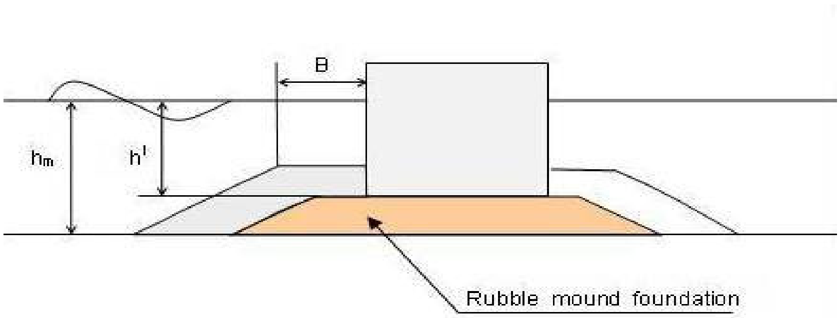

Figure 22.1: definition sketch of toe protection of a caisson breakwater

For $0.5 < h'/h_m < 0.8$ or $7.5 < h'/D_{n50} < 17.5$:

$$\frac{H_s}{\Delta D_{n50}} = \left(5.8\frac{h'}{h_m} - 0.6\right) \cdot N_{od}^{0.19} \qquad (22.1)$$

$$\Delta = (\rho_s / \rho_w) - 1 \qquad (22.1.1)$$

$$M_{50} = \rho_s D_{n50}^3 \qquad (22.1.2)$$

The berm width, *B*, should comply with the rule:

$0.30 < B/h_m < 0.55$ (22.1.3)

The depth of the toe berm, h_B, is defined as:

$h_B = h' - 2\ D_{n50}$ (22.1.4)

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D_{n50}	nominal mean diameter of the armour stone covering the rubble mound foundation material	[m]
g	Gravitational acceleration	[m/s^2]
h' / h_m	relative depth of the rubble mound foundation	[-]
h'	Water depth at the crest of the rubble mound foundation	[m]
h_B	Water depth of the toe berm	[m]
h_m	Water depth in front of the structure	[m]

parameter	short description	unit
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

22.2 input and output parameters

Input:	Output:
ρ_s, ρ_w, N_{od}, h_m, h', H_s	$\boldsymbol{D_{n50}}$, M_{50}

22.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D_{n50}	Nominal mean diameter, size of the armour stone covering the rubble mound foundation material	0.1 – 5 (i)	> 0
h' / h_m	relative water depth of the rubble mound foundation	$0.5 > h' / h_m < 0.8$	> 0
h'/D_{n50}	Non-dimensional water depth at the crest of the rubble mound foundation	$7.5 > h' / h_m < 17.5$	> 0
h'	Water depth at the crest of the rubble mound foundation	0.5 - h_s (i)	$0 < h' < h_s$
h_B	Water depth of the toe berm	0.5 – 30 (i)	> 0
h_m	Water depth in front of the structure	0.5 – 30 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	N_{od}=0.5 → almost no damage N_{od}=2 → acceptable damage N_{od}=5 → failure	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

22.4 References

Madrigal, B. G., and Valdés, J. M. 1995. “Study of Rubble Mound Foundation Stability,” Proceedings of the Final Workshop, MAST II, MCS-Project.

23 Rear side stability of a breakwater

23.1 equation

As long as breakwater are high enough to prevent overtopping, the armour on the crest and rear side can be smaller then on the front site. But when overtopping occurs the crest and rear side have top be protected against wave attack as well. The required size of material at the rear side of coastal and marine structures for a given amount of acceptable damage, can be estimated with equation 23.1 (Van Gent and Pozueta, 2004):

$$D_{n50} = \left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\sqrt{\Delta}}\right)(\cot\alpha_{rear})^{-2.5/6}\left(1+10\exp\left(-R_{c,rear}/H_s\right)\right)^{-1/6} \quad (23.1)$$

$$u_{1\%} = 1.7\left(g\gamma_{f-c}\right)^{0.5}\left(\left(R_{u1\%}-R_c\right)/\gamma_f\right)^{0.5}/\left(1+0.1B/H_s\right) \quad (23.1.1)$$

$$M_{50} = \rho_s D_{n50}^{\ 3} \quad (23.1.2)$$

$$S_d = A_e / D_{n50}^{\ 2} \quad (23.1.3)$$

$$\xi_{s,-1} = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{gT_{m-1,0}^{\ 2}}}} \quad (23.1.4)$$

$$\Delta = (\rho_s/\rho_w) - 1 \quad (23.1.5)$$

$$N = 3600 * t_r / T_m \quad (23.1.6)$$

The maximum velocity ($u_{1\%}$) is related to the rear-side of the crest for situations with $R_{u1\%} \geq R_c$, in which the (fictitious) run-up level $R_{u1\%}$ is obtained using the following expression (Van Gent, 2002):

for $\xi_{s,-1} \leq p$

$$R_{u1\%}/(\gamma H_s) = c_0\xi_{s,-1} \quad (23.2)$$

for $\xi_{s,-1} \geq p$

$$R_{u1\%}/(\gamma H_s) = c_1 - c_2/\xi_{s,-1} \quad (23.3)$$

$c_0 = 1.45$, $c_1 = 5.1$, $c_2 = 0.25\, c_1^2 / c_0$ and $p = 0.5\, c_1 / c_0$ (23.3.1)

$\gamma = \gamma_f\, \gamma_\beta$ (23.1.2)

$\gamma_\beta = 1 - 0.0022 \cdot \beta$ where $\beta \leq 80°$ (23.1.3)

in which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
β	Angle of wave attack with respect to the structure	[º]
γ	reduction factor	[-]

parameter	short description	unit
γ_β	Reduction factor for angular wave attack	[-]
γ_f	Reduction factor for roughness of the seaward slope	[-]
$\gamma_{f\text{-}c}$	Reduction factor for roughness at the crest	[-]
α_{rear}	angle of the rear side slope	[°]
$\xi_{s,-1,0}$	Surf similarity parameter using the spectral wave period $T_{m-1,0}$	[-]
A_e	Erosion area on structure	[m^2]
B	Structure crest width	[m]
c_0 , c_1 , c_2	coefficients	[-]
D_{n50}	Nominal mean diameter (material on the rear side slope)	[m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
N	Number of waves	[-]
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	[-]
p	coefficient	[-]
R_c	crest freeboard, level of crest relative to still water level	[m]
$R_{c,rear}$	crest freeboard relative to the water level at rear side of the crest	[m]
$R_{u1\%}$	fictitious run-up level	[m]
S_d	Damage level	[-]
T_m	Mean wave period	[s]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
t_r	Duration of storm or wave record	[hrs]
$u_{1\%}$	maximum velocity (depth-averaged) at the rear side of the crest (m/s) during a wave overtopping event, exceeded by 1% of the incident waves	[m/s]
M_{50}	Mass of a stone that is exceeded by 50% of the stones	[kg]
α	slope angle	[º]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

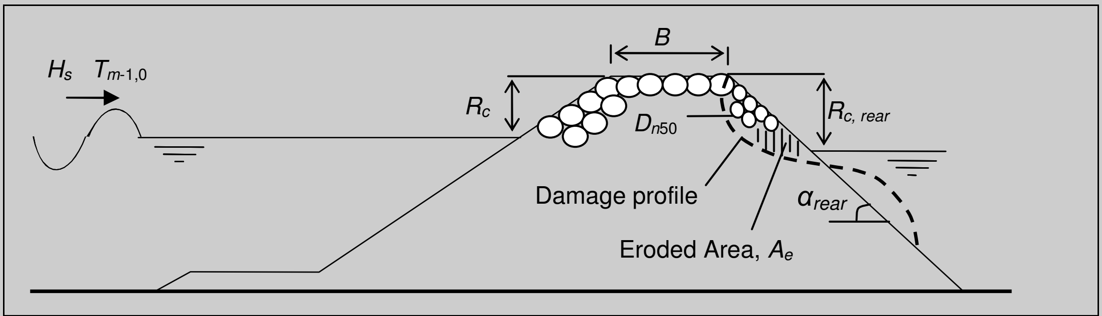

Figure 23.1 definition sketch

Table 23.1: Range of validity of equation 23.1

Parameter	**Range**
Wave steepness at toe: $s_{m-1,0}$ $(s_{m-1,0} = 2\pi/g \cdot H_s/T_{m-1,0}^2)$	0.019 – 0.036
Number of waves, N	< 4000
Relative freeboard at the seaward side, R_c/H_s	0.3 – 2.0
Relative freeboard at the rear side, $R_{c,rear}/H_s$	0.3 – 6.0
Relative crest width, B/H_s	1.3 – 1.6
Relative crest level w.r.t. run-up level, $(R_{u1\%}-R_c)/(\gamma H_s)$	0 - 1.4
Stability number, $H_s/(\Delta D_{n50})$	5.5 – 8.5
Rear side slope (V:H)	1:4 – 1:2
Damage level or parameter, S_d	2 - 30

23.2 input and output parameters

Input:	**Output:**
γ_f, β, H_s, c_0, c_1, c_2, p, $T_{m-1.0}$, $cot\ \alpha$, $cot\ \alpha_{rear}$	$\boldsymbol{R_{u1\%}}$, $\xi_{s,-1}$
B, $R_{u1\%}$, γ_{f-c}, γ_f, H_s, R_c	$\mathbf{u_{1\%}}$
S_d, N, H_s, $T_{m-1.0}$, α_{rear}, $R_{c,rear}$, $u_{1\%}$, ρ_w, ρ_s	D_{n50}, M_{50}

23.3 boundary- and default values

parameter	**short description**	**Indicative (i) or formulae (f) boundary values**	**Mathematical boundary values**
β	Angle of wave attack with respect to the structure	-90º– 90º (f)	-90º– 90º
γ_β	Reduction factor for angular wave attack	0,82 - 1 (f)	> 0
γ_f	Reduction factor for roughness of the seaward slope	0.55 - 1 (i)	> 0
γ_{f-c}	Reduction factor for roughness at the	0.55 - 1 (i)	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
	crest		
cot α_{rear}	rear side slope angle 1: …	2 - 4 (f)	> 0
$\xi_{s,-1}$	Surf similarity parameter for mean period wave	0.5 – 6 (i)	> 0
A_e	Erosion area on structure	10 - 1000 (i)	> 0
B	Structure crest width	1 – 100 (i)	> 0
c_0 , c_1	coefficients	c_0 =1.45, c_1 = 5.1	> 0
D_{n50}	Nominal diameter of the material on the rear side slope	0.1 – 5 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
N	Number of waves	< 4000 (f)	> 0
N_{od}	Number of displaced armour units within a strip of breakwater slope of width D_n	0.1 – 5 (i)	> 0
R_c	crest level relative to still water at the seaward side of the crest	0.1 – 5 (i)	> 0
$R_{c,rear}$	crest freeboard relative to the water level at rear side of the crest	0.1 – 5 (i)	> 0
S_d	Damage level	2 – 30 (f)	> 0
$T_{m-1,0}$	spectral wave period, also called the energy wave period	1 – 20 (i)	> 0
T_m	Mean wave period	1 – 20 (i)	> 0
t_r	Duration of storm or wave record	1 – 24 (i)	> 0
cot α	slope angle 1: …	1 - 10 (i)	> 0
ρ_s	Mass density of stone material	1500 - 3200 (i)	> ρ_w
ρ_w	Mass density of water	950 – 1050 (i)	> 0

23.4 References

Van Gent, M.R.A. and Pozueta, B. (2004), 'Rear-side stability of rubble mound structures', ASCE, Proc. ICCE 2004.

24 Wave load on crown wall

The overtopping performance of a rubble mound breakwater or seawall is often significantly improved by the use of a concrete crown wall.

24.1 Equation

The formulation of Jensen (1984) and Bradbury *et al* (1988) can be used to predict the wave forces on a crown wall. The formulation gives the maximum forces and tilting moments during a sea state defined by the significant wave height. The formulation of Jensen (1984) and Bradbury *et al* (1988) was based on test results for the structure configurations shown in Figure 24.1

The maximum horizontal force, F_H, is given by:

$$F_H = (\rho_w g\, d_c L_{op}) \cdot (a H_s / R_{ca} - b) \qquad (24.1)$$

$$L_{op} = g T_p^{\ 2} / 2\pi \qquad (24.1.1)$$

The uplift force, F_U, will then be given by:

$$F_U = (\rho_w g\, B_c L_{op} / 2) \cdot (a H_s / R_{ca} - b) \qquad (24.2)$$

in which:

parameter	short description	unit
ρ_w	Mass density of water	[kg/m^3]
a, *b*	coefficients	[-]
B_c	Width of the base of the crown wall	[m]
d_c	height of the crown-wall face	[m]
F_H	maximum horizontal force	[N/m]
F_u	uplift force	[N/m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
L_{op}	deepwater wavelength corresponding to the peak wave period	[m]
R_{ca}	armour crest level	[m]
T_p	Peak wave period	[s]

24.2 input and output parameters

Input:	**Output:**
a, *b*, R_{ca}, d_c, T_p, H_s, ρ_w	$\boldsymbol{F_H}$
a, *b*, R_{ca}, B_c, T_p, H_s, ρ_w	$\boldsymbol{F_u}$

24.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
ρ_w	Mass density of water	950 – 1050 (i)	> 0
B_c	Width of the base of the crown wall	1 – 100 (i)	> 0
d_c	height of the crown-wall face	0.5 -5 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
R_{ca}	armour crest level	0.1 – 5 (i)	> 0
T_p	Peak wave period	1 – 20 (i)	> 0

For the cross-sections shown in Figure 24.1, the values of the coefficients *a* and *b* have been summarised by Bucharth (1993) in Table 24.1. These values correspond with the force exceeded by 0.1% of the waves, $F_{H\,0.1\%}$.

Table 24.1 Empirical coefficients a and b for calculating wave forces on crown walls for cross-sections shown in Figure 24.1

Cross-section	Parameter ranges in tests			0.1% exceedance values for coefficients in Equation 24.1 and 24.2	
	R_{ca}	$s_{op} = H_s/L_{op}$	H_s/R_{ca}	*a*	*b*
A	5.6 – 10.6	0.016 – 0.036	0.76 – 2.5	0.051	0.026
B	1.5 – 3.0	0.005 – 0.011	0.82 – 2.4	0.025	0.016
C	0.10	0.023 – 0.07	0.9 – 2.1	0.043	0.038
D	0.14	0.04 – 0.05	1.43	0.028	0.025
E	0.18	0.04 – 0.05	1.11	0.011	0.0095

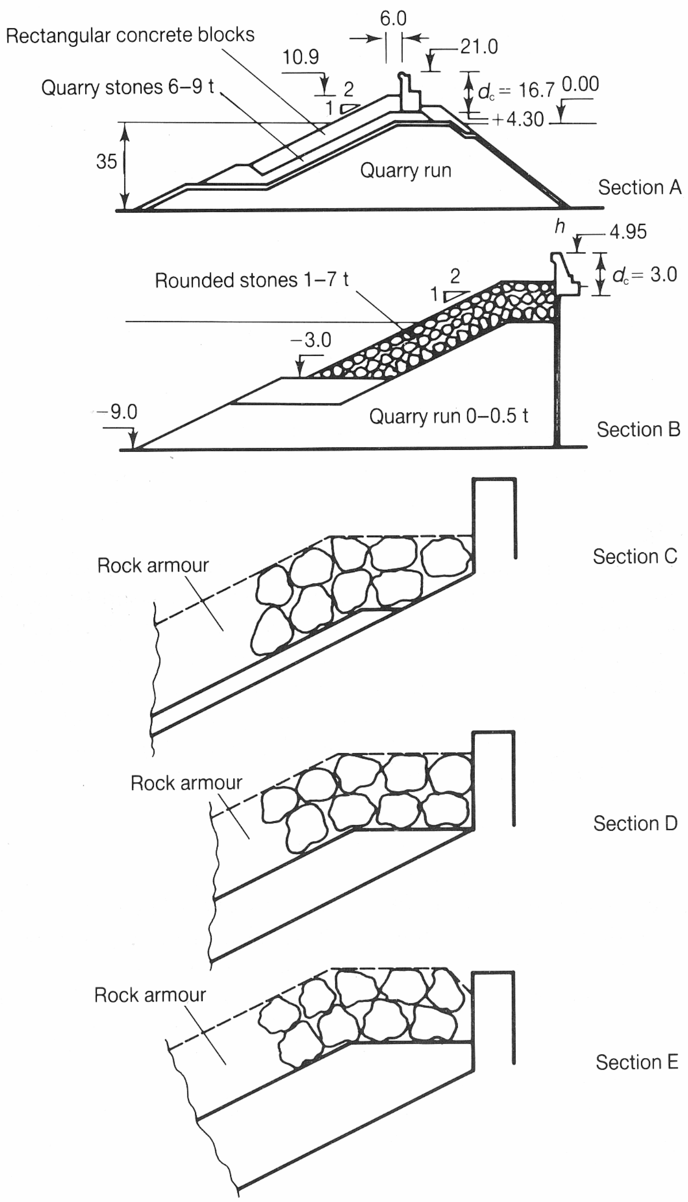

Figure 24.1 Crown wall sections tested by Jensen (1984) and Bradbury *et al* (1988)

24.4 References

JENSEN, O.J. (1984): A monograph on rubble mound breakwaters, Danish Hydraulic Institute.

BRADBURY, A.P., ALLSOP, N.W.H. and STEPHENS, R.V. (1988) Hydraulic performance of breakwater crown wall; report SR 146, Hydraulic Research Wallingford.

BURCHART, H.F. (1993) The design of breakwaters, Internal report, Aalborg University

25 Impact pressure on a crown wall

25.1 Equation

Pedersen (1996) presumed a pressure distribution on a crown wall as shown in figure 25.1

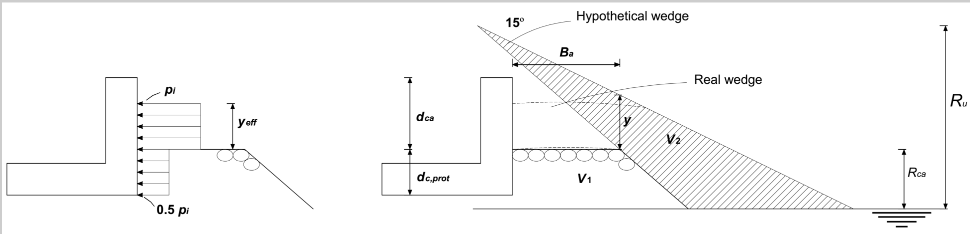

Figure 25.1: Pressure distribution according to Pedersen (1996)

The horizontal wave pressure component can be defined by:

$$p_i = g\rho_w \left(R_{u,0.1\%} - R_{ca}\right) \qquad (25.1)$$

in which according to Van der Meer and Stam (1992):

$$R_{u\,n\%} / H_s = a\xi_m \qquad \text{for } \xi_m < 1.5 \qquad (25.1.1)$$

$$R_{u\,n\%} / H_s = b\xi_m^{\ c} \qquad \text{for } \xi_m > 1.5 \qquad (25.1.2)$$

in which:
a = 1.12 and b = 1.34 for n = 0.1%

$$\xi_m = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{gT_m}}} \qquad (25.1.1.1)$$

In which:

parameter	short description	unit
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
p_i	Horizontal wave pressure component	[N/m^2]
R_{ca}	Vertical distance between SWL and the crest of the armour berm	[m]
$R_{u,0.1\%}$	Wave run-up level	[m]
T_m	mean wave period	[s]
α	Slope angle	[º]
ξ_m	Surf similarity parameter for mean period wave	[-]
ρ_w	Mass density of water	[kg/m^3]

The total horizontal force with a 0.1% probability of exceedance can be computed with:

$$F_{H0.1\%} = 0.21\sqrt{\frac{L_{om}}{B_a}}\left(1.6\, p_i\, y_{eff} + V\,\frac{p_i}{2}\, d_{c,prot}\right) \qquad (25.2)$$

in which:

V = min{V_2/V_1 , 1} (25.2.1)

$$L_{om} = \frac{gT_m}{2\pi} \qquad (25.2.2)$$

$$y = \frac{R_{u,0.1\%} - R_{ca}}{\sin\alpha}\,\frac{\sin 15°}{\cos(\alpha - 15°)} \qquad (25.2.3)$$

$$y_{eff} = \min\{y/2\,, d_{ca}\} \qquad (25.2.4)$$

$d_{ca} = R_c - R_{ca}$ (25.2.5)

The wave generated turning moment, $M_{H\,0.1\%}$ with a 0.1% probability of exceedance:

$$M_{H0.1\%} = a\,F_{H0.1\%} = 0.55\left(d_{c,prot} + y_{eff}\right)F_{H0.1\%} \qquad (25.3)$$

The wave uplift pressure, $p_{U\,0.1\%}$, with a 0.1% probability of exceedance:

$$p_{U0.1\%} = 1.0\; V\; p_i \qquad (25.4)$$

in which:

parameter	short description	unit
$F_{H0.1\%}$	Horizontal force with 0.1% probability of exceedance	[N]
$M_{H0.1\%}$	Wave generated moment	[Nm]
$p_{U0.1\%}$	Wave uplift pressure with 0.1% probability of exceedance	[N/m^2]
B_a	Berm width of armour layer in front of wall	[m]
$d_{c.prot}$	Difference between armour crest and bottom level of crown wall	[m]
d_{ca}	Difference of level between crown wall and armour crest	[m]
g	Gravitational acceleration	[m/s^2]
L_{om}	Deepwater wavelength corresponding to mean wave period	[m]
p_i	Horizontal wave pressure component	[N/m^2]
R_c	Crest freeboard, level of crest relative to still water level	[m]
R_{ca}	Vertical distance between SWL and the crest of the armour berm	[m]
$R_{u,0.1\%}$	Wave run-up level	[m]
T_{om}	Mean offshore wave period	[s]
V_1 , V_2	Areas shown in figure 25.1	[m^2]
y	Wedge thickness	[m]
y_{eff}	Effective impact zone height	[m]
α	Slope angle	[º]

25.2 input and output parameters

Input:	Output:
T_m, ρ_w, H_s, $\cot\alpha$, a, b	P_i
T_m, B_a, ρ_w, H_s, $\cot\alpha$, R_c, R_{ca} , $d_{c,prot}$, V_1, V_2	$F_{H0.1\%}$
T_m, B_a, ρ_w, H_s, $\cot\alpha$, R_c, R_{ca}, $d_{c,prot}$, V_1, V_2	$M_{H0.1\%}$,
T_m, B_a, ρ_w, H_s, $\cot\alpha$, R_c, R_{ca}, $d_{c,prot}$, V_1, V_2, P_i	$p_{U0.1\%}$

25.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
B_a	Berm width of armour layer in front of wall	1 – 100 (i)	> 0
$d_{c,prot}$	Difference between armour crest and bottom level of crown wall	0.1 – 5 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 – 10 (i)	> 0
R_c	Crest freeboard, level of crest relative to still water level	0.1 – 5 (i)	> 0
R_{ca}	Vertical distance between SWL and the crest of the armour berm	0.1 – 5 (i)	> 0
V_1	Areas shown in figure 25.1	figure 25.1	> 0
V_2	Areas shown in figure 25.1	figure 25.1	> 0
T_m	mean wave period	1 -20 (i)	> 0
$\cot\alpha$	Slope angle 1: …	1.5 – 3.5 (i)	> 0
ρ_w	Mass density of water	950 – 1050 (i)	> 0

The validity of the equations proposed by Pedersen is limited to the parametric ranges given in table 25.1

Table 25.1 Parameter ranges for method by Pedersen (1996)

Parameter	Symbol	Range
Breaker parameter using T_m	ξ_m	1.1 – 4.2
Relative wave height	H_s/R_{ca}	0.5 – 1.5
Relative run-up level	R_c/R_{ca}	1 – 2.6
Relative berm width	R_{ca}/B_a	0.3 – 1.1
Slope angle	$\cot\alpha$	1.5 – 3.5

25.4 References

PEDERSEN J. (1996) Wave forces and overtopping on crown walls of rouble mound breakwaters, Department of Civil Engineering, Aalborg University, series paper no. 12, pp140.

26 Forces on a crown wall according to martin

26.1 Equation

Martin (1999) derived a method to calculate wave forces for the case that waves hit the crown wall as broken waves. In this case the wave pressure was found to have to peaks. The first is generated during the direct change of direction of the bore front due to the crown wall. The second is occurs is related to the water mass rushing down the wall after the maximum run-up level is reached. The formulae by Martin are only valid for waves that reach the structure as surging or collapsing waves on the breakwater slope (ξ>3).

For preliminary design with this method it is recommended to use $H = H_{99.8\%}$ for the wave height. If no information on the wave height is available, then $H_{99.8\%} = 1.8 H_s$ can be used.

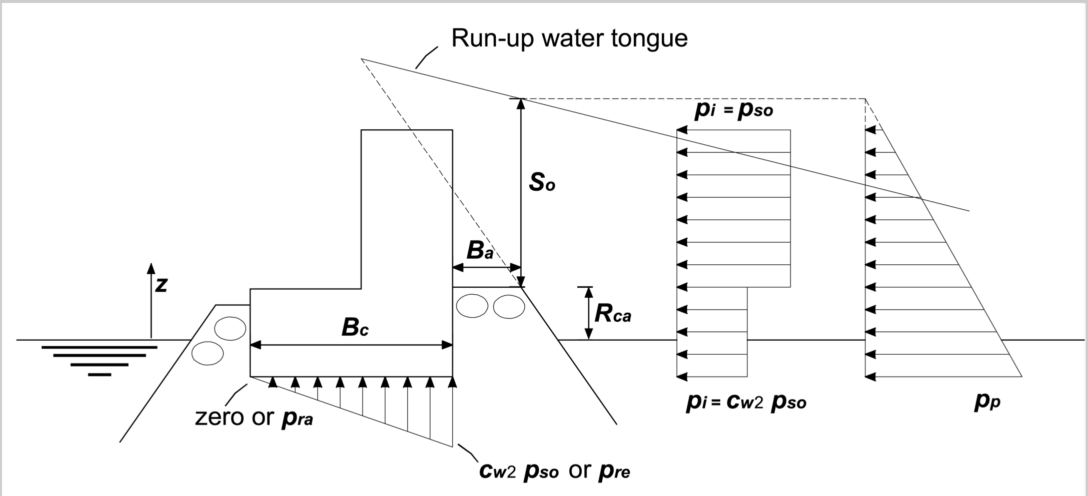

Figure 26.1: Pressure distribution according to Martin

The impact pressure over the unprotected region of the crown wall face (above R_{ca}) can be computed with:

$$p_i(z) = p_{s0} = c_{w1} \rho_w g S_0 \qquad (26.1)$$

in which:

$$S_0 = H(1 - R_{ca} / R_u) \qquad (26.1.1)$$

$$c_{w1} = 2.9\left(\left(R_u / H\right)\cos\alpha\right)^2 \qquad (26.1.2)$$

$$R_u / H = A_u \left(1 - \exp\left(B_u \xi\right)\right) \qquad (26.1.3)$$

$$\xi = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H_s}{g(\frac{1}{1.15}\cdot T_p)}}} \qquad (26.1.4)$$

for values of A_u and B_u see figure 26.2 (Losada et al.,1981)

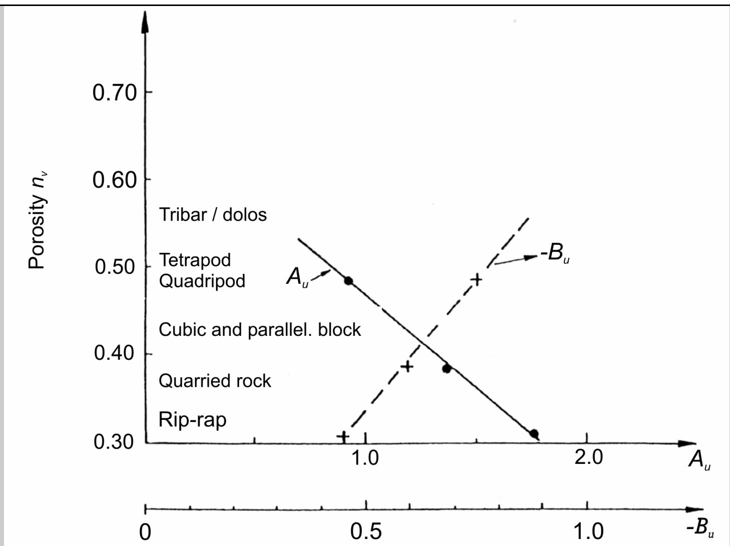

Figure 26.2: values for the run-up coefficients A_u and B_u (Losada et al., 1981)

The impact pressure over the region of the crown wall that is protected by the armour berm can be computed with:

$$p_i(z) = c_{w2} p_{so} = c_{w2} c_{w1} \rho_w g S_o \qquad (26.2)$$

For 0.03 < H/L_p < 0.075:

$$c_{w2} = 0.8 \exp\left(-10.9\, B_a / L_p\right) \qquad (26.2.1)$$

The pulsating pressure, p_p, distribution over the region of the crown wall that is protected by the armour berm is described by the following equations:

$$p_p(z) = c_{w3} \rho_w g \left(S_o + R_{ca} - z\right) \qquad (26.3)$$

in which:

$$c_{w3} = a \exp(c_o) \qquad (26.3.1)$$

$$c_o = c\left(H/L_p - b\right)^2 \qquad (26.3.2)$$

$$L_p = \frac{g T_p^{\ 2}}{2\pi} \qquad (26.3.3)$$

In which:

parameter	short description	unit
a, b, c	coefficients	[-]
A_u, B_u	Run-up coefficients	[-]

parameter	short description	unit
B_a	Width of armour berm at crest	[m]
C_0, C_{w1}, C_{w2}, C_{w3}	coefficients	[-]
D_{n50}	nominal mean diameter of the armour units forming the berm	[m]
g	Gravitational acceleration	[m/s^2]
H_s	Significant wave height, average of highest 1/3 of all wave heights	[m]
L_p	Wave length of peak wave period	[m]
n_v	Porosity of material on which the crown wall is founded	
P_i	Impact pressure	[N/m^2]
P_{s0}		[N/m^2]
P_p	Pulsating pressure	[N/m^2]
R_{ca}	Vertical distance between SWL and the crest of the armour berm	[m]
R_u	Wave run-up	[m]
S_0		
T_p	peak wave period	[s]
z		[m]
α	Slope angle	[º]
ξ	Surf similarity parameter	[-]
ρ_w	Mass density of water	[kg/m^3]

26.2 input and output parameters

Input:	Output:
ρ_w, H, R_{ca}, A_u, B_u, $\cot\alpha$, B_a, T_p, z, a, b, c, n_v	$\boldsymbol{P_i}$, P_p

26.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
a, b, c	coefficients	See table 26.1	
A_u, B_u	Run-up coefficients	See figure 26.2	
B_a	Width of armour berm at crest	1 -100 (i)	> 0
H_s	Significant wave height, average of highest 1/3 of all wave heights	0.1 -10 (i)	> 0
n_v	Porosity of material on which the crown wall is founded	0 – 1 (i)	> 0
R_{ca}	Vertical distance between SWL and the crest of the armour berm	0.1 – 5 (i)	> 0
T_p	peak wave period	1 – 20 (i)	> 0
z			
$\cot\alpha$	Slope angle 1: …	1 - 10 (i)	> 0
ρ_w	Mass density of water	950 – 1050 (i)	> 0

Values for the empirical coefficients *a*, *b* and *c* can be found in Table 26.1

Table 26.1: Empirical coefficients a, B and c for calculating pulsating pressures

B_a / D_{n50}	a	B	c
1	0.446	0.068	259.0
2	0.362	0.069	357.1
3	0.296	0.073	383.1

At the seaward edge, both the impact and the pulsating pressure beneath the structure are equal to the pressure at the front.

$p_i = c_{w2} p_{so}$

$p_p = p_{re}$

At the heel of the crown wall, the dynamic uplift pressure can be assumed negligible. The pulsating pressure at the heel can be predicted with Figure 26.2 using the porosity, n_v, of the material on which the crown wall is founded and the pressure at the seaward edge, p_{re}.

$p_i = 0$

$p_p = p_{ra}$ (see Figure 26.2)

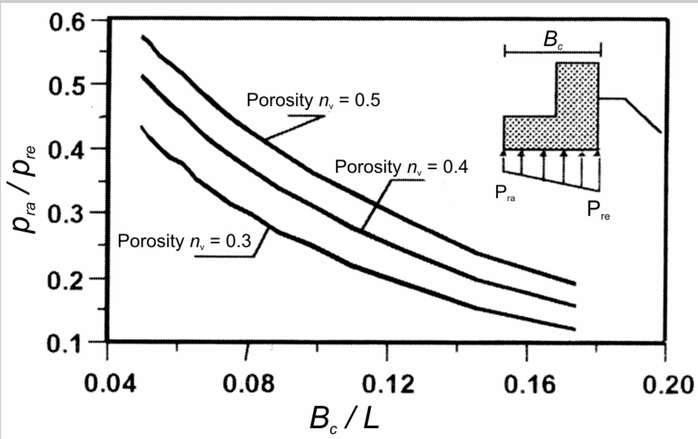

Figure 26.2: Pulsating pressure at the heel (*L* = peak wave length)

26.4 References

MARTIN, F.L., (1999). Experimental study of wave forces on rubble mound breakwater crown walls, PIANC Bulletin, 1999; pp 5-17

LOSADA, M.A., GIMINEZ-CURTO, L.A. (1981) Flow characteristics on rough permeable slopes under wave action, Coastal Engineering 4, 187-206

27 Escarameia and May

Escarameia and May (1992) suggested an equation that is a form of the Isbash equation in which the effects of the turbulence of the flow are fully quantified. This can be particularly useful in situations where the levels of turbulence are higher than normal: near river training structures, around bridge piers, cofferdams and caissons, downstream of hydraulic structures (gates, weirs, spillways, culverts), at variations in bed level, at abrupt changes in flow direction.

27.1 equation

This equation provides an envelope to the experimental data that was used to derive it and is valid for flat beds and slopes not steeper than 1V:2H. The laboratory data were further checked against field measurements of turbulence in the river Thames with water depths between 1 and 4m.

$$D_{n50} = c_T \frac{u_b^2}{2g\Delta} \quad (27.1)$$

$$\Delta = (\rho_s / \rho_w) - 1 \quad (27.1.1)$$

$$M_{50} = \rho_s D_{n50}{}^3 \quad (27.1.2)$$

In which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
c_T	turbulence coefficient	[-]
D_{n50}	nominal mean diameter	[m]
g	gravitational acceleration	[m/s^2]
M_{50}	mass of a stone that is exceeded by 50% of the stones	[kg]
u_b	near bed velocity, defined at 10% of the water depth above the bed	[m/s]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

27.2 input and output parameters

Input:	Output:
c_T, u_b, ρ_s, ρ_w	**D_{n50}**, M_{50}

27.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
D_{n50}	nominal mean diameter	0,1 – 5 (i)	> 0
c_T	turbulence coefficient	see table 27.1 and 27.2	> 0

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
		rip rap: ≥ 0,42 (f) gabion mattress: ≥ 0,2 (f)	
u_b	near bed velocity, defined at 10% of the water depth above the bed	see table 27.1 0,1 - 10 (i)	> 0
ρ_s	Mass density of stone material	1500 – 3200 (i)	> 0
ρ_w	Mass density of water	950 - 1050 (i)	> 0

Guidance on how to use Equation 27.1 is given in Table 27.1 In table 27.2 some specific values for the turbulence intensity are presented, that can be considered in the absence of site specific information. For further information on the development and use of this equation see Escarameia and May (1995) and Escarameia (1998).

Table 27.1 Design guidance for parameters in Equation 27.1

Mean diameter, D_{n50}	rip-rap: $D_{n50} = (M_{50} / \rho_r)^{1/3}$ gabion mattresses: D_{n50} = stone size within gabion Note: Equation 27.1 was developed from tests on gabion mattresses 300 mm in thickness
Turbulence coefficient, c_T	rip-rap (valid for $r \geq 0.05$): $c_T = 12.3\ r - 0.20$ gabion mattresses (valid for $r \geq 0.15$): $c_T = 12.3\ r - 1.65$ where r = turbulence intensity defined at 10% of the water depth above the bed ($r = u'_{rms} / u$), see also Table 27.2
Near bed velocity, u_b	If data is not available an estimation can be made in relation to the depth average velocity, U, as: u_b= 0.74 to 0.90 U
Relative buoyant density of material, Δ	$\Delta = \rho_r / \rho - 1$ where ρ_r = mass density of rock and ρ = mass density of water

Table 27.2 Typical turbulence levels

Situation	Turbulence level	
	Qualitative	Turbulence intensity, r
Straight river or channel reaches	Normal (low)	0.12
Edges of revetments in straight reaches	normal (high)	0.20
Bridge piers, caissons and spur dikes; transitions	medium to high	0.35 – 0.50
Downstream of hydraulic structures	very high	0.60

27.4 References

ESCARAMEIA, M. and MAY, R. W. P. (1992) Channel protection: turbulence downstream of structures. HR Wallingford, Report SR313

ESCARAMEIA, M. and MAY, R. W. P. (1995) Stability of rip-rap and concrete blocks in highly turbulent flows. Proceedings of the Institution of Civil Engineers, Water Maritime and Energy, Vol 112, Issue 3.September 1995, pp 227-237.

ESCARAMEIA, M. (1998) River and channel revetments. a design manual, Thomas Telford, London. ISBN 0 7277 2691 9.

28 Maynord

Maynord (1993) developed the US Army Corps of Engineers' Design Procedure and suggested a stability equation for rip-rap, which is not based on the threshold of movement criterion (unlike the Pilarczyk and Escarameia & May equations). It is instead based on not allowing that the underlying material be exposed and therefore takes the thickness of the rip-rap layer into account. The form of the equation presented here is in SI units:

28.1 Equation

$$D_{30} = S_f C_{st} C_v C_T h \left(\left(\frac{1}{\Delta} \right)^{0.5} \frac{U}{\sqrt{k_{sl} g h}} \right)^{2.5} \quad (28.1)$$

$$k_{sl} = -0{,}672 + 1{,}492 \cot \alpha - 0{,}449 \cot^2 \alpha + 0{,}045 \cot^3 \alpha \quad (28.1.1)$$

$$D_{n30} = 0{,}84 D_{30} \quad (28.1.2)$$

$$M_{30} = \rho_s D_{n30}^{\ 3} \quad (28.1.3)$$

In which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
C_{st}	stability coefficient	[-]
C_T	blanket thickness coefficient	[-]
C_v	velocity distribution coefficient	[-]
D_{30}	diameter of stone which exceeds the 30% value of sieve curve	[m]
D_{n30}	nominal diameter of stone which exceeds the 30% value of sieve curve	[m]
g	gravitational acceleration	[m/s^2]
h	water depth	[m]
k_{sl}	Side slope factor	[-]
M_{30}	mass of a stone that is exceeded by 70% of the stones	[kg]
S_f	safety factor	[-]
U	depth-averaged flow velocity	[m/s]
α	slope angle	[º]
ρ_s	Mass density of the stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]
Φ	angle of repose of the submerged granular material	[º]
ψ	angle of the flow to the upslope direction	[º]

28.2 input and output parameters

Input:	Output:
C_T, C_{st}, C_v, h, U, k_{sl}, cot α, ψ, Φ, ρ_s, ρ_w	$\boldsymbol{D_{30}}$, M_{30}

28.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
cot α	slope angle 1: …	1 - 10 (i)	> 0
C_{st}	stability coefficient	see table 28.1	
C_T	blanket thickness coefficient	see table 28.1	
C_v	velocity distribution coefficient	see table 28.1	
D_{30}	characteristic size of rip-rap	0,1 – 5 (i)	> 0
h	water depth	0 – 20 (i)	> 0
S_f	safety factor	see table 28.1	
U	depth-averaged flow velocity	0,1 – 20 (i)	> 0
ρ_s	Mass density of the material	1500 – 3200 (i)	> 0
ρ_w	Mass density of water	950 – 1050 (i)	> 0
Φ	angle of repose of the submerged granular material	0º – 90º (i)	0º - 90º
ψ	angle of the flow to the upslope direction	-90º – 90º (i)	-90º - 90º

New parameters specific to Maynord's equation are outlined below and guidance on the use of the different parameters is given in table 28.1. For more information on this equation reference is made to Maynord (1993).

Table 28.1 Design guidance for parameters in Equation 28.1

Characteristic size, D_{30}	$D_{30} \approx 0.70\ D_{50}$, or more precisely: $D_{30} = D_{50}\ (D_{15}/D_{85})^{0.32}$
Safety factor, S_f	minimum value: $S_f = 1.1$
Stability coefficient, C_{st}	angular rock: $C_{st} = 0.3$ rounded rock: $C_{st} = 0.375$
Velocity distribution coefficient, C_v	straight channels, inside of bends: $C_v = 1.0$ outside of bends: $C_v = 1.283 - 0.2 \log (r/B)$ where r = centre radius of bend (m) and B = water surface width at upstream end of bend (m) downstream of concrete structures or at the end of dykes: $C_v = 1.25$
Blanket thickness coefficient, C_T	standard design: $C_T = 1.0$ see Maynord (1993) otherwise
Relative buoyant density of material, Δ	$\Delta = \rho_r / \rho - 1$ where ρ_r = density of rock and ρ = density of water
Side slope factor, k_{sl}	$k_{sl} = -0.672 + 1.492 \cot\alpha - 0.449 \cot^2\alpha + 0.045 \cot^3\alpha$ where α = angle of the bank to the horizontal

28.4 References

MAYNORD, S. T. (1993) Corps rip-rap design guidance for channel protection. Fort Collins, Colorado, USA

29 Stability ship induced currents

A common type of loading for riverbanks and navigation channels is attributed to ship-induced water movements. Velocities and wave heights resulting from return current, water level depression, transversal stern wave, interference peaks (or secondary ship waves) and jet flow due to propeller thrust, determine the required size of protective elements.

29.1 Equations

$$\frac{U'^2/2g}{\Delta D_{50}} = 2\frac{k_{sl}}{k_t^2} \quad (29.1)$$

$$k_{sl} = \frac{\cos\psi\sin\beta + \sqrt{\cos^2\beta\tan^2\phi - \sin^2\psi\sin^2\beta}}{\tan\phi} \quad (29.1.1)$$

$$k_t = \frac{1+3r}{1.3} \quad (29.1.2)$$

$$D_{n50} = 0{,}84 D_{50} \quad (29.1.3)$$

$$M_{50} = \rho_s D_{n50}{}^3 \quad (29.1.4)$$

In which:

parameter	short description	unit
Δ	Relative buoyant density of material	[-]
D_{50}	diameter of stone which exceeds the 50% value of sieve curve	[m]
D_{n50}	nominal mean diameter	[m]
g	gravitational acceleration	[m/s^2]
k_t	turbulence factor	[-]
M_{50}	mass of a stone that is exceeded by 50% of the stones	[kg]
r	depth-averaged relative fluctuation intensity due to turbulence	[-]
U’	Velocity (U’ can be substituted by U_r for return currents and by U_p for propeller jets)	[m/s]
β	slope angle	[º]
ρ_s	Mass density of the material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]
Φ	angle of repose of the submerged granular material	[º]
ψ	angle of the flow to the upslope direction	[º]

29.2 input and output parameters

Input:	Output:
β, ψ, Φ, ρ_s, ρ_w	D_{50}, $\mathbf{D_{n50}}$, M_{50}

29.3 boundary- and default values

parameter	short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
cot β	slope angle 1: …	1 - 10 (i)	> 0
D_{50}	diameter of stone which exceeds the 50% value of sieve curve	0,1 – 5 (i)	>0
D_{n50}	nominal mean diameter	0,1 – 5 (i)	> 0
r	depth-averaged relative fluctuation intensity due to turbulence	0 – 1 (i)	> 0
U'	Velocity (U' can be substituted by U_r for return currents and by U_p for propeller jets)	0,1 – 10 (i)	>0
ρ_s	Mass density of stone material	2500 – 3200 (i)	>0
ρ_w	Mass density of water	950 – 1050 (i)	>0
Φ	angle of repose of the submerged granular material	0º – 90º (i)	0º - 90º
ψ	angle of the flow to the upslope direction	-90º – 90º (i)	-90º - 90º

29.4 References

30 Stability of near bed structures

Near-bed rubble mound structures are submerged structures of which the crest is relatively low, such that wave breaking does not have a significant influence. Near-bed structures are for example applied as river spur dykes, pipeline covers, and intake and outfall structures near power and desalination plants. Figure 30.1 shows a sketch of a near-bed structure.

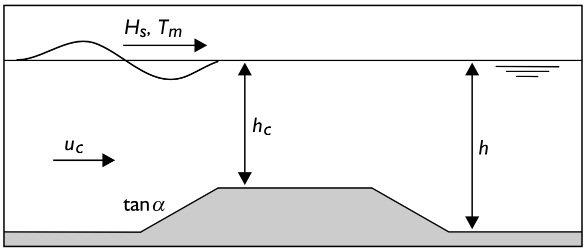

Figure 30.1 Definition sketch of a near-bed rubble mound structure

The load on near-bed structures consists of waves, currents, or a combination of waves and currents. Information on the stability of near-bed structures for conditions where waves or a current approach the structure at an angle (other than perpendicular) is scarce. This section focuses on the stability of near-bed structures under currents only.

30.1 Equation

The flow velocity above a near-bed structure, can be calculated with:

$$U = q/h_c = \mu (h_b - d)\sqrt{2g(H - h_b)} \tag{30.1}$$

$$H = h + \frac{U_{up}^{\ 2}}{2g} \tag{30.1.1}$$

$$U_{up} = \frac{q}{h} \tag{30.1.2}$$

The value of μ varies between 0.9 and 1.1. Equation 30.1 is valid under sub-critical flow conditions. This is generally the case if $d\,/\,h < 0.33$.
For the stability of the rock on a near-bed structure under currents only, the start of movement of rocks is an important design aspect. Because of the fact that the load of currents on the structure is present at a more or less constant level, especially compared to wave loads, a certain critical velocity should not be exceeded. The formulae by Hoffmans and Akkerman (1998) are based on the Shields parameter using such a velocity:

$$D_{n50} = 0.7\frac{(r_o U)^2}{g\Delta\psi} \tag{30.2}$$

$$r_o = \sqrt{c_s + 1.45\frac{g}{C^2}} \tag{30.3}$$

$$c_s = c_k\left(1 - \frac{d}{h_b}\right)^{-2} \tag{30.4}$$

$$M_{50} = \rho_s D_{n50}^{\ 3} \tag{30.5}$$

The formulae by Hoffmans and Akkerman take the turbulence into account. These empirical formulae fit very well on uniform, as well as on non-uniform flow conditions, although the factor 0.7 in Equation 30.2 can only be derived theoretically for uniform flow conditions.
In uniform flow the parameter 1.45 g / C^2 is about 0.01, resulting in r_o = 0.1 which is a well-known value. In the vicinity of structures non-uniform flow conditions are present and the turbulence is higher. Therefore the parameter c_s has been introduced, which depends on the relative structure height and c_k. The value of c_k depends on the structure type. Based on tests a value of c_k = 0.025 is recommended. For d / h_b = 0.33 (maximum structure height) the value of c_s becomes about 0.056 and consequently, the value of r_o becomes about 0.26. For design purposes it is recommended not to exceed a value of 0.035 for the Shields parameter.

An overview of the used parameters is given below:

parameter	**short description**	**unit**
μ	discharge coefficient	[-]
Δ	Relative buoyant density of material	[-]
σ	standard deviation of the flow velocity	[m/s]
ψ	Shields parameter	[-]
C	Chézy coefficient	[$m^{1/2}$/s]
c_k	turbulence factor related to the structure	[-]
c_s	parameter which depends on the relative structure height and c_k	[-]
d	structure height	[m]
D_{n50}	nominal mean diameter,	[m]
g	gravitational acceleration	[m/s^2]
H	upstream energy level	[m]
h	upstream water depth	[m]
h_b	downstream water depth	[m]
h_c	depth above the crest	[m]
M_{50}	mass of a stone that is exceeded by 50% of the stones	[kg]
q	specific discharge	[m^2/s]
r_o	turbulence intensity (= σ / u)	[-]
U	flow velocity above a near-bed structure	[m/s]
u	flow velocity	[m/s]
U_{up}	upstream flow velocity (= q / h)	[m/s]
ρ_s	Mass density of stone material	[kg/m^3]
ρ_w	Mass density of water	[kg/m^3]

30.2 Input and output parameters

Input:	Output:
h_b, d, q, μ, ρ_s, ρ_w, h, C, c_k, ψ	U, H, **D_{n50}**, r_0, c_s, M_{50}

30.3 Boundary- and default values

Parameter	Short description	Indicative (i) or formulae (f) boundary values	Mathematical boundary values
μ	discharge coefficient	0,9 – 1,1 (i)	> 0
σ	standard deviation of the flow velocity		> 0
ψ	Shields parameter	< 0,035 (i)	> 0
C	Chézy coefficient	10 – 90 (i)	> 0
c_k	turbulence factor related to the structure	0,025 (i)	> 0
d	structure height	$d / h_b < 0.33$ (f)	> 0
D_{n50}	nominal mean diameter,	0,1 – 5 (i)	> 0
h	upstream water depth	0,1 – 30 (i)	> 0
h_b	downstream water depth	$d / h_b < 0.33$ (f)	> 0
h_c	depth above the crest	0,1 – 20 (i)	> 0
q	specific discharge		> 0
r_o	turbulence intensity (= σ / u)	0,1 – 0,6 (i)	> 0
U	flow velocity above a near-bed structure	0,1 – 10 (i)	> 0
u	flow velocity	0,1 – 10 (i)	> 0
U_{up}	upstream flow velocity	0,1 – 10 (i)	> 0
ρ_s	Mass density of stone material	1500 – 3200 (i)	> 0
ρ_w	Mass density of water	950 – 1050 (i)	> 0

30.4 References

HOFFMANS, G. J. C. M. and AKKERMAN, G. J. (1998) Influence of turbulence on stone stability. Proceedings of 7th International Symposium on River Sedimentation, Hong Kong

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Bijlage 1
Notatielijst

parameter	Short description	unit
A	cross-sectional area	[m^2]
A	coefficient	[-]
A_e	erosion area on structure	[m^2]
A_t	area of structure cross section	[m^2]
A_u	run-up coefficient	[-]
a	coefficient	[-]
a	scale parameter	[m^3/m]
B	structure crest width	[m]
B_a	berm width of armour layer in front of wall	[m]
B_a	width of armour berm at crest	[m]
B_c	width of the base of the crown wall	[m]
B_n	bulk number (cross-section of stones)	[-]
B_u	run-up coefficient	[-]
B	coefficient	[-]
b	coefficient	[-]
b	shape parameter	[-]
C	parameter depending on the slope	[-]
C	chézy coefficient	[$m^{1/2}$/s]
C	coefficient	[-]
C_0	response slope as built	[-]
C_0, C_{w1}, C_{w2}, C_{w3}	coefficients for calculating pulsating pressures	[-]
C_{pl}	coefficient for plunging conditions	[-]
C_r	coefficient of wave reflection	[-]
C_s	coefficient for surging conditions	[-]
C_{st}	stability coefficient	[-]
C_t	coefficient of wave transmission	[-]
$C_{t,max}$	maximum value coefficient of wave transmission	[-]
C_T	banket thickness coefficient	[-]
C_T	turbulence coefficient	[-]
C_U	cefficient of uniformity	[-]
C_v	velocity distribution coefficient	[-]
c	cefficient	[-]
$c'_{a,h}$	coefficient	[-]
$c'_{c,h}$	coefficient	[-]
$c'_{c,u}$	coefficient	[-]
c_0	coefficient	[-]
c_1	coefficient	[-]
c_2	coefficient	[-]
$c'_{a,u}$	coefficient	[-]
$c_{c,h}'$	coefficient	[-]
$c_{c,h}''$	coefficient	[-]
$c_{c,u}'$	coefficient	[-]
$c_{c,u}''$	coefficient	[-]
c_k	turbulence factor related to the structure	[-]
c_s	parameter which depends on the relative structure height and c_k	[-]
c_T	turbulence coefficient	[-]
D	diameter of armour units	[m]
D_{10}	diameter of stone which exceeds the 10% value of sieve curve	[m]
D_{30}	diameter of stone which exceeds the 30% value of sieve curve	[m]

parameter	Short description	unit
D_{50}	diameter of stone which exceeds the 50% value of sieve curve	[m]
D_{60}	diameter of stone which exceeds the 60% value of sieve curve	[m]
d	cefficient	[-]
d	thickness of armour layer	[m]
d	crest height after wave attack	[m]
d	depth in front of the breakwater	[m]
d	structure height	[m]
d/D_{n50}	depth factor function	[-]
d_0	original crest height before wave attack	[m]
d_c	height of the crown-wall face	[m]
$d_{c,prot}$	difference between armour crest and bottom level of crown wall	[m]
d_{ca}	difference of level between crown wall and armour crest	[m]
D_n	nominal diameter of armour units	[m]
D_{n30}	nominal diameter of stone which exceeds the 30% value of sieve curve	[m]
D_{n15}	nominal diameter of stone which exceeds the 15% value of sieve curve	[m]
D_{n85}	nominal diameter of stone which exceeds the 85% value of sieve curve	[m]
D_{n50}	nominal mean diameter	[m]
$D_{n50\text{-}core}$	nominal mean diameter of core material	[m]
e	voids ratio defined as the ratio of volume of the voids and total rockfill volume	[-]
F_H	maximum horizontal force	[N/m]
$F_{H\text{-}0,1\%}$	horizontal force with 0.1% probability of exceedance	[N]
F_u	uplift force	[N/m]
$f(f_g)$	gradation factor function	[-]
f_g	gradation factor	[-]
f_L	friction factor at the landward slope	[-]
g	gravitational acceleration	[m/s^2]
H	upstream energy level	[m]
$H_{2\%}$	wave height exceeded by 2% of the incident waves at the toe	[m]
H_i	incident wave height	[m]
H_i	interference wave height	[m]
H_0T_0	period stability number	[-]
H_s	significant wave height, average of highest 1/3 of all wave heights	[m]
H_s (=H_{m0})	significant wave height calculated from the spectrum, $H_{m0}=4\sqrt{m_0}$	[m]
H_{sr}	wave height after reflection	[m]
H_t	transmitted wave height	[m]
h	water depth	[m]
h' / h_m	relative depth of the rubble mound foundation	[-]
h'	water depth at the crest of the rubble mound foundation	[m]
h_0	thickness of water layer at transition of crest and innerslope	[m]
$h_{2\%}$	thickness of water layer	[m]
h_B	water depth of the toe berm	[m]
h_b	downstream water depth	[m]
h_c	armour crest level relative to sea bed	[m]
h_c	depth above the crest	[m]
h_f	distance between point A and SWL	[m]
h_m	water depth in front of the structure	[m]
h_t	water depth at structure toe	[m]
i	gradient of (phreatic) water level	[-]

parameter	Short description	unit
K	coefficient that depends on stone shape	[-]
K_D	stability coefficient	[-]
k	wave number	[m^{-1}]
k_{sl}	side slope factor	[-]
k_t	layer thickness coefficient	[-]
k_t	turbulence factor	[-]
L	wave length	[m]
L_i	interference wave length	[m]
L_{om}	deepwater wavelength corresponding to mean wave period	[m]
L_{op}	deepwater wavelength corresponding to the peak wave period	[m]
L_p	wave length of peak wave period	[m]
$M_{H0,1\%}$	wave generated moment	[Nm]
M_{30}	mass of a stone that is exceeded by 70% of the stones	[kg]
M_{50}	mass of a armour unit that is exceeded by 50% of the stones	[kg]
N	number of waves	[-]
N	number of passages of relevant types of ships during the total life time of the structure	[-]
N^*_s	spectral (or modified) stability number	[-]
N_{od}	number of displaced armour units within a strip of breakwater slope of width D_n	[-]
N_{od_f}	damage number at failure.	[-]
N_{ov}	number of overtopping waves out of a total of N incoming waves in an examined time period T_r (= N T_m)	[-]
N_s	stability number	[-]
n	time counter	[-]
n_v	volumetric porosity of the medium	[-]
P	structure permeability	[-]
P(V)	Pr($\underline{V}$ < V) = probability of non-exceedance of a given volume, V	-
P_i	impact pressure	[N/m^2]
P_p	pulsating pressure	[N/m^2]
p	coefficient	[-]
p_i	horizontal wave pressure component	[N/m^2]
$p_{u0,1\%}$	wave uplift pressure with 0.1% probability of exceedance	[N/m^2]
Q	discharge through the rockfill	[m^3/s]
q	specific discharge	[$m^3/s/m$]
R^*	dimensionless freeboard	[-]
R_c	crest freeboard, level of crest relative to still water level	[m]
$R_{c,rear}$	crest freeboard relative to the water level at rear side of the crest	[m]
R_{ca}	armour crest level	[m]
$R_{d1\%}$	wave run down height for the 1% wave	[m]
$R_{d2\%}$	wave run down height for the 2% wave	[m]
Rec	width of berm eroded (B=Rec)	[m]
Rec_{3000}	width of berm eroded by 3000 waves	[m]
Rec_n	width of berm eroded by N waves	[m]
R_p	fraction of rounded stones in armour	[-]
R_u	wave run-up	[m]
$R_{u\ n\%}$	run-up level exceed by only n% of run-up tongues	[m]
$R_{u,0.1\%}$	wave run-up level	[m]

parameter	Short description	unit
$R_{u1\%}$	fictitious run-up level	[m]
$R_{u2\%}$	wave run up height for the 2% wave	[m]
r	dolos waist ratio	[-]
r	depth-averaged relative fluctuation intensity due to turbulence	[-]
r_o	turbulence intensity (= σ / u)	[-]
S	stability parameter	[-]
S(t0)	damage at time t0	[-]
S(tn)	damage at time tn	[-]
S_d	damage level	[-]
S_f	safety factor	[-]
s	the co-ordinate along the landward slope (with s = 0 at the landward side of the crest)	[m]
s_p	wave steepness for peak wave period	[-]
s_m	wave steepness for mean wave period	[-]
t_0	duration time of storm to reach a damage S(t0)	[s]
tan α	tangent of the slope angle	[-]
T_m	mean wave period	[s]
$T_{m-1,0}$	spectral wave period, also called the energy wave period	[s]
T_{om}	mean offshore wave period	[s]
T_p	peak wave period	[s]
t_n	duration time of additional storm	[s]
t_r	duration of storm or wave record	[s]
U	depth-averaged flow velocity	[m/s]
U_{up}	upstream flow velocity (= q / h)	[m/s]
U_v	velocity through the voids	[m/s]
u	flow velocity	[m/s]
u_0	velocity at the transition of crest and innerslope	[m/s]
$u_{1\%}$	maximum velocity (depth-averaged) at the rear side of the crest (m/s) during a wave overtopping event, exceeded by 1% of the incident waves	[m/s]
$u_{2\%}$	wave run-up velocity	[m/s]
u_b	near bed velocity, defined at 10% of the water depth above the bed	[m/s]
$\hat{u}_\delta$	peak bottom velocity	[m/s]
V_{max}	maximum expected individual overtopping volume	[m^3 /m]
V_s	velocity of the ship	[m/s]
W	weight of a stone	[N]
W_{50}	weight of a stone that is exceeded by 50% of the stones	[N]
x	position parameter (with x = 0 at seaward side of the crest)	[m]
y	wedge thickness	[m]
y_{eff}	effective impact zone height	[m]
y_s	the distance to the bank normal to the sailing line	[m]
z	position (vertical height) on the seaward slope relative to SWL	[m]
z_{max}	height of the stern wave	[m]

parameter	Short description	unit
α	slope angle	[º]
α_i	coefficient depending on the type of ship	[-]
α_{rear}	slope angle at the rear side	[º]
β	slope angle	[º]
β	angle of wave attack with respect to the structure	[º]
γ	reduction factor	[-]
γ_b	berm-factor	[-]
γ_β	correction factor for oblique waves	[-]
γ_β	angular wave attack	[-]
γ_f	roughness coefficient	[-]
γ_{f-c}	roughness at the crest	[-]
Δ	relative buoyant density of material	[-]
θ	mobility parameter	[-]
μ	coefficient	[-]
μ	discharge coefficient	[-]
ξ	surf-similarity parameter	[-]
ξ_c	critical value of the surf-similarity parameter	[-]
ξ_m	surf similarity parameter for the mean period wave	[-]
$\xi_{m-1,0}$	breaker parameter based on spectral analysis	[-]
ξ_p	surf similarity parameter for peak period wave	[-]
$\xi_{s-1,0}$	surf-similarity parameter using the spectral wave period Tm-1,0	[-]
ρ_s	mass density of stone material	[kg/m^3]
ρ_w	mass density of water	[kg/m^3]
σ	standard deviation of the flow velocity	[m/s]
Φ	angle of repose of the submerged granular material	[º]
ϕ	packing density	[-]
ψ	angle of the flow to the upslope direction	[º]
ψ	shields parameter	[-]