STABILITY DESIGN OF COASTAL STRUCTURES (SEADIKES, REVETMENTS, BREAKWATERS AND GROINS)

by

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1. INTRODUCTION

2. HYDRODYNAMIC BOUNDARY CONDITIONS AND PROCESSES

1 INTRODUCTION

The most common coastal structures, made of rock, are:

- Seawalls; almost vertical or steep-sloped impermeable structures at the landward end of beaches or at locations where beaches and dunes are absent;
- Seadikes; mild-sloped, impermeable structures at locations without beaches and dunes;
- Shore revetments; mild-sloped structures to protect the high water zone of dunes/boulevards;
- Shore-attached and shore-detached breakwaters; structures to reduce wave heights/currents;
  - high-crested breakwaters;
  - low-crested, emerged breakwaters with crest above still water level (groins);
  - low-crested, submerged breakwaters with crest below still water level (reefs).

Seawalls, seadikes and revetments generally have an almost impermeable outer layer consisting of closely fitted rocks or concrete blocks (sometimes asphalt layers) on filter layers and a core body of sand and/or clay. Breakwaters are permeable structures (open outer layer).

The geometrical dimensions (shape, cross-section, materials, etc.) of a coastal structure depends on:

- Location (backshore, nearshore, offshore) and type of structure;
- Fuctional requirements;
  - flood protection (seawall, seadike; high crest levels are required),
  - wave reduction (berm breakwater) and/or flow protection (groin; low crest level is sufficient),
  - dune/shore protection (revetment),
  - beach fill protection (terminal groins).

Geometrical definitions are (see also Figure 1.1):

- Crest height (R_c) = distance between the still water level and the crestpoint where overtopping water can not flow back to the sea through the permeable armour layer (= freeboard).
- Armour slope = slope of the outer armour layer between the run-up level above SWL and a distance equal to 1.5 H_i below SWL.

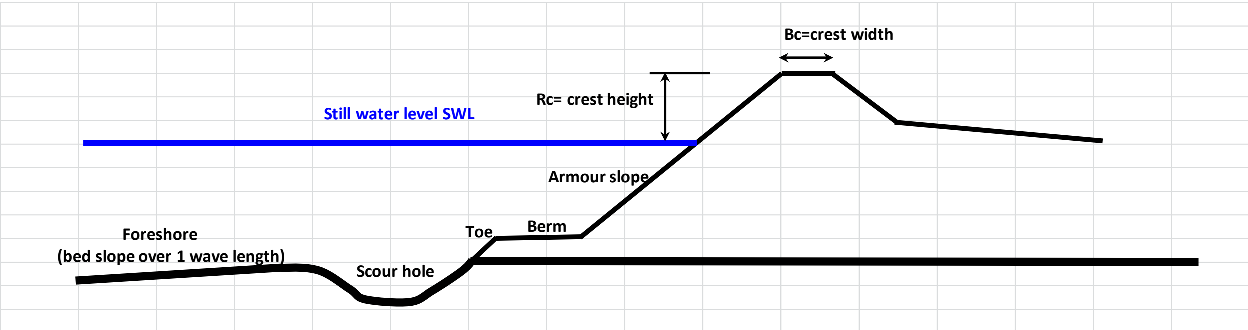


Figure 1.1 *Ccoastal structure*

All stability equations used herein are implemented in the **spreadsheet-model ARMOUR.xls (see ANNEX I)**, which can be used for the design of seadikes/revetments, high-crested and low-crested (emerged and submerged) breakwaters and toe/bottom protections.

2. HYDRODYNAMIC BOUNDARY CONDITIONS AND PROCESSES

2.1 Basic boundary conditions

The stable design of a coastal structure requires determination of various hydrodynamic parameters at the toe of the structure:

- Wave height, length, period (annual wave climate, extreme wave climates);
- Maximum water levels (including historic flood levels) due to tides and storm surges (setup);
- Maximum predicted sea level rise;
- Joint probability distribution of wave heights and water levels;
- Tide-, wind, and wave-driven currents;
- Subsidence.

2.2 Wave height parameters

2.2.1 Definitions

The wave attack at the toe of a structure depends on:

- Offshore wave climate;
- Nearshore bathymetry;
- Type and orientation of the structure (wave reflection);
- Maximum water levels.

The wave conditions at a structure site strongly depend on the water depth (including scour depth) at the toe of the structure.
Wind waves generated by near-field winds have wave periods smaller than about 15 s. Swell waves, generated by far-field winds, are long-period waves with periods in the range of 15 to 25 s and can travel over long distances without much deformation. Generally, swell waves are relatively large (with heights in the range of 2 to 4 m) at open ocean coasts. Swell waves at less exposed sea coasts are in the range of 1 to 2 m.

The type of wave action experienced by a structure may vary with position along the structure (shore-parallel or shore-connected structures). For this reason shore-connected structures should be divided in subsections; each with its own characteristic wave parameters.
The determination of wave impact forces on nearshore vertical structures (seawalls, caissons, piles) requires the estimation of the wave breaker height. As a general rule, the breaker height H_{br} in shallow water is related to the water depth $H_{br}= \gamma_{br}\, h$ with h= local water depth (including scour) and γ_{br} = wave breaking coefficient in the range of 0.5 to 1.5 depending on bed slope, structure slope and wave steepness (Van Rijn, 2011).

The statistical wave parameters used ($H_{33\%}$, $H_{10\%}$, $H_{5\%}$ or $H_{2\%}$) depend on the flexibility (allowable damage) of the structure involved, see Table 2.2.1. Rigid structures such as foundation piles should never fail and thus damage is not allowed. Thus, the highest possible wave height should be used as the design wave height. Flexible structures such as (berm) breakwaters protecting harbour basins and beach groins protecting beaches may have minor allowable damage during extreme events. The design of flexible structures generally includes the acceptance of minor damage associated with maintenance and overall economics of construction (availability of materials).
Most formulae to determine the stability of armour units are based on the significant wave height (H_s or $H_{1/3}$) at the toe of the structure. This wave height is defined as the mean of the highest 1/3 of the waves in a wave record of about 20 to 30 minutes.

Type of structure		Damage allowed	Design wave height
Rigid	(foundation pile)	No damage	$H_{5\%}$ to $H_{1\%}$
Semi-Rigid	(seawall, seadike)	Minimum damage	$H_{10\%}$ to $H_{5\%}$
Flexible	(berm breakwater, groin)	Minor damage	$H_{33\%}$ to $H_{10\%}$

$H_{33\%} = H_{1/3} = H_s$ = average of 33% highest wave heights
$H_{10\%} = 1.27\ H_{1/3}$ = average of 10% highest wave heights
$H_{5\%} = 1.37 H_{1/3}$ = average of 5% highest wave heights
$H_{1\%} = 1.76\ H_{1/3}$ = average of 1% highest wave heights
(assuming Rayleigh wave height distribution)
Table 2.2.1 *Design wave height*

In deep water where the wave heights approximately have a Rayleigh distribution, the significant wave height H_s is about equal to the spectral wave height $H_{mo} = 4(m_o)^{0.5}$ with m_o= area of wave energy density spectrum. In shallow water with breaking waves, the significant wave H_s is somewhat smaller than the H_{mo}-value. Extreme wave heights can be represented by $H_{10\%}$, $H_{5\%}$ and $H_{2\%}$.
Battjes and Groenendijk (2000) have presented a method to estimate the extreme wave heights in shallow water. Based on their results, the ratio $H_{1/3}/H_{2\%}$ is about 0.8 in shallow water with breaking waves ($H_{1/3}/h \cong 0.6$). Assuming Rayleigh distributed waves in deeper water ($H_{1/3}/h < 0.3$), it follows that $H_{1/3}/H_{2\%} \cong 0.7$. Using these values, the ratio $H_{1/3}/H_{2\%}$ can be tentatively described by a linear function, as follows: $H_{1/3}/H_{2\%} = 0.4(H_{1/3}/h) + 0.58$ yielding 0.7 for $H_s/h \leq 0.3$ and 0.82 for $H_s/h \geq 0.6$.
The wave period generally is represented by the peak wave period T_p of the wave spectrum (T_p= 1.1 to 1.2 T_{mean}). Wave run-up is most often based on the spectral wave period $T_{m-1,o}$ (= m_{-1}/m_o), whichs better represents the longer periods of the wave spectrum (in the case of relatively flat spectra of bi-modal spectra). In the case of a single peaked wave spectrum, it follows that: $T_p \cong 1.1\ T_{m-1,o}$.
Wave steepness is the ratio of wave height and wave length. Low steepness waves ($H/L \cong 0.01$) generally are long-period swell-type waves; while high-steepness waves ($H/L \cong 0.04$ to 0.06) are wind-induced waves. Wind waves breaking on a mild sloping foreshore may also become low steepness waves.
Wave breaking strongly depends on the ratio of the slope of the bottom or structure and the wave steepness. This ratio is known as the surf similarity parameter ξ:

$$\xi = \frac{\tan\alpha}{[H_{1/3}/L_o]^{0.5}} = \frac{1.25\ T_{m-1,o} \tan\alpha}{(H_{1/3})^{0.5}} \qquad (2.2.1)$$

with:
α = slope angle of bottom or structure;
$H_{1/3}$ = significant wave height at toe of structure (or H_{mo});
L_o = $[g/(2\pi)]\ [T_{m-1,o}]^2 = 1.56\ (T_{m-1,o})^2$ = deep water wave length;
$T_{m-1,o}$ = wave period (or T_p).

Equation(2.2.1) shows that the surf similarity parameter is linearly related to the wave period and $\tan\alpha$ and inversely related to the root of the wave height.

The type of wave breaking is:
- Spilling breaking on gentle slopes for $\xi < 0.2$;
- Plunging breaking with steep overhanging wave fronts $0.2 < \xi < 2.5$;
- Collapsing and surging breaking waves on very steep slopes $\xi > 2.5$.

2.2.2 Wave models

The nearshore wave heights in complex geometries such as a harbour basin protected by breakwaters can only be determined accurately by using a two dimensional horizontal wave model including refraction, shoaling, breaking and bottom friction (SWAN). A first estimate of the nearshore wave height can be obtained by using a one dimensional cross-shore wave energy model, which are most often used for straight, regular coasts. Preferably, a wave by wave model (CROSMOR-model, see Van Rijn et al. 2003) should be used to cover the total wave spectrum of low and high waves, including long wave energy. Such a wave model can also compute the wave-driven longshore and cross-shore currents.

Figure 2.2.1 shows an example of computed and measured wave heights ($H_{1/3}$ and $H_{1/10}$) along a sloping beach in a large-scale wave flume (Hannover GWK flume, Germany). The computed wave heights are based on a wave by wave model (CROSMOR-model), which computes all individual waves of the total wave spectrum. Both the measured $H_{1/3}$ and $H_{1/10}$ are well represented by the computed results of this wave model results (see also Van Rijn et al. 2011).

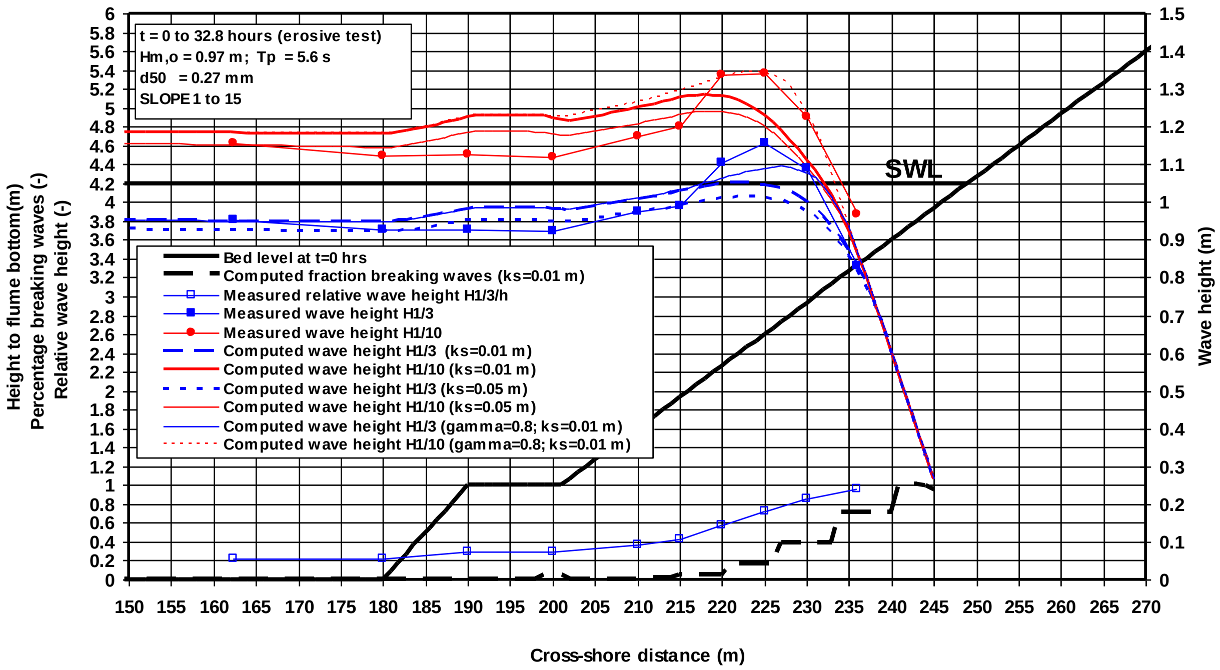


Figure 2.2.1 *Wave parameters at beach slope of 1 to 15, Hannover test, (CROSMOR-model)*

Figure 2.2.2 shows the cross-shore distributions (based on the CROSMOR-model; Van Rijn et al., 2003) of the significant wave height and the longshore velocity during storm conditions with an offshore wave height of 6 and 3 m (T_p = 11 and 8 s), storm set-up value of 1 m (no tide) and an offshore wave incidence angle of 30° for a coast protected by a seadike.

During major storm conditions with $H_{s,o}$ = 6 m, the wave height is almost constant up to the depth contour of -10 m. Landward of this depth, the wave height gradually decreases to a value of about 2 m at the toe of the dike (at x = 1980 m).

During minor storm conditions with $H_{s,o}$= 3 m, the wave height remains constant to the -4 m depth contour. The wave height at the toe of the dike is about 1.8 m. Thus, the wave height at the toe is almost the same for both events. The longshore velocity increases strongly landward of the -10 m depth contour where wave breaking becomes important (larger than 5% wave breaking). The longshore current velocity has a maximum value of about 1.6 m/s for $H_{s,o}$ = 6 m and about 1.7 m/s for $H_{s,o}$ = 3 m (offshore wave angle of 30°) just landward of the toe of the dike slope.

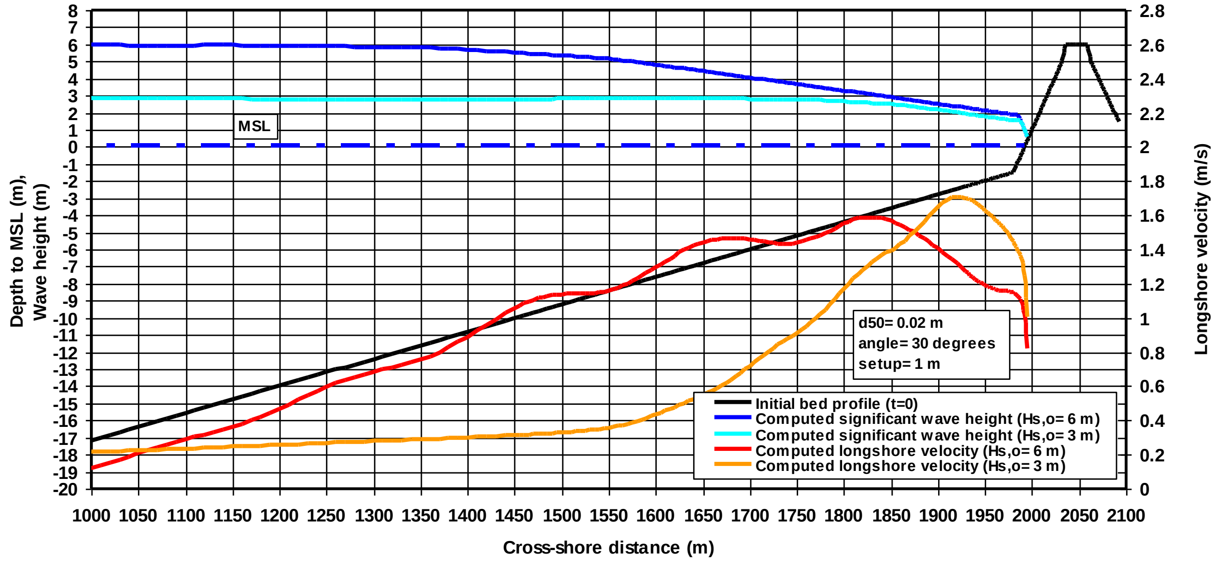


Figure 2.2.2 *Bed profile, wave height, longshore velocity for offshore wave height of $H_{s,o}$= 3 and 6 m; setup= 1 m; offshore wave incidence angle= 30^o for coast protected by seadike*

2.2.3 Design wave conditions

The design of an armour layer of rocks requires information of the complete distribution of extreme waves, as shown in **Figure 2.2.3**. This plot shows the annual extreme significant wave height in deep water as function of the return period for various coastal sites. Assuming that the lifetime of a structure is about 100 years, the extreme wave height with a return period of 100 years is often used as the design wave height. A relatively flat line of **Figure 2.2.3** implies that wave heights close to the 100 years-condition occur frequently, but will not be exceeded regularly. Usually, a relatively flat line is representative for shallow water with breaking waves during design conditions. In shallow water the wave height depends on the water depth: $H_s \cong 0.7(h_{MSL} + \Delta h_{surge})$ with h_{MSL}= water depth to mean sea level and Δh_{surge}= setup due to storm surge including tide. If the storm surge value is relatively small (about 1 to 2 m along open coasts), the wave height during extreme events will only be slightly larger than that during more frequent events (return period of 1 year). A steep line means that the annual extreme waves with a return period of 1 to 10 years are rather low, but extreme waves with a return period of 100 years are rather high. This is more representative for deep water. A joint probability plot of wave height and water level should be used to determine appropriate combinations of water level and wave height. **Figure 2.2.4** shows this plot for Pevensey Bay on the south coast along the English Channel of the UK (Van Rijn, 2010 and Van Rijn and Sutherland, 2011). The tidal range varies between 4 m (neap tide) and 7 m (spring tide). The joint probability curves represent a standard shape for conditions where the wave height and surge are weakly correlated. At Pevensey Bay the largest surges would probably come from the South-West (Atlantic Ocean) as would the largest offshore waves. However, Pevensey Bay is sheltered by Beachy Head and the offshore bathymetry; so the largest waves in deep water are not the largest waves inshore. The most severe wave conditions are for waves from the South, which would generate a smaller surge. It is highly unlikely that the highest waves will come at the same time as the highest water levels. In fact, water levels and wave heights are almost completely uncorrelated. For the uncorrelated case a 400 year return interval occurs for any combination of wave height and water level return interval that, when multiplied, gives 400 years. An example of a joint return interval of 400 years is a 100 year return interval for the wave height and 4 year return interval for the water level.

The most practical method for the stability design of structures is to make computations for a range of extreme conditions (scenarios) resulting in rock dimensions and damage rates for each scenario and for each section of the structure, see **Table 2.2.1**.

Damage of the structure is not acceptable for very small return periods (<20 years). Minor damage is acceptable for higher return periods (50 to 100 years).

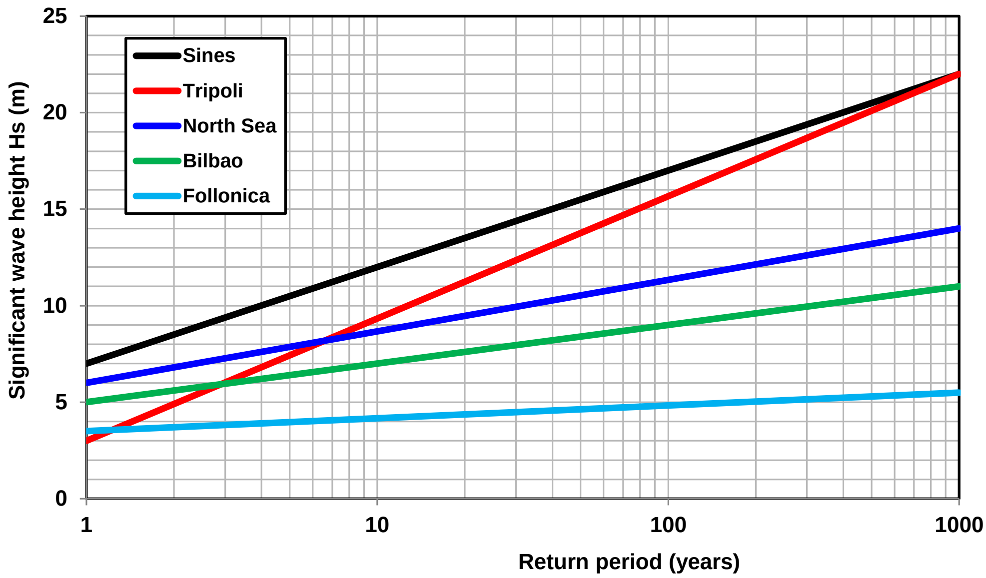


Figure 2.2.3 *Extreme (deep water) significant wave height as function of return period (PIANC 1992)*

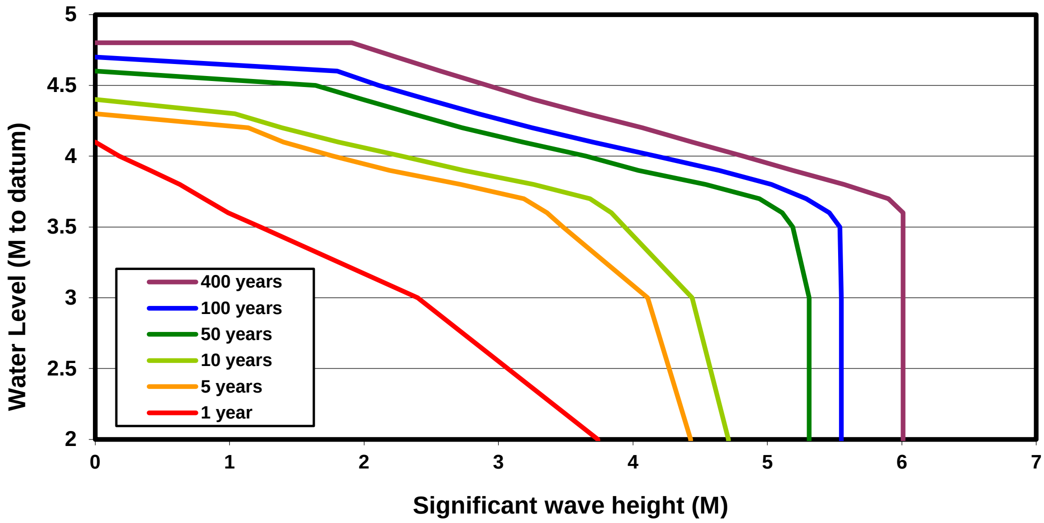


Figure 2.2.4 *Joint probability plot for wave height and water level (OD≅ mean sea level) for Pevensey Bay, UK*

Scenario	Water depth to MSL at toe of structure (m)	Maximum Water level above MSL (m)	Offshore Significant wave height (m)	Peak wave period (s)
1 Return period 10 years	….			
2 Return period 20 years				
3 Return period 50 years				
4 Return period 100 years				

Table 2.2.2 *Scenarios of extreme conditions*

2.2.4 Example wave computation

Three cross-shore wave models have been used to compute the wave height at a depth of 8 m based on given offshore wave height:

1) simple refraction-shoaling wave model;
2) Battjes-Janssen wave model (Van Rijn, 2011);
3) CROSMOR wave model (Van Rijn et al. 2003).

The refraction-shoaling model and the Battjes-Janssen model are implemented in the spreadsheet model **ARMOUR.xls**. All results are given in **Table 2.2.3**.
The computed wave heights of the B-J model and the CROSMOR-model are in good agreement. The simple refraction-shoaling model yields a wave height at the depth of 8 m which is about 25% too large.

Parameters		Refraction-shoaling wave model	Battjes-Janssen wave model	CROSMOR-wave model
Offshore water depth	h	28 m	28 m	28 m
Offshore significant wave height	$H_{s,o}$	7 m	7 m	7 m
Offshore wave angle to shore normal	θ	30°	30°	30°
Wave period	T_p	16 s	16 s	16 s
Bed slope from depth of 28 to 8 m		-	1 to 200	1 to 200
Bed roughness	k_s	-	0.01 m	0.01 m
Breaker coefficient	γ	0.7	0.7	variable
Wave height at depth = 8 m (breakerline	H_s	5.6 m (breaker line at depth= 8.9 m)	4.2 m	4.35 m
Wave angle at depth= 8 m	θ	17.3°	16.7°	16.6°
Longshore current at depth= 8 m	v	-	-	1.4 m/s

Table 2.2.3 *Computed wave heights*

2.3 Tides, storm surges and sea level rise

2.3.1 Tides

In oceans, seas and estuaries there is a cyclic rise and fall of the water surface, which is known as the vertical astronomical tide. This phenomenon can be seen as tidal wave propagating from deep water to shallow water near coasts. Basic phenomena affecting the propagation of tidal waves, are: reflection, refraction, amplification, deformation and damping.
Tidal waves are long waves (semi-diurnal to diurnal) generated by gravitational forces exerted by the Moon and the Sun. At most places the tide is a long wave with a period of about 12 hours and 25 minutes (semi-diurnal tide).

The tidal wave height between the crest and trough of the wave is known as the tidal range. Successive tides have different tidal ranges because the propagation of the tide is generated by the complicated motion of the Earth (around the Sun and around its own axis) and the Moon (around the Earth). Moreover, tidal wave propagation is affected by shoaling (funnelling) due to the decrease of the channel cross-section in narrowing estuaries, by damping due to bottom friction, by reflection against boundaries and by deformation due to differences in propagation velocities.

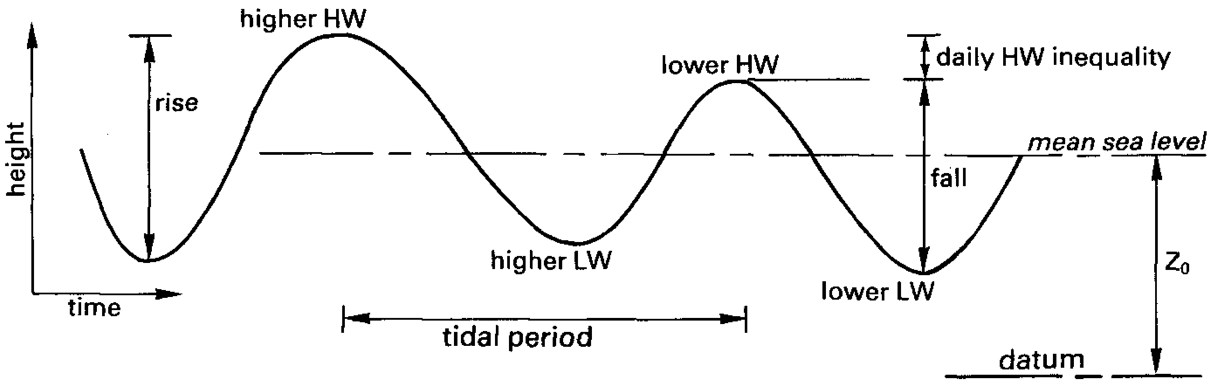


Figure 2.3.1 *Tidal curve*

The following definitions of tidal levels are given (see also **Figure 2.3.1**):

Mean Sea Level (M.S.L.)	= average level of the sea surface over a long period (≅18.6 years)
Mean Tide Level (M.T.L.)	= average of all high water levels and low waterlevels
Mean High Water (M.H.W.)	= average of the high water levels
Mean Low Water (M.L.W.)	= average of the low water levels
Lowest Astronomical Tide (L.A.T.)	= lowest water level which can occur
Mean Tidal Range	= difference between M.H.W. and M.L.W.
High Water Slack (HWS)	= time at which velocity changes from flood to ebb direction
Low Water Slack (LWS)	= time at which velocity changes from ebb to flood direction

The generation of the astronomical tide is the result of gravitational interaction between the Moon, the Sun and the Earth. Meteorological influences, which are random in occurrence, also affect local tidal motions. The orbit of the Moon around the Earth has a period of 29.6 days and both have an orbit around the Sun in 365.2 days.
There are 4 tides per day generated in the oceans. The Moon causes 2 tides and the Sun also causes 2 tides. The tides of the Sun are only half as high as those generated by the Moon. Even though the mass of the Sun is 27 million times greater than that of the Moon, the Moon is 390 times closer to the Earth resulting in a gravitational pull on the ocean that is twice as large as that of the Sun.
The tide is a long wave with a period of about 12 hours and 25 minutes (semi-diurnal tide) in most places.

The 25 minutes delay between two successive high tides is the result of the rotation of the Moon around the Earth. The Earth makes a half turn in 12 hours, but during those 12 hours the Moon has also moved. It takes about 25 minutes for the Earth to catch up to the new position of the Moon. The orbit of the Moon around the Earth is, on average, 29 days, 12 hours and 44 minutes (total of 708,8 hours to cover a circle of 360^o or a sector angle of 0.508^o per hour or 6.1^o per 12 hours). Thus, the Moon moves over a sector angle of 6.1^o per 12 hours. The Earth covers a circle of 360^o in 24 hours or a sector angle of 15^o per hour. So, it takes about 6.1/15 = 0.4 hour (25 minutes) for the Earth to catch up with the Moon.
Based on this, the tide shifts over 50 minutes per day of 24 hours; so each new day HW will be 50 minutes later. If the time of the first High Water (HW) at a certain location (semi-diurnal tide) is known at the day of New Moon (Spring tide), the time of the next HW is 6 hours and 12 minutes later and so on. The phase shift of 50 minutes per day is not constant but varies between 25 and and 75 minutes, because of the elliptical shape of the orbit of the Moon. Over the period of 29,6 days there are 2 spring tides and 2 neap tides; the period from spring tide to neap tide is, on average, 7.4 days.
The orbits of the Moon around the Earth and the Earth around the Sun are both elliptical, yielding a maximum and a minimum gravitational force. The axis of the Earth is inclined to the plane of its orbit around the Sun and the orbital plane of the Moon around the Earth is also inclined to the axis of the Earth. Consequently, the gravitational tide-generating force at a given location on Earth is a complicated but deterministic process. The largest force component is generated by the Moon and has a period of 12.25 hr (M_2-constituent). This force reaches its maximum value once in 29 days when the Moon is nearest to the Earth.
The decomposition of the tidal astronomical constituents (see **Table 2.3.1**) provides us with information of the frequencies of the various harmonic constituents of the tide at a given location. The magnitude and phase lag of these constituents could be determined from a theoretical model, but they can also be determined from observations at that location. This procedure is known as tidal analysis. Usually, water level registrations are used for tidal analysis because water level registrations are more easily obtained than current velocity measurements.
The **International Hydrographic Bureau** in Monaco publishes the harmonic constituents for many locations all over the world.
The **British Admiralty Tide Tables** provide information of the four principal harmonic constituents (M_2, S_2, K_1 and O_1) for many locations.
The periods and relative amplitudes of the seven major astronomical constituents, which account for about 83% of the total tide-generating force, are presented in **Table 2.3.1**.
In deep water the tidal phenomena can be completely described by a series of astronomical constituents. In shallow water near coasts and in estuaries, the tidal wave is deformed by the effect of shoaling, reflection and damping (bottom friction). These deformations can be described by Fourier series yielding additional higher harmonic tides which are known as **partial tides** or **shallow water tides**. These higher harmonic components can only be determined by tidal analysis of water level registrations at each location.
The neap-spring tidal cycle of 14.8 days is produced by the principal lunar and solar semi-diurnal components M_2 and S_2, and has a mean spring amplitude of M_2+S_2 and a mean neap amplitude of M_2–S_2.

Origin	**Symbol**	**Period (hours)**	**RelativeStrength (%)**
Main Lunar, semi-diurnal	M_2	12.42	100
Main Solar, semi-diurnal	S_2	12.00	46.6
Lunar Elliptic, semi-diurnal	N_2	12.66	19.2
Lunar-Solar, semi-diurnal	K_2	11.97	12.7
Lunar-Solar, diurnal	K_1	23.93	58.4
Main Lunar, diurnal	O_1	25.82	41.5
Main Solar, diurnal	P_1	24.07	19.4

Table 2.3.1 *Tidal constituents*

2.3.2 Storm surges

A storm is an atmospheric disturbance characterized by high wind speeds. A storm originating from the tropics is known as a ***tropical*** storm and a storm originating from a cold or warm front is known as an ***extra-tropical*** storm. A severe tropical storm with wind speeds larger than 120 km/hours is known as a ***hurricane***. Hurricanes generally are well-organized systems and have a circular wind pattern which revolves around a center or eye where the atmospheric pressure is low. The maximum wind speed does occur in a zone (at about 100 km) outward from the eye.
Storms and hurricanes can produce large rises in water level near coasts, which are known as ***storm surges*** or ***wind set-up***. In combination with springtide conditions the water level rise may reach a critical stage (flooding). Accurate storm surge predictions require the application of mathematical models including wind-induced forces and atmospheric pressure variations. Simple approximations can be made for schematized cases, see below.
Storm surges in addition to tidal water levels consist of various effects:

- water level rise due to onshore wind forces including resonance and amplification (funnelling);
- barometric water level rise due to variation in atmospheric pressure;
- wave-induced setup due to breaking waves near the shore.

Wind blowing towards the coast causes a gradual increase of the water level (**Figure 2.3.2**). Although the wind stress generally is small, its effect over a long distance can give a considerable water level increase. The fluid velocities near the water surface are in onshore direction; the fluid velocities near the bottom are in offshore direction when equilibrium is established. The discharge is zero everywhere (no net flow).

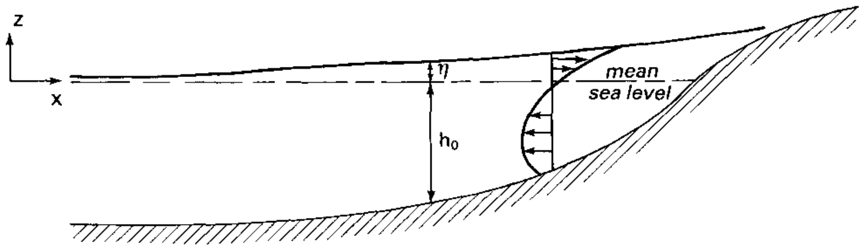


Figure 2.3.2 *Wind-induced circulation and water level set-up near the coast*

In the case of a constant wind stress $\tau_{s,x}$ over a distance L with constant water depth h_o and boundary condition η = 0 at x = 0, the storm surge level can be described by (Van Rijn 2011):

$$\eta/h_o = -1 + [1 + (2a\,x)/L]^{0.5} \qquad (2.3.1)$$

in which:

a = $(\alpha\, \tau_{s,x}\, L)/(\rho\, g\, h_o^2)$ and $\alpha \cong 0.8$;
η = water level setup due to wind stress;
$\tau_{s,x}$ = $\rho_a f_a \vec{W}_{10} |W_{10,x}| = \rho_a f_a (\vec{W}_{10})^2 \cos\theta$ = onshore wind shear stress at surface (x-direction);
$W_{10,x}$ = $\vec{W}_{10}\cos\theta$ = onshore wind velocity at 10 m above surface;
f_a = friction coefficient ($\cong$ 0.001 to 0.002);
ρ_a = density of air ($\cong$ 1.25 kg/m^3);
h_o = water depth to MSL;
L = wind fetch length;
θ = angle of wind vector $\vec{W}_{10}$ to shore normal.

This yields: $\eta/h_o \cong 0.05$ at x = L for a = 0.05,
$\eta/h_o \cong 0.01$ at x = L for a = 0.01.

Maximum wind set-up values that have been observed, are:

- η = 9.0 m in Biloxi east of New Orleans, Mississippi, USA (Katrina hurricane, 2005),
- η = 4.5 m in Galveston, Texas, USA (1900),
- η = 3 m in North Sea, Europe (1953),
- η = 2 m in Atlantic Ocean near USA coast (1962).

Storm surge levels along open ocean coasts are relatively low with values up to 2 m above mean sea level (MSL). Storm surge levels in funnel-shaped estuaries, bays and bights (southern North Sea bight) may be as large as 4 to 5 m above MSL. Surge levels can be derived from water level measurements in harbour basins by eliminating the tidal and barometric pressure effects.

2.3.3 River floods

Extreme river flood levels are important for the design of flood protection structures along tidal rivers. The occurrence of extreme floods are independent of the occurrence of extreme storms, but may occur are the same time.
The most dangerous situation with a very small probability of occurrence is an offshore storm during springtide and an extreme river flood level due to heavy rainfall and/or snow melt.

2.3.4 Sea level rise

Sea level rise presently (around 2000) is about 2 mm per year or 0.2 m per 100 years.
Future sea level rise (around 2100) due to global warming may be as large as 10 mm per year or 1 m per 100 years.
Assuming a lifetime of about 50 to 100 years for coastal structures, it is necessary to include sea level rise effects of the order of 0.5 to 1 m. In the case of very expensive large-scale flood protection structures with a lifetime of 200 years, the sea level rise effect to be taken into account, may be as large as 2 m.

2.4 Wave reflection

Wave reflection is the reflection of the incoming waves at the structure. Strong reflection of regular type of waves (swell waves) will lead to an increase of the wave height at the toe and thus to a higher crest level and larger stone sizes of the armour layer, while it may also lead to increased erosion of sediment at the toe of the structure. Close to shipping channels it may also lead to hinder for navigation.
In general, reflection from rubble mound breakwaters is fairly low.
The reflected wave height is expressed as : $H_r = K_r H_i$ with H_r= reflected wave height, H_i= incoming wave height at toe of structure and K_r=reflection coefficient.
Sigurdarson and Van der Meer (2013) have studied the wave reflection coefficients (in the range of 0.2 to 0.4) at berm breakwaters and found:

$K_r = 1.3 - 1.7\, s^{0.15}$ for hardly and partly reshaping berm breakwaters $H_{s,toe}/(\Delta D_{n,50}) < 2.5$ (2.4.1)
$K_r = 1.8 - 2.6\, s^{0.15}$ for hardly and partly reshaping berm breakwaters $H_{s,toe}/(\Delta D_{n50}) > 2.5$ (2.4.2)

with: s = wave steepness= $H_{s,toe}/L_o = (2\pi/g)\, H_{s,toe}/T_p^2$.

2.5 Wave runup

2.5.1 General formulae

Wave runup, defined as the runup height R above the still water level (**Figure 2.5.1**), occurs along all structures with a sloping surface (see **Figures 2.5.1**); the runup level strongly depends on the type of structure and the incident wave conditions.
Wave transmission (**Figure 2.5.1**) is the generation of wave motion behind the structure due to wave overtopping and wave penetration trough the (permeable) structure.

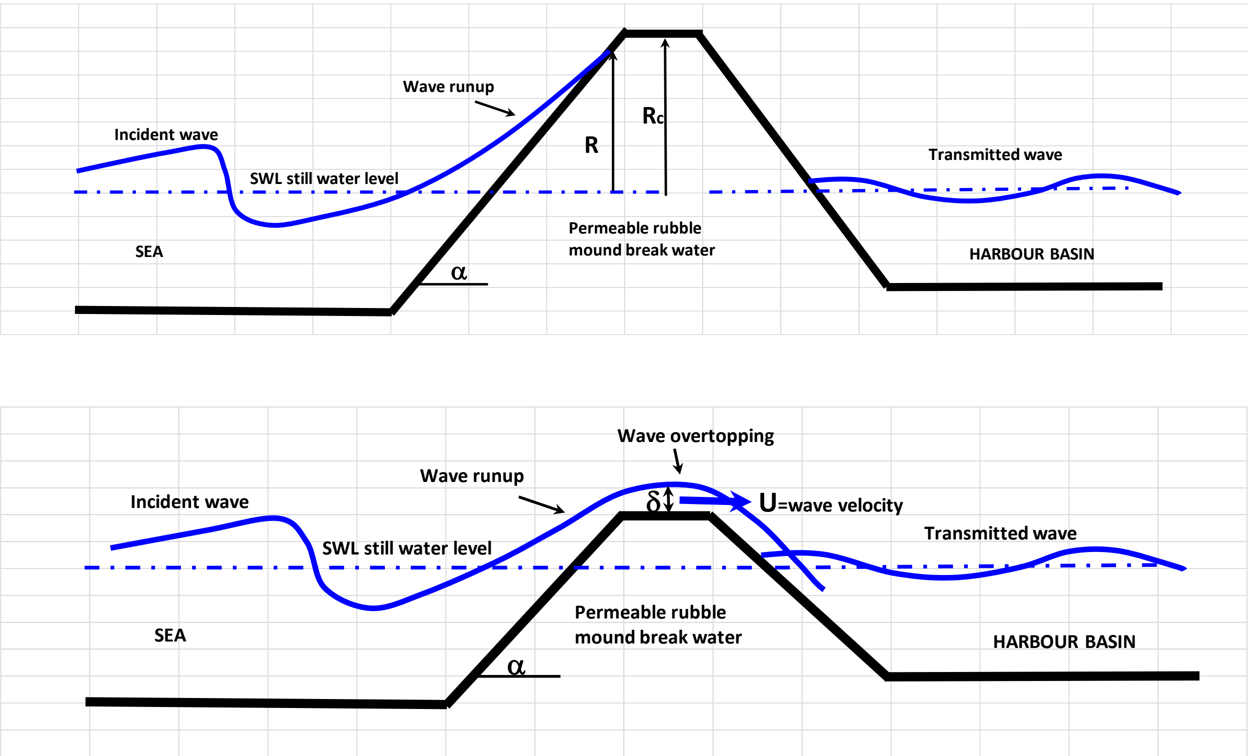


Figure 2.5.1 *Wave runup, wave overtopping and wave transmission*

If a seadike is designed for flood protection, the structure should have a high crest level well above the maximum wave runup level during design storm conditions. Wave overtopping should be negligible (< 1 l/m/s), as wave overtopping often is a threat to the rear (erodible) side of a dike.
If a (berm) breakwater is designed to protect a harbour basin against wave motion, minor wave overtopping in the range of 1 to 10 litres/m/s may be allowed during design storm conditions resulting in a lower crest level and hence lower construction costs. Wave transmission inside the harbour basin due to wave overtopping should be negligible during daily operational conditions, but transmitted wave heights up to 1 m may be acceptable during extreme storm events.

In this section only the general aspect of the above-mentioned processes are discussed briefly. As these processes are strongly related to the type of structure and the seaward slope of the structure, detailed information can only be obtained when the design of a specific structure is known.
The crest level of a high-crested structure strongly depends on the maximum water level and wave run-up. During design conditions only a small percentage of the waves may reach the crest of a structure.

The wave runup depends on:

- the incident wave characteristics,
- the geometry of the structure (slope, crest height and width, slope of foreshore),
- the type of structure (rubble mound or smooth-faced; permeable or impermeable).

When high waves approach a nearshore structure during a storm event, the majority of the wave energy is dissipated across the surf zone by wave breaking. However, a portion of that energy is converted into potential energy in the form of runup along the seaward surface of a sloping structure.

Generally, the vertical wave runup height above the still water level (SWL) is defined as the run-up level which is exceeded by only 2% of the incident waves ($R_{2\%}$).

Runup is caused by two different processes (see **Figure 2.5.2**):

- maximum wave set-up (h'), which is the maximum time-averaged water level elevation at the shoreline with respect to mean water level (MSL);
- swash oscillations (s_t), which are the time-varying vertical fluctuations about the temporal mean value (setup water level); the runup is approximately equal to $R = h' + 0.5H_{swash}$ with $H_{swash} = 2s_{max}$ = swash height.

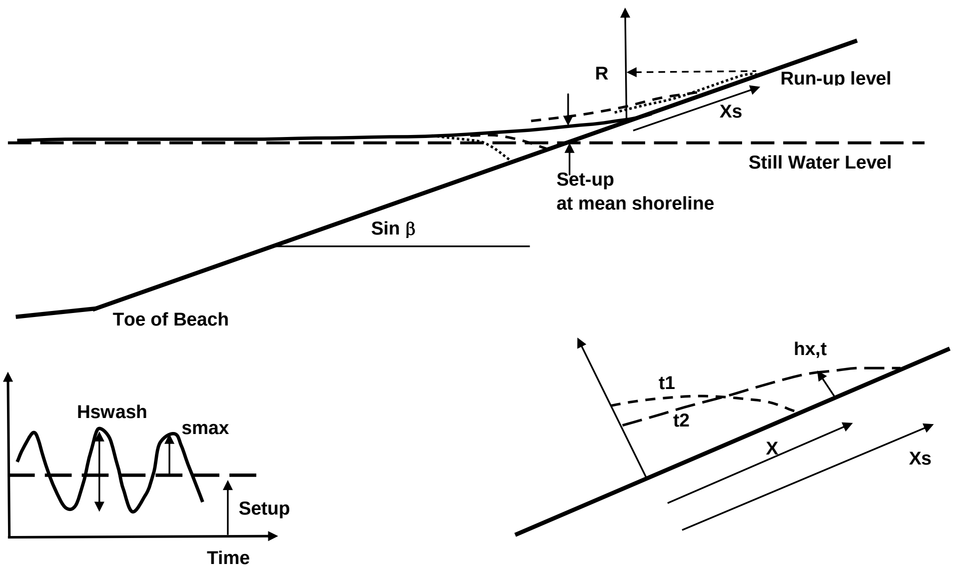


Figure 2.5.2 *Wave run-up processes along a structure*

Laboratory measurements with monochromatic waves on a plane beach have shown that the vertical swash height R increases with growing incident wave height until R reaches a threshold value. Any additional input of the incident wave energy is then dissipated by wave breaking in the surf zone and does not result in further growth of the vertical swash and runup, i.e the swash is saturated.

Usually, the runup height up to the threshold value is represented as:

$$R = \zeta_o \gamma H_i \qquad (2.5.1)$$

in which:

R = runup height measured vertically from still water level (including wave setup) to runup point;
H_i = incident wave height at toe of structure,
ζ_o = $\tan\alpha/s^{0.5}$ = surf similarity parameter;
s = $(H_i/L_o)^{-0.5}$ = wave steepness;
L_o = wave length in deep water;
$\tan\alpha$ = slope of structure;
γ = proportionality coefficient.

Low ζ_o-values (< 0.3) typically indicate dissipative conditions (high breaking waves on flat slopes), while higher values (> 1) indicate more reflective conditionss (breaking waves on steep slopes).
In dissipative conditions, infragravity energy (with periods between 20 and 200 s) generally tends to dominate the inner surf zone.
Various field studies have shown the important contributions of the incident wave periods (T < 20 s) and the infragravity wave periods (T > 20 s) to the runup height above SWL.

Various empirical formulae based on laboratory tests and field data, are available to estimate the wave runup level. Because of the large number of variables involved, a complete theoretical description is not possible. Often, additional laboratory tests for specific conditions and geomteries are required to obtain accurate results.

Van Gent (2001) has presented runup data for steep slope structures such as dikes with shallow foreshores based on local incident wave parameters. Various types of foreshores were tested in a wave basin: foreshore of 1 to 100 with a dike slope of 1 to 4; foreshore of 1 to 100 with a dike slope of 1 to 2.5 and foreshore of 1 to 250 with a dike slope of 1 to 2.5. The test programme consisted of tests with single and double-peaked wave energy spectra, represented by a train of approximately 1,000 waves. The water level was varied to have different water depth values at the toe of the dike.

The experimental results for steep, smooth slope structures can be represented by (**Figure 2.5.3**):

$$R_{2\%}/H_{s,toe} = 2.3\ \gamma_s\ \gamma_{berm}\ \gamma_{\beta eta}\ (\zeta)^{0.3} \quad \text{for } 1 < \zeta < 30 \qquad (2.5.2)$$

with:

ζ = $\tan\alpha/s^{0.5}$ = surf similarity parameter;
s = $H_{s,toe}/L_o$ = wave steepness;
L_o = $T^2_{m-1}\,g/(2\pi)$ wave length in deep water;
$H_{s,toe}$ = significant wave height at toe of the structure (or spectral wave height H_{mo});
T_{m-1} = wave period based on zero-th and first negative spectral moment of the incident waves at the toe of the structure (= 0.9 T_p for single peaked spectrum);
T_p = wave period of peak of spectrum;
α = slope angle of structure;
γ_{berm} = berm factor (see Section 2.5.3);
$\gamma_{\beta eta}$ = oblique wave factor (see Section 2.5.4);
γ_s = safety factor (about 1.2 to use upper enveloppe of data).

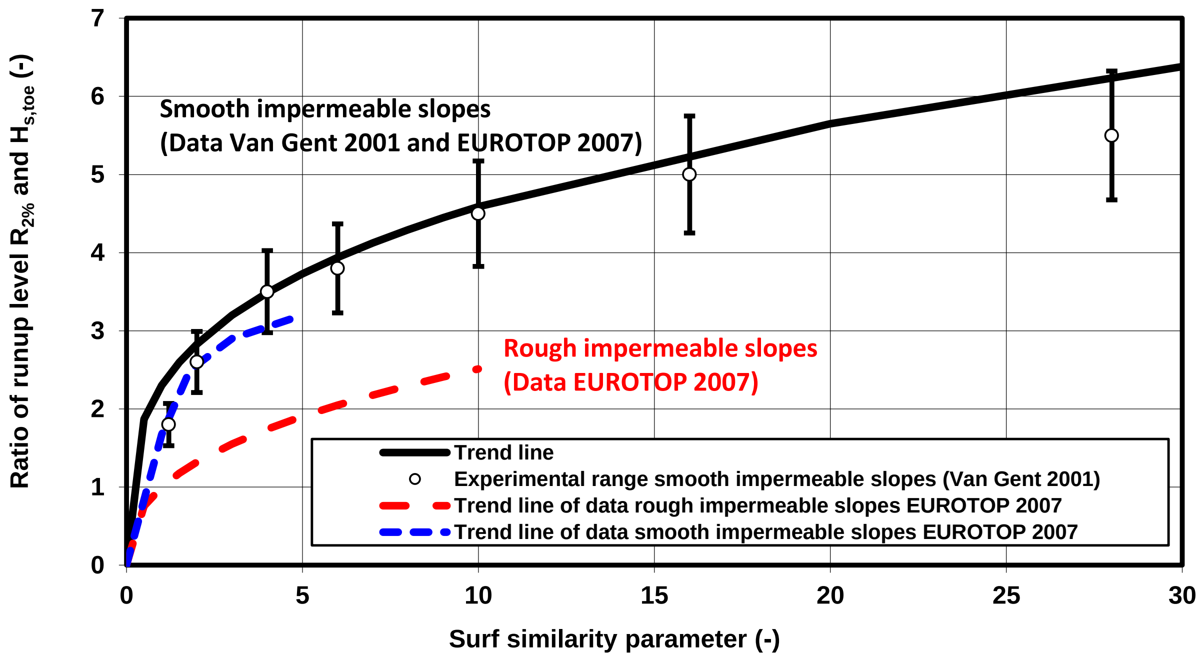


Figure 2.5.3 *Runup level for smooth and rough slopes slopes as function of surf similarity parameter*

The run-up level $R_{2\%}$ according to Equation (2.5.2) varies roughly from $1H_{s,toe}$ to $5H_{s,toe}$ depending on the value of the surf similarity parameter. The influence of the wave energy spectrum can be accounted for by using the spectral wave period T_{m-1} of the incident waves at the toe of the structure.
During storm conditions with a significant offshore wave height of about 6 m (peak period of 11 s), the significant wave height at the toe of a structure may be about 2 m (see **Figure 2.2.2**) resulting in a ζ_o-value of 2 to 3 and thus $R/H_{s,toe} \cong 2.5$ to 3 and $R \cong 5$ to 6 m above the mean water level, based on Equation (2.5.2).
The runup values along rough rock-type slopes are significantly smaller due to friction and infiltration processes.

An expression similar to Equation (2.5.2) can be fitted to the available wave runup data (EUROTOP 2007) for rough slopes including rock-type slopes in the range of 1 to 2 and 1 to 4, yielding (red curve of **Figure 2.5.3**):

$$R_{2\%}/H_{s,toe} = \gamma_s\ \gamma_p\ \gamma_{berm}\ \gamma_{\beta eta}\ (\zeta)^{0.4} \qquad (2.5.3)$$

ζ = $\tan\alpha/s^{0.5}$ = surf similarity parameter;
s = $H_{s,toe}/L_o$ = wave steepness = surf similarity parameter based on the T_{m-1} wave period;
L_o = $(g/(2\pi)T^2_{m-1}$ = wave length in deep water;
$H_{s,toe}$ = significant wave height at toe of the structure (or spectral wave height H_{mo});
T_{m-1} = wave period based on zero-th and first negative spectral moment of the incident waves at the toe of the structure (= 0.9 T_p for single peaked spectrum);
T_p = wave period of peak of spectrum;
α = slope angle of structure;
γ_p = permeability factor (=1 for impermeable structures and 0.8 for permeable structures);
γ_{berm} = berm factor (see Section 2.5.3);
$\gamma_{\beta eta}$ = oblique wave factor (see Section 2.5.4);
γ_s = safety factor (about 1.2 to use upper enveloppe of data).

Based on the EUROTOP Manual 2007, the wave runup for smooth and rough slopes is described by (see blue curve for smooth slopes of **Figure 2.5.3**):

$$R_{2\%}/H_{s,toe} = \gamma_s\, C_1\, \gamma_r\, \gamma_{berm}\, \gamma_{\beta eta}\, \zeta \qquad \text{for } \zeta < 1.7 \qquad (2.5.4a)$$

$$R_{2\%}/H_{s,toe} = \gamma_s\, \gamma_r\, \gamma_{berm}\, \gamma_{\beta eta}\, (C_2 - C_3/\zeta^{0.5}) \qquad \text{for } \zeta > 1.7 \qquad (2.5.4b)$$

with:

γ_r = roughness factor (see Section 2.5.2 and **Table 2.5.1**), γ_r = 1 for smooth slope;
γ_{berm} = berm factor (see Section 2.5.3);
$\gamma_{\beta eta}$ = oblique wave factor (see Section 2.5.4);
γ_s = safety factor (about 1.1 to 1.2 to use upper enveloppe of data).

C_1=1.65 (σ_{c1}≅0.1), C_2=4.0 (σ_{c2}≅0.2), C_3= 1.5 (σ_{c3}≅0) and γ_s = 1 in the case of probabilistic design method,
γ_s = 1.2 in the case of deterministic design method.

Using a deterministic design method, the model coefficients C_1, C_2 and C_3 should be somewhat larger to include a safety margin (upper enveloppe of experimental range). This can be represented by using a safety factor equal to 1.2. Using a safe factor of 1.5, a very conservative estimate is obtained.

Using a probabilistic design method, each input parameter is represented by a mean value and a standard deviation; the coefficients of the functional relationships involved are also represented by a mean value and standard deviation. Many computations (minimum 10) are made using arbitrary selections (drawings based on a random number generator) from all variables (Monte Carlo Simulations). The mean and standard deviation are computed from the results of all computations.

2.5.2 Effect of rough slopes

The wave runup decreases with increasing roughness.
The wave runup for rock-type slopes can be computed by Equation (2.5.3) for rough slopes.
The wave runup can also be computed by Equation (2.5.4), which is valid for both smooth and rough slopes.
Using Equation (2.5.4), the roughness is taken into account by a roughness factor (γ_r).

Table 2.5.1 shows some roughness reduction values defined as $\gamma_r = R_{2\%,rough\ slope}/R_{2\%,smooth\ slope}$ based on many laboratory tests (Shore Protection Manual, 1984 and EUROTOP Manual 2007).
This means that very rough rock slopes can have a much lower crest level. Often, roughness elements are constructed on smooth slopes to reduce the wave run-up, see **Figure 4.1.1**.

Most smooth slopes have roughness elements (blocks) at the upper part of the slope to reduce the wave runup and wave overtopping rate. Some values of the γ_r-factor are given in **Table 2.5.1**.

The roughnes elements are placed in the zone 0.25 to 0.5 $H_{s,toe}$ above the design water level (SWL). The height of the roughness blocks is of the order of 0.3 to 0.5 m (about 0.1 to 0.2 $H_{s,toe}$). The width of the blocks is about 2 to 3 times the height. The spacing of the blocks is about 3 to 5 times the roughness height. The dimensions and arrangement should be optimized by laboratory scale tests.

Type of surface slope	Placement method	Reduction factor γ_r
Concrete surface	-	1
Asphalt surface	-	1
Grass surface	-	0.9
Basalt blocks	closely fitted	0.9
Concrete blocks	Closely fitted	0.9
Small blocks over 4% of surface	-	0.85
Small blocks over 10% of surface	-	0.8
Small ribs	-	0.75
One layer of quarrystone on impermeable foundation layer	random	0.75
Three layer of quarrystone on impermeable foundation layer	random	0.6
Quarrystone	fitted random	0.75 0.5
Concrete armour units	random	0.45

Table 2.5.1 *Reduction factor for wave runup and wave overtopping along smooth and rough slopes*

2.5.3 Effect of composite slopes and berms

Many seadikes have a seaward surface consisting of different slopes interrupted by one or more berms. Various methods are available to determine a representative slope (EUROTOP Manual 2007).
Herein, it is proposed to determine the representative slope angle as the angle of the line between two points at a distance $1.5H_{s,toe}$ below and above the still water level (see **Figure 2.5.4**), as follows:

$$\tan(\alpha_r)= 3H_{s,toe}/(L - B) \qquad (2.5.5)$$

with: L= horizontal distance between two points at 1.5 Hs,toe below and above SWL, B= berm width.
A berm above the design water level reduces the wave runup and overtopping during a storm event, depending on the berm width (see **Figure 2.5.4**): γ_{berm} = 0.6 for very wide berms (berm width = $0.25L_{toe}$ with L_{toe} = wave length at toe) to γ_{berm} = 1 for very small berms. Wide berms are very effective.
Berms should be placed at a high level to be effective, just above the water level with a return period of 100 years (2 to 4 m above mean sea level MSL).

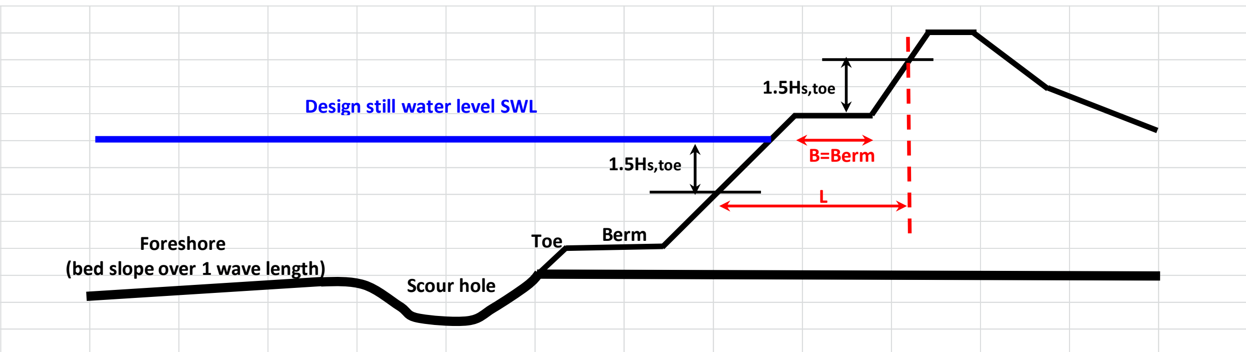


Figure 2.5.4 *Effect of composite slopes*

The berm effect can be simply expressed, as:

$$\gamma_{berm} = (1H_{s,toe}/B)^{0.3} \qquad \text{for } B > 1H_{s,,toe} \qquad (2.5.6)$$

This yields: $\gamma_{berm} = 1$ for $B \leq 1H_{s,toe}$, $\gamma_{berm} = 0.8$ for $B = 2H_{s,toe}$, $\gamma_{berm} = 0.6$ for $B \geq 5H_{s,,toe}$.

2.5.4 Effect of oblique wave attack

Based on laboratory test results, the effect of oblique waves on wave runup can be taken into account by (EUROTOP Manual 2007):

$$\gamma_{\beta eta} = 1 - 0.0025|\beta| \qquad \text{for } 0 \leq \beta < 80^o \qquad (2.5.7a)$$
$$\gamma_{\beta eta} = 0.8 \qquad \text{for } \beta \geq 80^o \qquad (2.5.7b)$$

with: β = wave angle to shore normal (in degrees), see **Figure 2.5.5**.

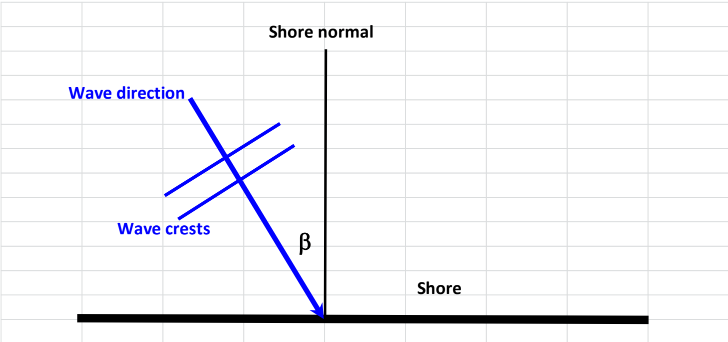


Figure 2.5.5 *Wave direction*

2.6 Wave overtopping

2.6.1 General formulae

Wave overtopping occurs at structures with a relatively low crest (see **Figure 2.5.1**); the overtopping rate strongly depends on the type of structure, the crest level and the incident wave conditions.
Wave overtopping does not occur if the runup height R is smaller than the crest height R_c above still water level ($R < R_c$).

Wave overtopping consists of:
- continuous sheet of water during passage of the wave crest (green water);
- splash water and spray droplets (white water) generated by wave breaking somewhat further away from the crest.

Wave overtopping is of main concern for flood protection structures such as vertical seawalls and sloping dikes /revetments/embankments. These types of structures should have a high crest level to minimize wave overtopping.
The wave overtopping rate is the time-averaged mean rate of water passing the crest per unit length of the structure. In practice, there is no constant rate of water passing the crest during overtopping conditions, but the process is random in volume and time due to the randomness of the incoming waves.

Table 2.6.1 presents damage levels (based on wave overtopping simulator tests) in relation to the overtopping rate.

Wave overtopping (litres per second per m crestlength)	Type of overtopping	Damage to erodible surface
< 0.1	-	No damage
0.1 to 1.0	-	Clay surface: first signs of erosion Grass surface: no damage
1.0 to 10	Film of water passing over crest; walking on crest is possible; Acceptable once a year	Clay surface: moderate erosion Grass surface: very minor erosion Breakwater: minor wave transmission
10 to 30	Thin layer of water of 0.01 to 0.03 m passing over crest with velocities of about 1 to 2 m/s; Driving at low speed is possible; Acceptable once in 10 years	Clay surface: significant erosion Grass surface: minor erosion; most grass layers do not show significant damage up to 30 l/m/s (Van der Meer 2011) Breakwater: considerable transmission
30 to 100	Layer of water with thickness of 0.03 to 0.1 m passing over crest with velocities of 2 to 3 m/s; driving is dangerous; loose objects will be washed away; Acceptable once in 30 years	Clay surface: armour protection is required Grass surface: armour protection is required Breakwater: major wave transmission

Table 2.6.1 *Damage due to wave overtopping*

Percentage of overtopping waves

Figure 2.6.1 shows the percentage of overtopping waves as function of the relative crest height parameter $R_c D_n/H_{s,toe}^2$ based on laboratory tests of conventional breakwaters armoured with tetrapods and accropods and a relative low-crested concrete superstructure; D_n= nominal cubical size of the armour units; H_s= significant incident wave height at toe of structure; R_c= difference between water level on seaward slope of structure and crest level of structure (freeboard), see **Figure 2.5.1**.

The percentage overtopping waves for straight, smooth and impermeable slopes (seadikes and revetments) can be computed by (EUROTOP manual 2007):

$$P_{ow} = 100 \exp[- A (R_c/R_{2\%})^2] \quad (2.6.1a)$$

with:
p_{ow} = percentage overtopping waves (0 to 100%); R_c = crest height above SWL (m); $R_{2\%}$ = wave runup height (m); A = -ln(0.02) = 3.91

The percentage overtopping waves for straight rough slopes (breakwaters) as function of the crest height can be computed by (EUROTOP manual 2007):

$$P_{ow} = 100 \exp[-10 [(R_c D_n)/(H_{s,toe}^2)]^{1.4}] \quad (2.6.1b)$$

with:
p_{ow} = percentage overtopping waves (0 to 100%), see **Figure 2.6.1**; D_n = nominal cubical size of the armour units; $H_{s,toe}$ = significant incident wave height at toe of structure; R_c = difference between water level on seaward slope of structure and crest level of structure (freeboard), see **Figure 2.5.1**.

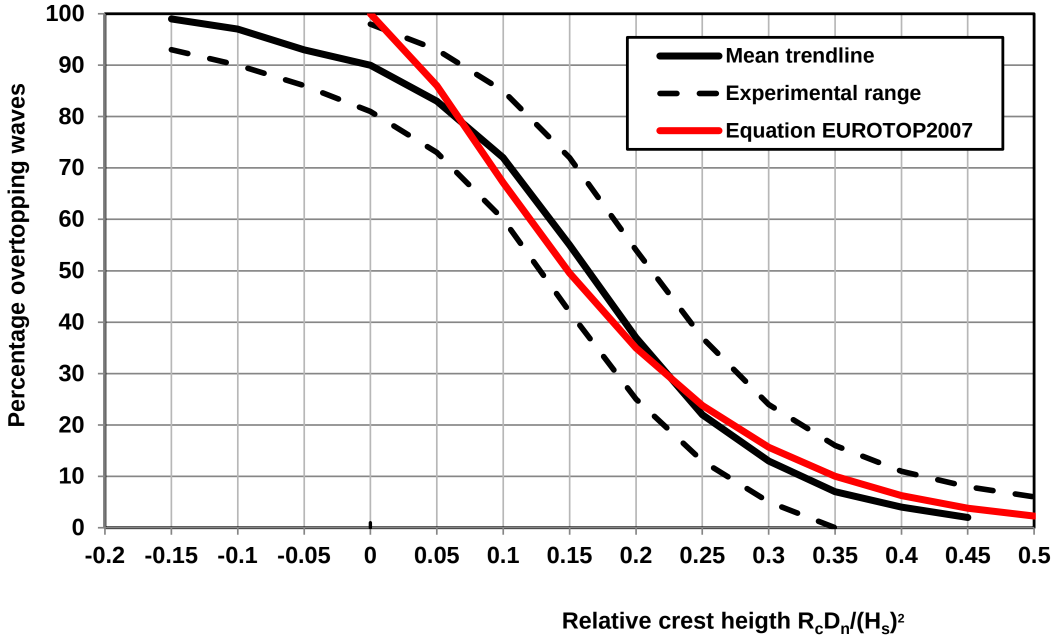


Figure 2.6.1 *Percentage of overtopping waves as function of crest height parameter for conventional breakwater (Van der Meer, 1998)*

Wave overtopping formulae

A simple approach to determine the wave overtopping rate (q_w) per unit length of structure, is as follows:

$$q_{wo} = p_{wo}\, e\, \delta_w\, u_w \qquad (2.6.2)$$

with:

- q_{wo} = wave overtopping rate (in $m^3/m/s$);
- p_{wo} = percentage of overtopping waves being a function of R_c/H_i ($p_w \cong 0.1$ to 0.2);
- e = efficiency factor ($\cong$0.3; as only the wave crest is involved);
- δ_w = thickness of wave layer above the crest of the structure ($0.1H_i$ to $0.5\,H_i$ depending on R_c/H_i);
- H_i = incident wave height (1 to 3 m);
- u_w = $\varepsilon\,(gH_i)^{0.5}$ = wave velocity above crest of structure;
- ε = coefficient ($\cong$ 0.5) depending on R_c/H_i.

Using these values, the wave overtopping rate is in the range of 0.005 to 0.25 $m^3/m/s$ or 5 to 250 litres/m/s (per unit length of structure). These estimates show a crude range of overtopping rates during storm events for low-crested structures. For a seadike the overtopping rate should not be larger than about 1 litre/m/s.

Assuming that: $\delta_w/H_i = a_1 \exp[(-a_2\, R_c/H_i)]$, it follows that:

$$q_{wo} = p_w\, e\, \varepsilon\, (gH_i)^{0.5}\, a_1 \exp[(-a_2\, R_c/H_i)] = A\, (gH_i^3)^{0.5} \exp[(-B\, R_c/H_i)]$$

with: A, B= bulk coefficients to be determined from laboratory tests.

Thus, the principal equation for the wave overtopping rate reads, as (see also Eurotop, 2007):

$$q_{wo} = A\, (gH_{s,toe}^3)^{0.5} \exp(-B\, R_c/H_{s,toe}) \qquad (2.6.3)$$

with:
q_{wo} = wave overtopping rate (in $m^3/m/s$);
$H_{s,toe}$ = incident significant wave height;
R_c = crest height above SWL =freeboard (see **Figure 2.5.1**);
A,B = coefficient related to a specific type of structure (laboratory tests).

Based on the EUROTOP Manual 2007, the wave overtopping rate for smooth and rough impermeable slopes can be described by:

$$q_{wo} = \gamma_s\, \xi\, [A_1/(\tan\alpha)^{0.5}]\, (gH_{s,toe}^3)^{0.5} \exp\{(-A_2 R_c/(\xi\, \gamma_{berm}\gamma_r\gamma_{\beta eta} H_{s,toe})\} \quad \text{for } \xi < 1.8 \qquad (2.6.4a)$$

$$q_{wo,max} = \gamma_s\, A_3\, (gH_{s,toe}^3)^{0.5} \exp\{(-A_4\, R_c/(\gamma_r\, \gamma_{berm}\, \gamma_{\beta eta}\, H_{s,toe})\} \quad \text{for } \xi > 1.8 \text{ and } < 7 \qquad (2.6.4b)$$

For very shallow foreshores ($\xi > 7$):

$$q_{wo} = \gamma_s\, A_5\, (gH_{s,toe}^3)^{0.5} \exp\{(-A_6\, R_c/(\gamma_r\, \gamma_{berm}\, \gamma_{\beta eta}\, (0.33+0.022\xi)H_{s,toe})\} \quad \text{for } \xi > 7 \qquad (2.6.5)$$

with:
q_{wo} = time-averaged wave overtopping rate (in $m^3/m/s$);
$H_{s,toe}$ = incident significant wave height at toe;
ξ = surf similarity parameter (see Equation 2.2.1);
R_c = crest height above SWL =freeboard (see **Figure 2.5.1**);
α = slope angle of structure;
γ_r = roughness factor (see Section 2.6.2, **Tables 2.5.1** and **2.6.3**), γ_r = 1 for smooth slope;
γ_{berm} = berm factor (see Section 2.6.3); γ_{berm} = 1 for no berm;
$\gamma_{\beta eta}$ = oblique wave factor (see Section 2.6.4); $\gamma_{\beta eta}$ = 1 for waves perpendicular to structure;
γ_s = safety factor (about 1.1 to 1.2 to use upper envelope of data).
A = coefficients, see **Table 2.6.1**.

Equation (2.6.4a) shows that: $q_{wo} \approx \xi$ and thus: $q_{wo} \approx T$. If H_s = constant, the wave overtopping rate increases with increasing wave period T. As the wave period T may have an inaccuracy up to 20%, it is wise to use a conservative estimate of the wave period (safety factor of 1.1 to 1.2 for T). Equation (2.6.4b) is not dependent on the wave period. The prescribed wave period is the T_{m-1} period, which better represents the longer wave components of the wave spectrum. This is of importance for the surf zone where the spectrum may be relatively wide (presence of waves with the approximately same height but rather different periods).
The coefficients A_1 to A_6 of Equations (2.6.4) and (2.6.5) are given in **Table 2.6.2**. The coefficients of the deterministic design method are slightly different to obtain a conservative estimate. To obtain the upper enveloppe of the data, an additional safety factor of about 1.5 should be used.
The coefficients of the probabilistic method represents a curve through all data points (best fit). If the coefficients of the probabilistic method are used for deterministic computations, the safety factor should be about 2. If A_6= 1.11 , then Equation(2.6.5) is equal to Eq. (2.6.4b) for $\xi > 7$.
Calculation tools for wave overtopping rate can be used at: www.overtopping-manual.com

Using a probabilistic design method, each input parameter is represented by a mean value and a standard deviation; the coefficients of the functional relationships involved are also represented by a mean value and standard deviation. Many computations (minimum 10) are made using arbitrary selections (drawings based on a random number generator) from all variables (Monte Carlo Simulations). The mean and standard deviation are computed from the results of all computations.

Coefficients	Probabilistic design method	Deterministic design method
A_1	0.067; $\sigma_{A1} = 0$	0.067
A_2	4.75; $\sigma_{A2} = 0.5$	4.3
A_3	0.2; $\sigma_{A3} = 0$	0.2
A_4	2.6; $\sigma_{A4} = 0.35$	2.3
A_5	0.12; $\sigma_{A5} = 0.03$	0.2
A_6	1; $\sigma_{A6} = 0.15$	1.11

γ_s = safety factor (about 1.1 to 1.2)

Table 2.6.2 *Coefficients*

Figure 2.6.2 shows the dimensionless overtopping rate as function of the relative crest height R_c/H_i. based on data from EUROTOP (2007). It can be observed that the wave overtopping rate is largest for smooth slopes and smallest for gentle rubble mound slopes. Rough permeable surfaces strongly reduces the overtopping rate.

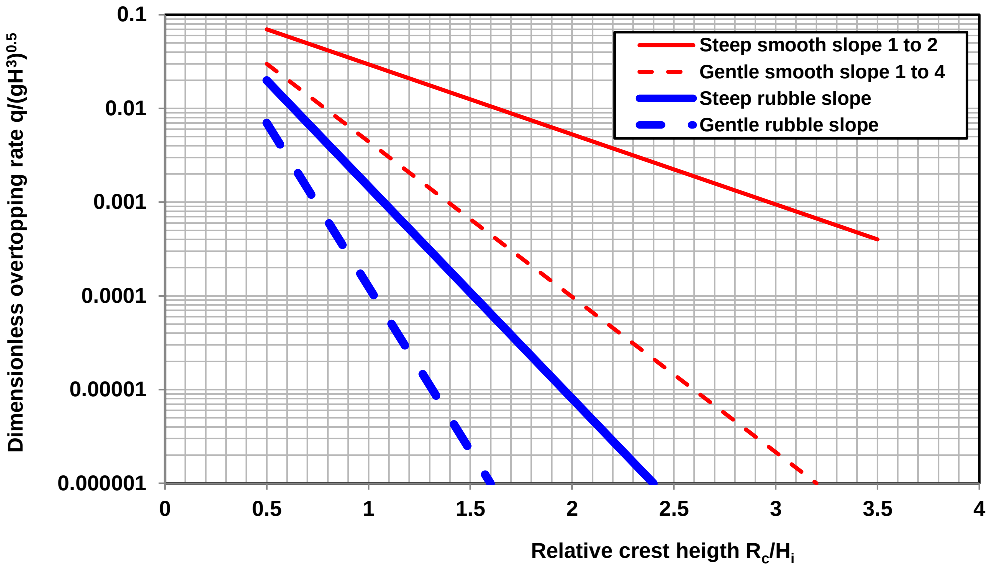


Figure 2.6.2 *Dimensionless overtopping rate* $q_{wo}/(gH_i^3)^{0.5}$ *as function of* R_c/H_i *(EUROTOP, 2007)*

According to the EUROTOP Manual 2007, the wave overtopping rate for straight, rough slopes of permeable breakwaters with a crest width of maximum $B_c = 3D_n$ can also be computed by Equation (2.6.4b), which is valid for steep, rough slopes as the surf similarity parameter for steep slopes is in the range of 1.8 to 7 (see **Figure 2.6.3**). The equation to be used, reads as::

$$q_{wo} = 0.2\ \gamma_{crest}\,(gH_{s,toe}^3)^{0.5}\ \exp\{(-2.3\ \gamma_s\ R_c/(\gamma_r\ \gamma_{berm}\ \gamma_{\beta eta}\ H_{s,toe})\}\quad \text{for } R_c/H_{s,toe} > 0 \qquad (2.6.6)$$

with:

q_{wo} = time-averaged wave overtopping rate (in $m^3/m/s$);
$H_{s,toe}$ = incident significant wave height at toe;
ξ = surf similarity parameter (see Equation 2.5.1);
R_c = crest height above SWL =freeboard (see **Figure 2.6.3**);
α = slope angle;
γ_r = roughness factor (see Section 2.6.2 and **Table 2.6.3**);
γ_{berm} = berm factor (γ_{crest} =1 for $B_c \leq 3D_n$; see Section 2.6.3);
$\gamma_{\beta eta}$ = oblique wave factor (see Section 2.6.4);

γ_{crest} = crest width factor (see Section 2.6.5);
γ_s = safety factor (= 1.1 to 1.2 for deterministic design method).

The maximum value is $q_{wo,max} = 0.2\ (gH_{s,toe}^3)^{0.5}$ for $R_c = 0$ (crest at still water level).
This yields $q_{wo,max} = 7\ m^3/s/m$ for $H_{s,toe} = 5$ m.

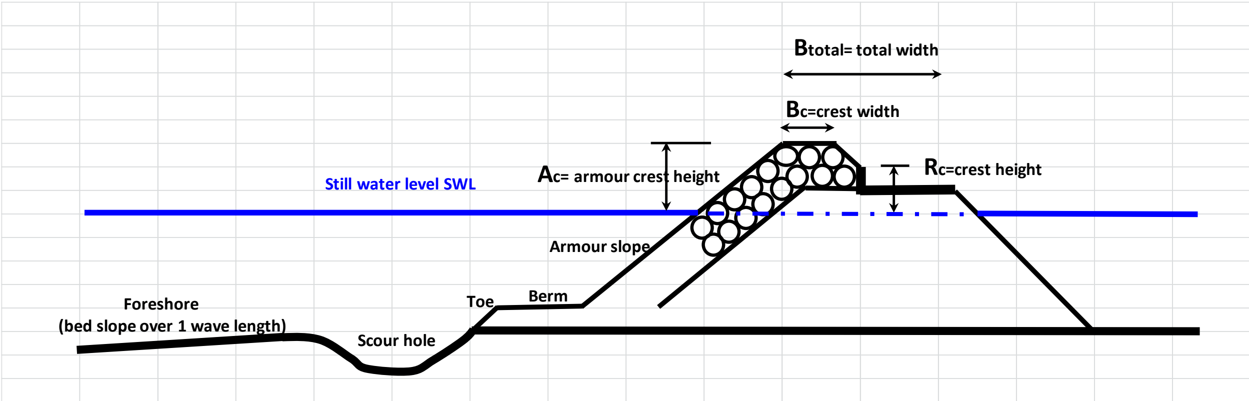


Figure 2.6.3 *Definitions breakwater*

2.6.2 Effect of rough slopes

Seadikes and revetments
The roughness factor of relatively smooth surfaces with roughness elements are given in **Table 2.5.1**.

Breakwaters
Most rubblemound structures have an armour layer consisting of rock or concrete blocks.
Some values of the roughness γ_r-factor are given in **Table 2.6.3**. The roughness of a smooth surface = 1.

Type of roughness	**Reduction factor for wave overtopping γ_r**
Smooth surface (concrete, asphalt, grass)	1
Rocks; straight slope, 1 layer on impermeable core	0.6
Rocks; straight slope, 2 layers on impermeable core	0.55
Rocks; straight slope, 1 layer on permeable core	0.45
Rocks; straight slope, 2 layers on permeable core	0.4
Rocks; berm breakwaters, 2 layers, permeable core (reshaping profile)	0.4
Rocks; berm breakwaters, 2 layers, permeable core (non-reshaping)	0.35
Cubes; straight slope, 1 layer random	0.5
Cubes; straight slope, 2 layers random	0.45
Accropods, X-blocks, Dolos; straight slope of random blocks	0.45
Tetrapods; straight slope of random blocks	0.4

Table 2.6.3 *Roughness factors for wave overtopping at a breakwater slope 1 to 1.5 (EUROTOP 2007)*

2.6.3 Effect of composite slopes and berms

Seadikes and revetments

A berm above the design water level reduces the wave overtopping rate during a storm event, depending on the berm width (see **Figure 2.5.4**): γ_{berm} = 0.6 for very wide berms (berm width = 0.25L_{toe} with L_{toe} = wave length at toe) to γ_{berm} = 1 for very small berms. Wide berms are very effctive.
Berms should be placed at a high level to be effective, just above the water level with a return period of 100 years (2 to 4 m above mean sea level MSL).
The berm effect on the overtopping rate can be expressed by Equation (2.5.6): $\gamma_{berm} = (1H_{s,toe}/B)^{0.3}$.

Breakwaters

The effect of composite slopes on the overtopping rate is minor for breakwaters as the seaward slopes are already relatively steep in the range between1 to 1.5 and 1 to 2.5. Laboratory test results with varying slopes in this slope range donot show a marked slope effect (EUROTOP Manual, 2007).

Berm breakwaters

In the case of a berm breakwater the outer slope is approximately the slope of the line between the toe and the crest. A better estimate is the slope of the line between the point at 1.5$H_{s,toe}$ below the design water level and the runup-point. This latter method requires, however, iterative calculations as the runup point is a priori unknown.
A berm above the design water level reduces the wave runup and overtopping during a storm event, depending on the berm width. Wide berms are very effective.
Berms should be placed at a high level to be effective, just above the design water level with return period of 100 years (2 to 4 m above mean sea level MSL).

According to Sigurdarson and Van der Meer (2012), Equation (2.6.6) is not very accurate for rough armour slopes of berm breakwaters. They have analysed many data of wave overtopping of berm breakwaters and found a clear effect of longer-period wave steepness and the berm width.

They have proposed to replace the berm reduction factor and the roughness factor by a new factor ($\gamma_{berm,new}$), as follows:

$\gamma_{berm,new} = \gamma_r\, \gamma_{berm} = 0.68 - 4.5\, s - 0.05\, B/H_{s,toe}$ for hardly to partly reshaping breakwaters (2.6.7a)
$\gamma_{berm,new} = \gamma_r\, \gamma_{berm} = 0.7 - 9\, s$ for fully reshaping breakwaters (2.6.7b)

with: B= berm width, $s = H_{s,toe}/L_o = (2\pi/g)H_{s,toe}/T_p^2$ and $H_{s,toe}$= design significant wave height at toe of structure based on 100 year return period, $L_o = (g/2\pi)T_p^2$= deep water wave length.

Using equation (2.6.7), Equation (2.6.6) for rough slopes bcomes:

$$q_{wo,max} = 0.2\, \gamma_{crest}\, (gH_{s,toe}^3)^{0.5} \exp\{(-2.3\, \gamma_s\, R_c/(\gamma_{berm,new}\, \gamma_{\beta eta}\, H_{s,toe})\} \quad (2.6.8)$$

The method was used to compute the wave overtopping rate at the Husavik berm breakwater in NW Iceland during a storm event with $H_{s,offshore}$ = 11 m, $H_{s,toe}$ = 5 m, T_p= 13.5 s, resulting in wave overtopping values in the range of 1.5 to 2.5 l/m/s. These values are in good agreement with upscaled overtopping rates from laboratory tests of this breakwater. The observed damage at the Husavik breakwater, which was heavily overtopped, was almost none.

2.6.4 Effect of oblique wave attack

Based on laboratory test results, the effect of oblique waves on wave overtopping can be taken into account by (EUROTOP Manual 2007):

Smooth slopes (dikes/revetments)	$\gamma_{\beta eta}$=1-0.0025|β|	for $0 \leq \beta < 80^o$	(2.6.9a)
	$\gamma_{\beta eta}$= 0.8	for $\beta \geq 80^o$	(2.6.9b)
Rough slopes (breakwaters	γ_β = 1-0.0063|β|	for $0 \leq \beta < 80^o$	(2.6.9c)
	γ_β = 0.5	for $\beta \geq 80^o$	(2.6.9d)

with: β = wave angle to shore normal (in degrees), see **Figure 2.5.5**.

2.6.5 Effect of crest width

If a breakwater has a crest width larger than $B_{crest} = 3D_n$, the wave overtopping rate is reduced, because the overtopping water can more easily drain away through the permeable structure. A wide crest of a seadike or revetment has no reducing effect.
This effect (only for structures with a permeable crest) can be taken into account by using (EUROTOP Manual 2007);

$$\gamma_{crest}= 3 \exp(-1.5B_{crest}/H_{s,toe}) \quad \text{for } B_{crest} > 0.75\ H_{s,toe} \quad (2.6.10a)$$

This yields: γ_c = 1 for for B_{crest}= 0.75$H_{s,toe}$, γ_c= 0.7 for B_{crest}= 1$H_{s,toe}$ and γ_c= 0.15 for B_{crest}= 2$H_{s,toe}$.

A more conservative expression is:

$$\gamma_{crest}= 0.75\ H_{s,toe}/B_{crest} \quad \text{for } B_{crest} > 0.75\ H_{s,toe} \quad (2.6.10b)$$

This yields: γ_c = 1 for for B_{crest}= 0.75$H_{s,toe}$, γ_c= 0.75 for B_{crest}= 1$H_{s,toe}$ and γ_c= 0.375 for B_{crest}= 2$H_{s,toe}$.

2.6.6 Example case

Seadike with smooth slope of 1 to 4: tan(α)= 0.25, ρ_w= 1025 kg/m^3
Water depth at toe = 3 m
Wave heights and wave periods are: $H_{s,toe}$ = 2, 3, 4 m and T_p = 8, 10, 12 s.
Maximum water level is 3 m above mean sea level (MSL).
Safety factor wave overtopping γ_s = 1.5.
The spreadsheet-model **ARMOUR.xls** has been used to compute the wave overtopping rate.

Figure 2.6.4 shows the wave overtopping rate (litres/m/s) as function of the crest height above the maximum water level for three wave conditions and a rougness factor γ_r = 1. The overtopping rate is strongly dependent on the wave height (factor 10 for a wave height increase of 1 m).
The wave overtopping rate for a wave height of 3 m has also been computed for two roughness factors γ_r= 0.8 and 0.6 (see **Table 2.5.1**). A very rough slope surface yields a large reduction of the wave overtopping rate (factor 10 to 100).

To reduce the overtopping rate to 1 litres/m/s for a wave height of 3 m at the toe of the dike, the crest height should be about 11 m above the maximum water level and thus 14 m above MSL. Using roughness elements (γ_r = 0.8) on the dike surface, the crest height can be reduced by about 2 m. Other coefficients (γ_{berm}, γ_{beta}) have a similar strong effect.

Given the strong effect of the roughness factor, the proper roughness value of a seadike with roughness elements should be determined by means of scale model tests.

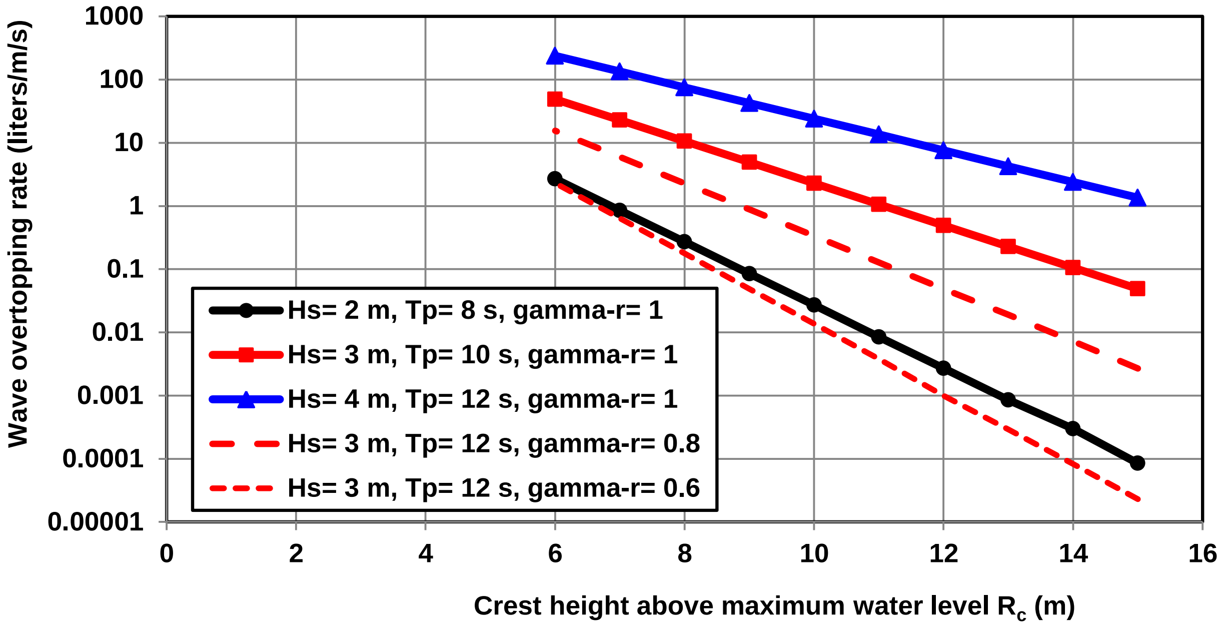


Figure 2.6.4 *Wave overtopping rates at crest of seadike*

2.7 Wave transmission

When waves attack a structure, the wave energy will be either reflected from, dissipated on (through breaking and friction) or transmitted through or over (wave overtopping) the structure.
The amount of transmitted wave energy depends on:

- the incident wave characteristics;
- the geometry of the structure (slope, crest height and width);
- the type of structure (rubble mound or smooth-faced; permeable or impermeable).

Ideally, harbour breakwaters should dissipate most of the incoming wave energy. Transmission of wave energy should be minimum to prevent wave motion and resonance within the harbour basin.
Large overtopping rates (if more than 10% of the waves are overtopping) will generate transmitted waves behind the structure which may be higher than 10% of the incident wave height.
The most accurate information of wave transmission can only be obtained from laboratory tests, particularly for complex geometries.

Generally, the transmission coefficient K_T is expressed as: $K_T = H_{s,T}/H_{s,toe}$.
Figure 2.7.1 shows the K_T-coefficient as function of the relative crest height ($R_c/H_{s,toe}$) for rubble mound structures (Van der Meer, 1998) with R_c = crest height above the still water level (SWL) and $H_{s,toe}$ = incident significant wave height at toe of structure.
$R_c/H_{s,toe} = 0$ means crest height at still water level.
$R_c/H_{s,toe} = 1$ means crest height at distance H_i above the still water level.
$R_c/H_{s,toe} = -1$ means crest height at distance H_i below the still water level.

The available data can be represented by (Van der Meer 1998):

$K_T = 0.1\gamma_s$ for $R_c/H_{s,toe} \geq 1.2$
$K_T = 0.8\gamma_s$ for $R_c/H_{s,toe} \leq -1.2$ (2.7.1)
$K_T = -0.3\gamma_s (R_c/H_{s,toe}) + 0.45$ for $-1.2 < R_c/H_{s,toe} < 1.2$

with: γ_s = safety factor (=1.2 to use the upper enveloppe of the data).

Equation (2.7.1) including the experimental range is shown in **Figure 2.7.1** and can be used for the preliminary design of a structure.

Figure 2.7.1 also shows the wave transmission coefficient (K_T) for a conventional breakwater armoured with tetrapods and accropodes and a relatively low-crested concrete superstructure. The results show that even for relatively high crest levels ($R_c/H_{s,toe} > 2$) always some wave transmission (5% to 10%) can be expected due to waves penetrating (partly) through the upper part of the permeable structure consisting of rocks and stones. Test results for smooth slopes of 1 to 4 with wave steepness values of 0.01 (long waves) and 0.05 (wind waves) are also shown (EUROTOP 2007). These latter two curves fall in the experimental range of the K_T-values for rough rubble mound surfaces. These curves show that longer waves produce more wave runup, wave overtopping and thus wave transmission.

The effects of other parameters such as the crest width, slope angle and type of structure (rough rock surface, smooth surface) have been studied by others (De Jong 1996 and D'Angremond et al. 1996). More accurate results can only be obtained by performing laboratory tests for the specific design under consideration.

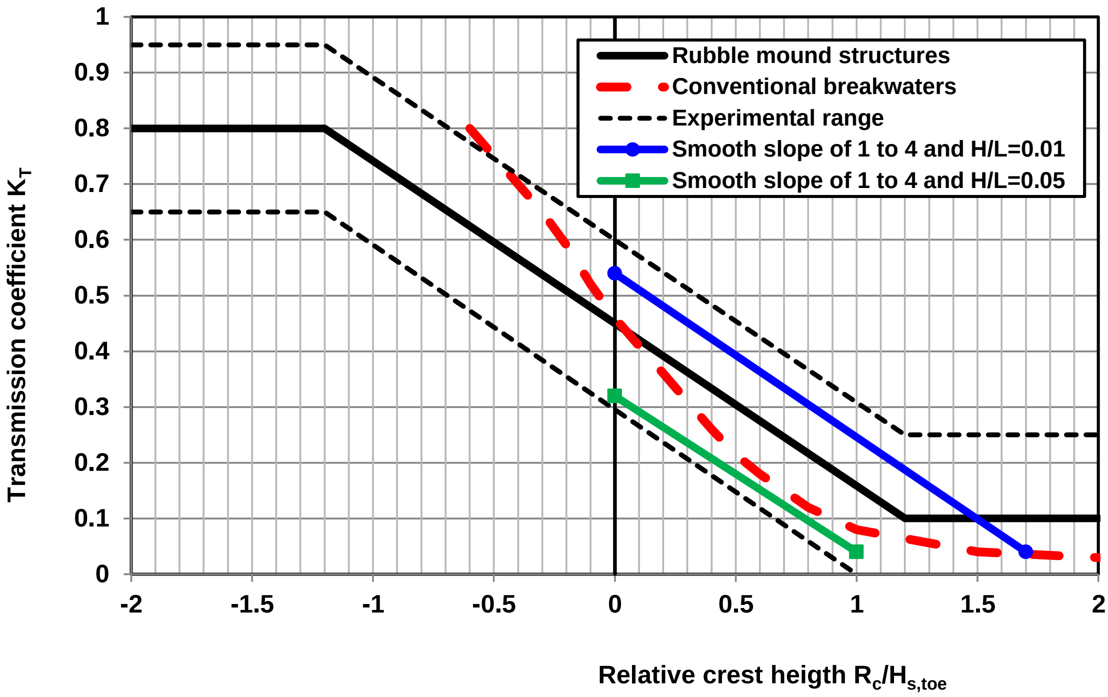


Figure 2.7.1 *Wave transmission coefficient K_T as function of relative crest height $R_c/H_{s,toe}$ for rubble mound structures and conventional breakwaters (based on Van der Meer 1998)*

Figure 2.7.2 shows the wave transmission coefficient for a wide-crested, submerged breakwater (reef-type breakwater) based on the results of Hirose et al. (2002).
B= width of crest, $L_{s,toe}$= wave length at toe of structure, R_c= crest height below still water level.

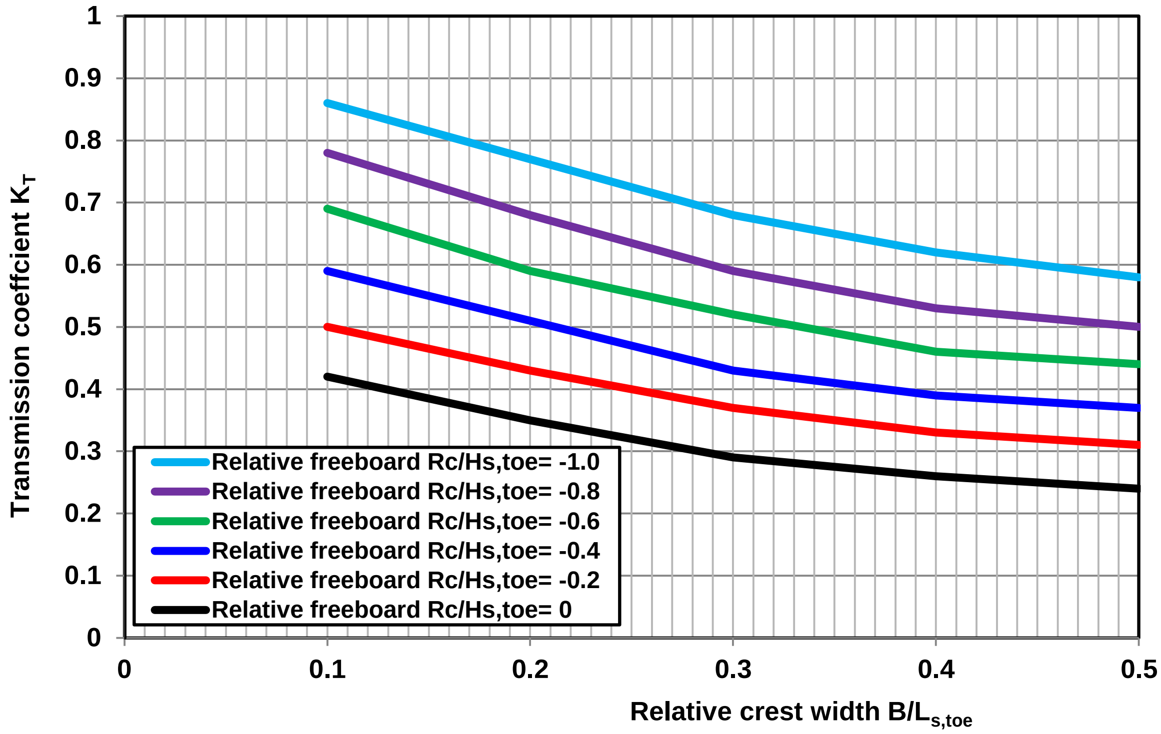


Figure 2.7.2 *Wave transmission coefficient K_T as function of relative crest width B/L height for rubble mound structures and conventional breakwaters*

3 STABILITY EQUATIONS FOR ROCK AND CONCRETE ARMOUR UNITS

3.1 Introduction

The stability of rocks and stones on a mild sloping bottom in a current with and without waves can be described by the method of Shields (Shields' curve) for granular material. This method is also known as the critical shear stress method. A drawback of this method is that the value of the friction coefficient is required which introduces additional uncertainty.
Therefore, the stability of stones and rocks in coastal seas is most often described by a stability number based on the wave height only. This method is known as the critical wave height method.
Both methods are described hereafter.

3.2 Critical shear-stress method

3.2.1 Shields mobility number

The problem of initiation of motion of granular materials due to a flow of water (without waves) has been studied by Shields (1936). Based on theoretical work of the forces acting at a spherical particle (see Figure 3.2.1) and experimental work with granular materials in flumes, he proposed the classical Shields' curve for granular materials in a current. The Shields' curve expresses the critical dimensionless shear stress also known as the Shields mobility number (θ_{cr}) as function of a dimensionless Reynolds' number for the particle, as follows:

$$\theta_{cr} = \tau_{b,cr}/[(\rho_s - \rho_w)\, g\, D_{50}] = \text{Function}\,(u_{*,cr}\, D_{50}/\nu) \qquad (3.2.1.)$$

with: $\tau_{b,cr} = \rho_w\,(u_{*,cr})^2$ = critical bed-shear stress at initiation of motion, $u_{*,cr}$= critical bed-shear velocity, ρ_s = density of granular material (2700 kg/m^3), ρ_w = density of water (fresh or saline water), ν= kinematic viscosity coefficient of water (=0.000001 m^2/s for water of 20 degrees Celsius), D_{50} = representative diameter of granular material based on sieve curve (Shields used rounded granular materials in the range of 0.2 to 10 mm; stones and rocks are represented by $D_{n,50}$, see Equation 3.3.1).

The Shields' curve is shown in Figure 3.2.1 and represents the transition from a state of stability to instability of granular material.
Granular material is stable if:

$$\theta \le \theta_{cr} \quad \text{or} \quad \tau_b/[(\rho_s - \rho_w)\, g\, D_{50}] \le \theta_{cr} \qquad (3.2.2)$$

Based on detailed analysis of the data of existing research papers on the inititiation of fine cohesionless sediment particles in the range of 10 to 400 μm in the laminar and the tubulent flow range and the work of Soulsby (1997) and Van Rijn (1993), the following equation for the critical bed-shear stress related to initiation of motion is proposed:

$$\theta_{cr,shields} = 0.3/(1+D_*) + 0.055\,[1-\exp(-0.02D_*)] \qquad \text{for } D_* > 0.1 \qquad (3.2.3)$$

The θ_{cr}-value according to Shields-curve (solid black curve of **Figure 3.2.1**) is approximately constant at $\theta_{cr,shields} \cong 0.05$ (independent of the Reynolds' number; right part of Shields' curve) for coarse grains > 10 mm or $u_* D_{50}/\nu > 100$.
The precise definition of initiation of motion used by Shields is not very clear. Experimental research at Deltares (1972) based on visual observations shows that the Shields'curve actually represents a state with frequent movement of particles at many locations (p_m=percentage of moving particles is about 100%), see **Figure 3.2.1**. Hence, the Shields' curve cannot really be used to determine the critical stability of a particle.

The critical mobility parameter can be defined as: θ_{cr}= r $\theta_{cr,shields}$ with $\theta_{cr,shields}$ =0.05; r= reduction factor (range of 0.4-1).

The reduction parameter (r) from the relationship θ_{cr}= r $\theta_{cr,shields}$ can be seen as a correction parameter acting on the Shields curve to define a particular stage of movement (or damage). Based on visual observations during initiation of motion experiments of Deltares (Deltares 1972; see Van Rijn 1993), the r-parameter is herein defined, as follows:

r= 0.4 (occasional particle movement at some locations; ≅0.1% of surface is moving);
damage level 1 with minimum movement;

r= 0.6 (frequent particle movement at some locations; ≅1% of surface is moving);
damage level 2 with very limited movement;

r= 0.8 (frequent particle movement at many locations; ≅10% of surface is moving);
damage level 3 with some movement;

r= 1.0 (frequent particle movement at nearly all locations; ≅50% of surface is moving; Shields' curve);
damage level 5 with failure.

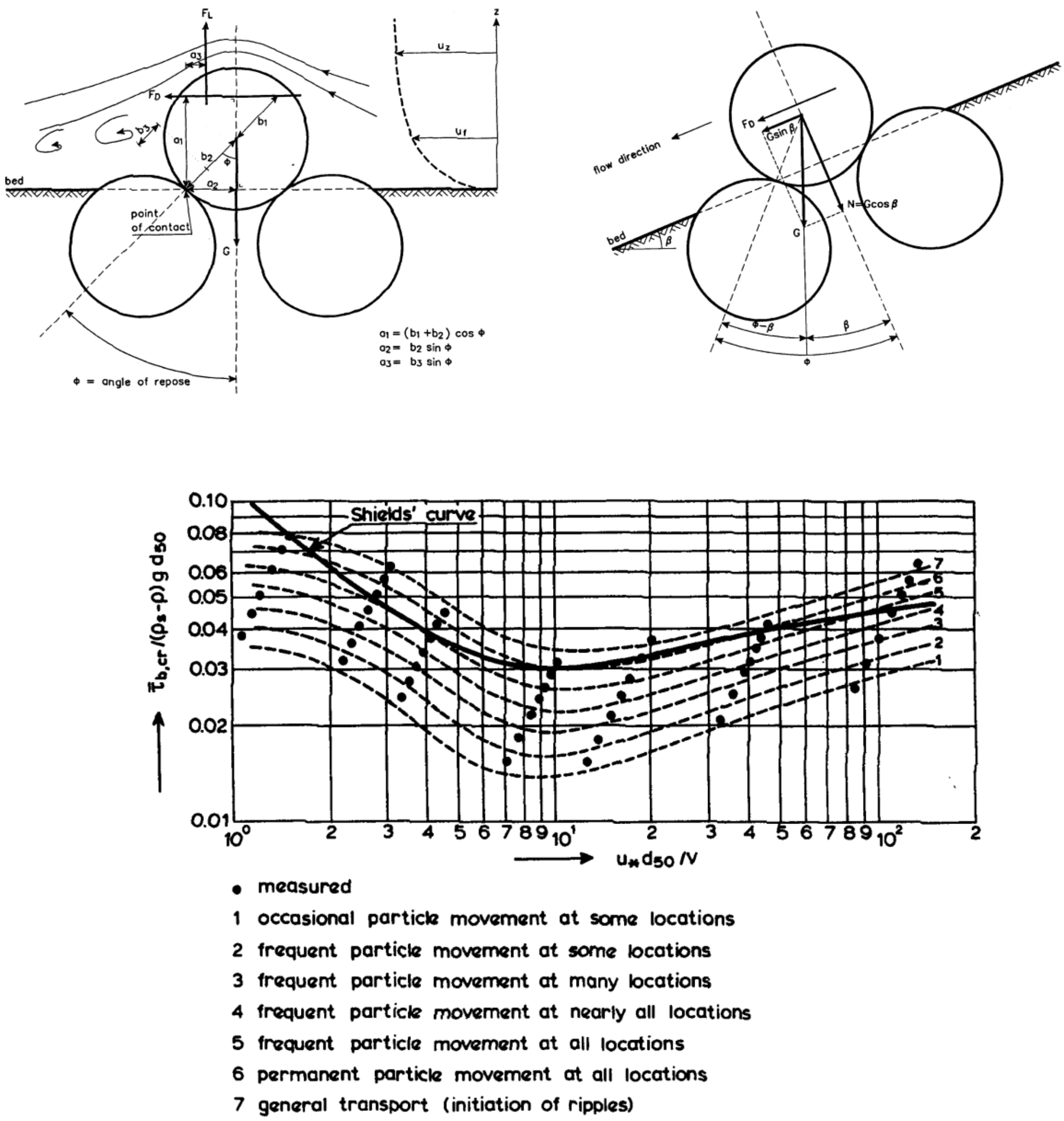


Figure 3.2.1 *Initiation of motion and Shields curve*

Van Rijn (1993) has shown that the Shields curve is also valid for conditions with currents plus waves, provided that the bed-shear stress due to currents and waves ($\tau_{b,cw}$) is computed as:

$$\tau_{b,cw} = \tau_{b,c} + \tau_{b,w} \qquad (3.2.4)$$

with:
$\tau_{b,c}$ = $1/8\ \rho_w\ f_c\ \bar{u}^2$ = bed-shear stress due to current (N/m^2);
$\tau_{b,w}$ = $1/4\ \rho_w\ f_w\ \hat{U}^2$ = bed-shear stress due to current (N/m^2);
$\bar{u}$ = depth-mean current velocity (m/s);
$\hat{U}$ = near-bed peak orbital velocity (m/s) = $\pi\ H_s\ (T_p)^{-1}\ [\sinh(2\pi h/L_s)]^{-1}$ (linear wave theory);
f_c = $0.24[\log(12h/k_s)]^{-2}$; $f_{c,approx} \cong 0.11(h/k_s)^{-0.3}$ = current-related friction factor (-);
f_w = $\exp\{-6 + 5.2(\hat{A}/k_s)^{-0.19}\}$; $f_{w,approx} = 0.1(\hat{A}/k_s)^{-0.3}$ = wave-related friction factor (-);
h = water depth (m);
H_s = significant wave height (m);
L_s = significant wave length (m);
T_p = wave period of peak of wave spectrum (s);
$\hat{A}$ = $(T_p/2\pi)\ \hat{U}$ = near-bed peak orbital amplitude;
k_s = effective bed roughness of Nikurade; ($k_s = D_{max}$ with $D_{max} = \alpha\ D_{n,50}$ and α= 1.3 to 2 for narrow graded stones/rocks).

The relative roughness parameters ($h/(\alpha D_{50})$; $\hat{A}/(\alpha D_{50})$) are in the range of 10 to 300 for coarse materials. **Table 3.2.1** shows the current-related and wave-related friction factors for relative roughness values in the range of 10 to 300. Approximate functions are defined enabling explicit computations. The current-related approximation function is quite accurate, but the wave-related approximation function is less accurate. This latter function can be much better represented by the function $f_{w,approx} = 0.2[\hat{A}/(\alpha D_{50})]^{-0.5}$. However, if current-related and wave-related approximation functions with different powers are used, an explicit equation for a stable value of $D_{n,50}$ can not be determined for combined curent+wave conditions.
Therefore, fairly conservative approximation functions with an equal power of -0.3 have been used to obtain an explicit equation for the stable value of $D_{n,50}$ in combined current+wave conditions.

Relative roughness $h/(\alpha D_{50})$; $\hat{A}/(\alpha D_{50})$ (-)	Current-related friction			Wave-related friction	
	Chézy coefficient $C=18\log(12h/(\alpha D_{50}))$ ($m^{0.5}/s$)	friction factor $f_c=8g/C^2$ (-)	approximate friction factor $f_{c,approx}=0.11[h/(\alpha D_{50})]^{-0.3}$ (-)	friction factor f_w (-)	approximate friction factor $f_{w,approx}=0.1[\hat{A}/(\alpha D_{50})]^{-0.3}$ (-)
10	37.4	0.056	0.055	0.071	0.050
15	40.6	0.048	0.049	0.055	0.044
20	42.8	0.043	0.045	0.047	0.041
50	50.0	0.031	0.034	0.029	0.031
100	55.4	0.026	0.028	0.022	0.025
150	58.6	0.023	0.024	0.018	0.022
200	60.8	0.021	0.022	0.017	0.020
300	64.1	0.02	0.020	0.014	0.018

Table 3.2.1 *Current-related and wave-related friction factors*

3.2.2 Slope effects

In the case of a mild sloping bed (Van Rijn 1993) the θ_{cr}-value can be computed as:

$$\theta_{cr} = K_{\alpha 1}\, K_{\alpha 2}\, r\, \theta_{cr,shields} \qquad (3.2.5)$$

with:
K_{slope1} = sin(ø-α_1)/sin(ø) = slope factor for upsloping velocity; sin(ø+α_1)/sin(ø) for upsloping velocity;
K_{slope2} = [cos(α_2)][1 - {tan(α_2)}^{2}/{tan(ø)}^{2}]$^{0.5}$ = slope factor for longitudinal velocity;
$\theta_{cr,shields}$ = critical Shields' number at horizontal bottom;
r = reduction parameter;
α_1 = angle of slope normal to flow or wave direction (slope smaller than 1 to 5);
α_2 = angle of slope parallel to flow or waves (slope smaller than 1 to 3);
ø = angle of repose (30 to 40 degrees).

3.2.3 Stability and damage equations for rocks in currents

<u>**Stability equation**</u>
Using the available formulae (3.2.3, 3.2.4 and 3.2.5) and $\tau_{b,c} = 1/8\, \rho_w\, f_c\, \bar{u}^2$; $f_{c,approx} \cong 0.11[h/(\alpha D_{n,50})]^{-0.3}$ for large rocks; $\tau_{b,w} = 0$, $\theta_{cr} = r\theta_{cr,shields}$, $\theta_{cr,shields} = 0.05$ and $\tau_{b,c} = \theta_{cr}\,(\rho_s-\rho_w)\, g\, D_{n,50}$; the critical diameter can be expressed as:

$$D_{n,50} = \gamma_s\, [\Delta\, g\, K_{\alpha 1}\, K_{\alpha 2}\, r\, \theta_{cr,shields}]^{-1.4}\, [0.013\,(h/\alpha)^{-0.3}\,(\gamma_{str}\,\bar{u}\,)^2]^{1.4} \qquad (3.2.6)$$

with:
$\theta_{cr,shields}$ =critical Shields mobility parameter based on Equation 3.2.3; ($\theta_{cr,shields} \cong 0.05$ for coarse materials),
r = reduction factor=0.5-1 (larger r-value yields smaller diameter);
α = bed roughness coefficient ($k_s=\alpha D_{n,50}$ with α=1 to 2); larger α yields larger diameter;
γ_s = safety factor;
γ_{str}= velocity+turbulence enhancement factor due to the presence of structures such as bridge piles or windmill piles (γ_{str}= 1 to 1.5; γ_{str}= 1 if no structures are present).
Equation (3.2.6), which is implemented in the **ARMOUR.xls** model, is an explicit equation based on a relatively simple friction factor. Using the more accurate Chézy-equation (C= $5.75g^{0.5}\log(12/(\alpha D_{n,50}))$), the $D_{n,50}$-value can only be determined by an interative method.

<u>**Flume and field of critical velocity and bed-shear stress of rocks and boulders**</u>
Most of the research on initiation of motion of particles is related to relatively small size particles with diameters < 10 mm (see Shields 1936, Graf 1971, Yalin 1977, Van Rijn 1993, Soulsby 1997) and will not be discussed here. In this paper, the attention is focused on critical conditions of large size pebbles, cobbles, boulders and rocks in the laboratory and at various field sites.
Atal and Lavé (2009) have studied the movement of pebbles and rocks with diameters in the range of 5 to 70 mm in a circular laboratory flume. Particle trajectories and collisions were recorded and analyzed using a high-speed camera. Pebbles moved by saltation and occasionally by rolling. Mean saltation velocities during the saltations (hops) were measured directly from particle tracking on the movies. They have found that the particle velocity can be expressed as $u_{particle} \cong 0.7\, u_{fluid}$. The critical velocities are roughly: $u_{critical}$= 1 m/s for D_{50}= 15 mm to $u_{critical}$= 1.75 m/s for D_{50}= 70 mm, see **Table 3.2.2**.
Helley (1969) has studied the threshold velocities of large size rocks (0.15 to 0.45 m) in the Blue Creek mountain river in the USA. Thirty-six natural rocks of various sizes and shapes were painted fluorescent red, tagged by a float and placed in a study reach in the Blue Creek (summer and fall of 1967) where near-bed velocities could be measured close to the tagged rocks. The water depth was in the range of 1 to 1.2 m. Initiation of motion was defined as the sudden movement of the floats. Helley also presents the field data of similar measurements by Fahnestock (1963), see **Table 3.2.2**.

Inbar and Schick (1979) have studied the critical movement of boulders and rocks during flash floods in the upper Jordan River and the Meshushim River in Israel. An extreme rainstorm (once in 100 years) in January 1969 generated a flood in the Jordan River. Boulders up to 1.3 m were moved, and the channel was completely reshaped. During the same event, a 300 m^3/s peak flow occurred in the neighbouring Nahal Meshushim River. Here too, numerous boulders of 1 m were transported. The critical flow velocities in depths of 1 to 1.5 m can be summarized, as follows: $u_{critical}$= 2-2.5 m/s, for D_{50}= 0.25-0.5 m and $u_{critical}$= 2.5-3 m/s for D_{50}= 0.5-1 m, see **Table 3.2.2**.

Turowski et al. (2009) have studied the movement of boulders and rocks in the Erlenbach mountain stream; a small stream with step-pool morphology in the canton of Schwyz, Switzerland. Three exceptional events have occurred and partly or completely rearranged the existing step-pool morphology. In the aftermath of the events, sediment transport rates at a given discharge and total sediment yield remained at higher values for about a year or longer. For the last event, dated on the 20 June 2007, observations of boulder mobility and step destruction were used to interpret channel stability. Boulders with median diameters of up to 1.35 m and estimated weights of more than 2.5 tons have moved during the 2007 event. Boulders of about 0.5 to 0.65 m were found to have been fully mobile in peak conditions with mean velocities of about 3 m/s (discharge of about 15 m^3/s, mean flow width of 4 m and mean flow depth of 1.2 m).

Mueller et al. (2005) have studied the threshold bed-shear stress (θ_{cr}) of bed-load transport of rocks and boulders with D_{50}-values in the range of 0.025 to 0.21 m in various mountain streams (Idaho, USA). The θ_{cr}-value is defined as the dimensionless bed-shear stress at a very low dimensionless reference bed-load transport rate of 0.002. The reference transport rate is assumed to represent flow conditions that are just high enough to begin mobilizing sediment from the bed surface. A typical phenomenon of mountain streams is the wide grain-size distribution including patches of finer, more mobile sediment and large, relatively immobile boulders that are often arranged into cascades or steps. Annual snowmelt and stormflows can mobilize fine sediment while boulders only move in rare, larger floods or debris flows. The large, relatively immobile grains disrupt the flow and increase turbulence. Analysis of their data shows that the critical Shields-parameter varies considerably in the range of 0.01 to 0.12, which is a much wider range than the laboratory range of 0.02 to 0.06. Based on this field data set, the movement of cobbles/boulders/rocks is possible at very low mobility values of 0.01 to 0.02. The mean value of all values is about 0.047. It is found that the ratio D_{50}/D_{90} has a clear effect on the θ_{cr}-value. This ratio expresses the grading of the bed material; a small value means a wide grading resulting in hiding-exposure effects. Smaller rocks/fragments are more difficult to mobilize, as they are hiding between the larger rocks. Uniform cobbles have a smaller θ_{cr}-value. Another parameter of importance is the ratio h/D_{50} with h= water depth. Rocks are more difficult to move in the case of small submergence (h/D_{50} is small), because the flow resistance is relatively large and the near-bed velocities are relatively small. Flow resistance typically increases as relative submergence decreases and streams become steeper, smaller, and coarser. Large, relatively immobile grains cause local accelerations and decelerations in the flow, hydraulic jumps, large vertical velocities, localized intense turbulence, and areas of eddying water with weak net flow.

Summary of results

All data are summarized in **Table 3.2.2** and in **Figure 3.2.2**. The uncertainty range of particle size and critical velocity is about 10% for the flume data and 15% to 20% for the field data. It is noted that the field data with large rock sizes up to 0.7 m refer to very shallow mountain rivers with depths in the range of 0.5 to 1.2 m (h/D_{50}<10). This type of flow is generally known as the "wild" water regime with exceptionally high turbulence levels exceeding those of normal open channel flow (h/D_{50}>100). Furthermore, the field data tests of relatively large rocks concern the initiation of motion of very isolated rocks which are placed on top of the river bed and are thus extremely exposed to the local turbulent velocities. This test arrangement is very different from a bed protection layer consisting of rocks of approximately the same size with sheltering effects due to the presence of neighbouring rocks.

Equation (3.2.6) has three input parameters: α-coefficient related to bed-roughness effect (range 1 to 2), r-coefficient related to the most appropriate critical Shields mobility number range (range 0.3 to 0.6) and the

γ_{str}-coefficient related to the velocity and turbulence enhancement due to the bed-structure arrangement (range 1 to 1.5). The α-coefficient is set to α=2, which means that the effective bed roughness is equal to $k_s=2D_{50}\cong 1D_{90}$. The other two coefficients have been calibrated using the data of **Figure 3.2.2**. Computed results are shown for r=0.3, 0.4 and 0.5 and γ_{str}=1.2. This latter coefficient represents the effect that the rocks used in de field tests are isolated rocks fully exposed to the flow with extreme turbulence levels. Fairly good agreement between measured and computed results can be observed for r=0.4 and r=0.5 in combination with γ_{str}=1.2.

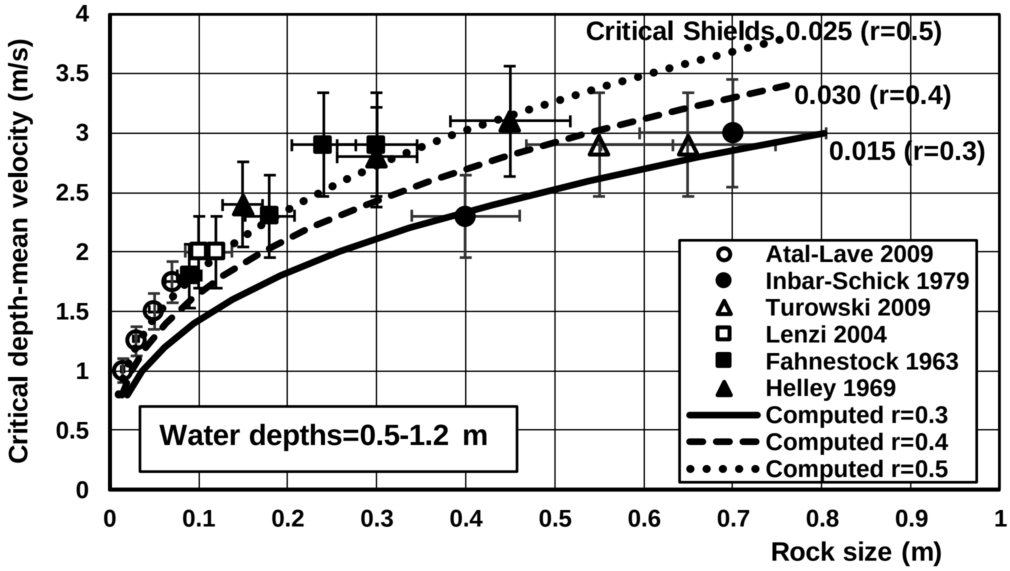


Figure 3.2.2 *Critical depth-mean velocity as function of rock size in currents (γ_{str}=1.2)*

Data set	Water depth (m)	Particle/rock size (m)	Critical depth-mean velocity (m/s)
Flume: Atal and Lavé 2009	<0.5	0.015; 0.03; 0.05; 0.07	1.0; 1.25; 1.5; 1.75
Field: Inbar and Schick 1979	1-1.2	0.4; 0.7	2.3; 3.0
Field: Turowski et al. 2009	0.5-1	0.6	3.0
Field: Lenzi 2004, Lenzi et al. 2006, Mao and Lenzi 2007, Rainato et al. 2017	<0.5	0.11	2.0
Field: Fahnestock 1963	0.5-1	0.09; 0.18; 0.24; 0.3	1.8; 2.3; 2.9; 2.9
Field: Helley 1969	1-1.2	0.15; 0.3; 0.45	2.4; 2.8; 3.1

Table 3.2.2 *Summary of critical conditions for large size particles/rocks in current conditions*

<u>**Damage equation**</u>

A damage estimate can be obtained if the bed load transport at low mobility parameters (θ) in the range of 0.01-0.05 is known.
An important contribution to the study of the stability of granular material has been made by Paintal (1971), who has measured the dimensionless (bed load) transport of granular material at conditions with θ-values in in the range of 0.01 to 0.05, see **Figures 3.2.3, 3.2.4** and **Table 3.2.3**.

Using the approach of Paintal (1971) or Cheng (2002), the bed load tranport of granular material at very small θ-values can be computed by:

Paintal (1971): $\Phi_b = 6.6\ 10^{18}\ \theta^{16}$ (3.2.7a)

$q_b = 6.6\ 10^{18}\ \theta^{16}\ \rho_s\ (\Delta\ g)^{0.5}\ (D_{50})^{1.5}$

or

Cheng (2002): $\Phi_b = 13\ \theta^{1.5}\exp(-0.05/\theta^{1.5})$ (3.2.7b)

$q_b = 13\theta^{1.5}\exp(-0.05/\theta^{1.5})\ \rho_s\ (\Delta\ g)^{0.5}\ (D_{50})^{1.5}$

with:

Φ_b = $(\rho_s)^{-1}\ (\Delta\ g)^{-0.5}\ (D_{50})^{-1.5}\ q_b$;

$q_{b,mass}$ = bed load transport by mass (kg/m/s);

$q_{b,vol}$ $=1/[\rho_s(1-\varepsilon)]q_{b,mass}$= volumetric bed load transport (m²/s);

θ = $\tau_b/[(\rho_s - \rho_w)\ g\ D_{50}]$ = dimensionless bed-shear stress (Shields number);

Δ = $(\rho_s-\rho_w)/\rho_w$ = relative density;

ε = porosity factor (≅0.35-0.45);

τ_b = bed-shear stress due to current (N/m²).

θ-values	**Dimensionless bed load transport Φ measured by Paintal (1971)**
0.02	$4.3\ 10^{-9}$
0.025	$1.5\ 10^{-7}$
0.03	$3.\ \ 10^{-6}$
0.04	$3.\ \ 10^{-4}$

Table 3.2.3 *Bed load transport values measured by Paintal (1971)*

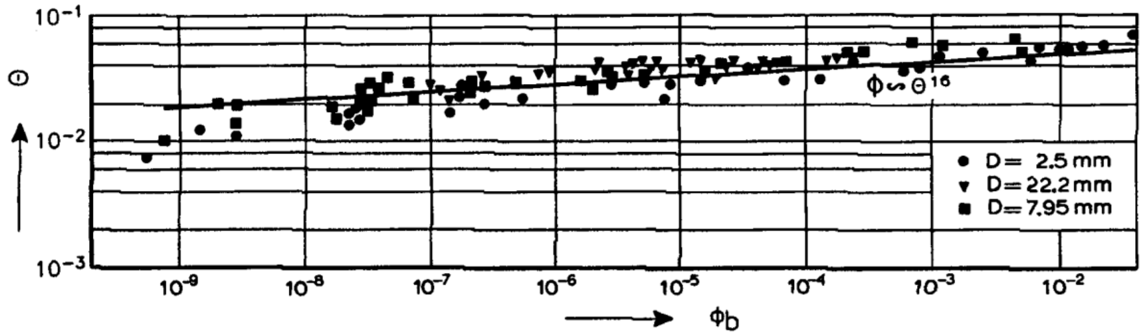


Figure 3.2.3 *Dimensionless bed load transport according to Paintal (1971)*

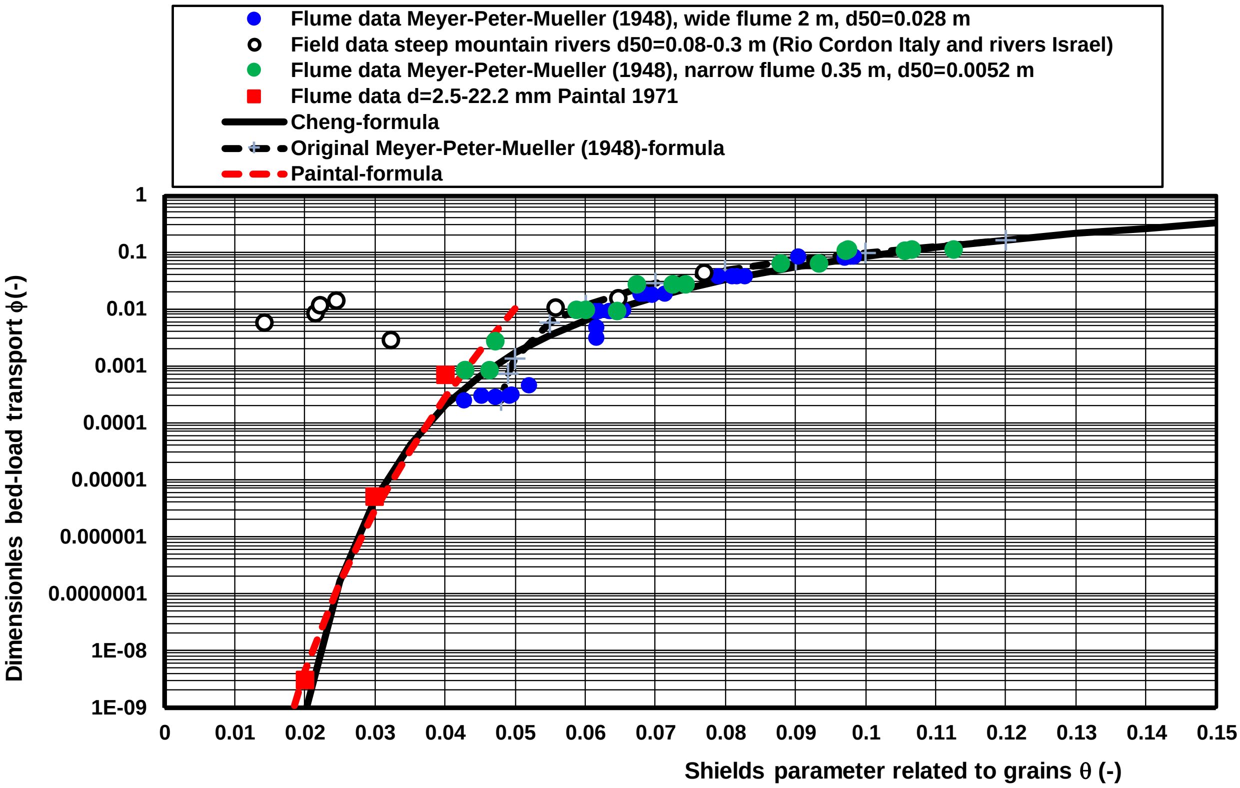


Figure 3.2.4 *Dimensionless bed load transport at low values of the Shields parameter*

The bed load transport (q_b) is given in kg/m/s, which can be converted into the number of moving rocks per unit width (m) and time (s) by using the mass of one rock $M_{rock}= (1/6)\ \pi\ \rho_s\ D_{50}^3$, resulting in:

Paintal (1971): $N_{moving\ rocks} = \gamma_s\ [6.6\ 10^{18}\ \theta^{16}\ \ \rho_s\ (\Delta g)^{0.5}\ (D_{50})^{1.5}\ ]/[(1/6)\ \pi\ \rho_s\ D_{50}^3]$

$$N_{moving\ rocks} = \gamma_s\ [13\ 10^{18}\ \theta^{16}\ \ (\Delta g)^{0.5}\ (D_{50})^{-1.5}\ ] \qquad (3.2.8a)$$

Cheng (2002): $N_{moving\ rocks} = \gamma_s\ [13\ \ \theta^{1.5}\ \exp(-0.05/\theta^{1.5})\ \rho_s\ (\Delta g)^{0.5}\ (D_{50})^{1.5}]/[(1/6)\ \pi\ \rho_s\ D_{50}^3]$

$$N_{moving\ rocks} = \gamma_s\ 25\ (r\theta_{cr,shields})^{1.5}\ \exp\{-0.05/(r\theta_{cr,shields})^{1.5}\}\ (\Delta g)^{0.5}\ (D_{50})^{-1.5} \qquad (3.2.8b)$$

with:
$N_{moving\ rocks}$ = number of moving rocks per m width and per second (per day by multiplication with 86400 s);
θ = mobility parameter; γ_s = safety factor.

Equation (3.2.8b) is implemented in the **ARMOUR.xls** model.
Equation (3.2.8) can be related to the damage parameter S_d (see **Section 3.3.1**), as follows:
$S_d = A_e/(D_{n,50})^2 = \Delta t\ [(1/(1-\varepsilon)]\ N_{movingrocks}\ V_{1rock}\ /(D_{n,50})^2 = \Delta t\ N_{movingrocks}\ (\pi/6)/(1-\varepsilon)(D_{n,50})^3/(D_{n,50})^2$
$\cong \Delta t\ N_{movingrocks}\ D_{n,50}$
with: A_e= eroded area per unit width in a given time period Δt, Δt= time period considered (usually 5000 to 10000 waves or about 1 day of storm), ε= porosity factor ($\cong$ 0.45).

The number of rocks moving out of the bed protection area in a given time period can be seen as damage requiring maintenance. The damage percentage in a given time period can be computed as the ratio of the number of rocks moving away and the total number of rocks available in the bed protection area.

The loss of rocks from a bed protection area with length L and thickness $\delta_{bp}=\alpha_{bp}D_{n,50}$ can be determined, as follows:
Volume of bed protection per unit width: $V_{bp} = L_{bp}\ \delta_{bp}$
Number of rocks in bed protection area: $N_{rocks,bp} = (1-\varepsilon)L_{bp}\ \alpha_{bp}D_{n,50}/\{(\pi/6)D_{n,50}^{\ 3})\} = (6\alpha_{bp}/\pi)\ (1-\varepsilon)\ L_{bp}\ (D_{n,50})^{-2}$
Number of rocks moving out of bed protection area during the lifetime: $N_{rock,out}= N_{moving\ rocks}\ \ T_{extreme\ event}\ T_{life}$.

The Loss coefficient is:

$$P_{Loss}= [N_{moving\ rocks}T_{extreme\ event}T_{life}]/N_{rocks,bp}=[N_{moving\ rocks}\ T_{extreme\ event}T_{life}]/\ [2\alpha_{bp}(1-\varepsilon)\ L_{bp}\ (D_{n,50})^{-2}]$$

with:
L_{bp}= length of bed protection area (normal to flow or waves);
$\delta_{bp}= \alpha_{bp}D_{n,50} =$ thickness of bed protection ($\alpha \cong 2$ to 3);
ε = porosity of bed protection layer ($\cong$0.45);
$T_{extreme\ event}$= duration of extreme events per year (in days per year); storm event or river flood event;
T_{life}= lifetime of structure (in years);
$N_{moving\ rocks}$= number of rocks moving out of bed protection area (per unit width and per day); Eq. (3.2.8b).

Figure 3.2.5 shows the number of moving rocks (per m width and per day) based on the Cheng-equation (3.2.8b) for rocks in the range of D= 0.02 to 0.5 m and current velocities in the range of 1 to 7 m/s. The water depth= 5 m. The Chézy-coefficient is computed as $C = 5.75g^{0.5}\,(\log 12h/\alpha D)$, α=2 and $\tau_b= \rho_w\, g\, (\bar{u}/C)^2$ and $\theta = \tau_b/[(\rho_s-\rho_w)\ g\ D]$.

Assuming almost no movement for $N_{rocks} < 0.0001$ (per m and per day), the critical depth-mean velocities are:

D=0.02 m: $\bar{u}_{cr} \cong 1.3$ m/s,
D=0.05 m: $\bar{u}_{cr} \cong 1.8$ m/s,
D=0.1 m: $\bar{u}_{cr} \cong 2.3$ m/s,
D=0.2 m: $\bar{u}_{cr} \cong 3.0$ m/s,
D=0.5 m: $\bar{u}_{cr} \cong 4.2$ m/s.

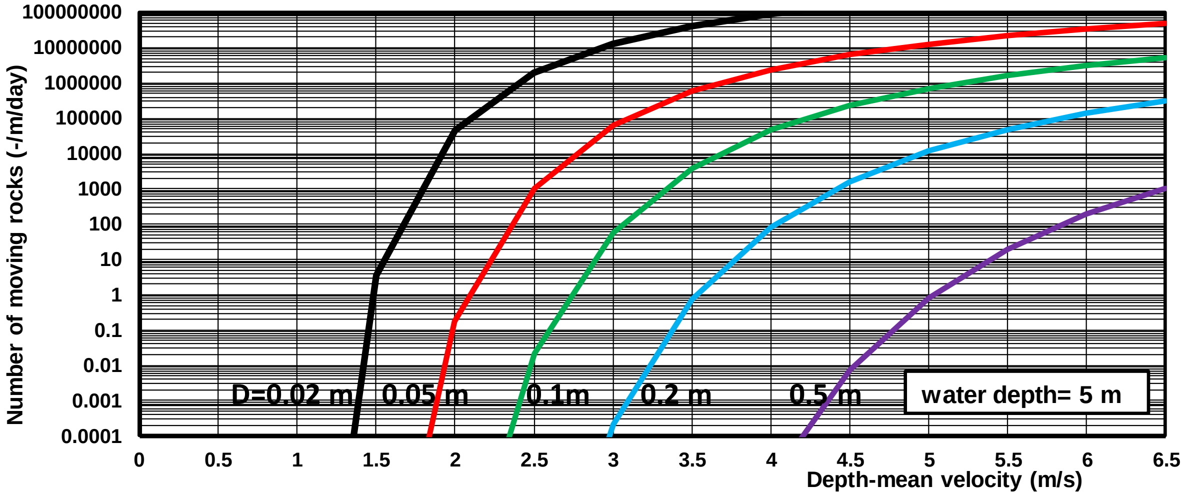


Figure 3.2.5 *Number of moving rocks of horizontal bed protection in current conditions (h= 5 m)*

Practical applications bed protection

Equation (3.2.6) has been used to produce a design graph for horizontal bed protections in water depths of h_o= 3 to 20 m and depth-mean velocities u_{mean}= 1 and 5 m/s. The thickness of the protection layer is set to δ_{bp}= 0.5 m. The effective water depth is: $h_{bp}=h_o-\delta_{bp}$.
Other parameters are: density seawater= 1020 kg/m^3; density sediment= 2650 kg/m^3; α=2, $\theta_{cr,o}$= r $\theta_{cr,shields}$ with r= 0.5 and $\theta_{cr,shields}$ = 0.05; γ_{str} = 1 (no additional velocity-turbulence enhancement due to bed protection layer) and γ_s =safety factor= 1. The computed results are shown in **Figure 3.2.6**.
The results clearly show that the rock diameter decreases for increasing water depth at the same depth-mean velocity, because the bed-shear stress decreases with increasing water depth (less flow resistance).

In the case of a bed protection with rock size=D_{50}=0.1 m, thickness δ_{bp}=0.5 m, $\alpha_{bp}=\delta_{bp}/D_{50}$=5, length L_{bp}= 50 m, porosity ε= 0.45 in a flow with depth of h=5 m and current velocity u=2.5 m/s (during 30 days per year), the number of moving rocks per m and per day is N_{mr}=0.02 (**Figure 3.2.5**) with S_d= 0.002 for Δt=1 day.
The loss coefficient of rocks for an extreme event time of 30 days per year and a lifetime of 50 years for the bed protection layer is:
P_{Loss}= [0.02x30x50]/[2x5x(1-0.45)x50x(0.1)$^{-2}$]= 30/27500= 0.0011 (0.11%) during the lifetime of the stucture.
The thickness of the protection layer can be reduced to 0.3 m resulting in a loss coefficient of about 0.2%.
In both cases (δ_{bp}=0.5 m or 0.3 m), the damage is so low that a static bed protection is obtained.
Using: rock size=D_{50}=0.08 m and δ_{bp}=0.5 m, the number of moving rocks per m per day goes up to N_{mr}=1 (50 times larger, Figure 3.2.5) and S_d=0.08. The loss coefficient goes up by a factor of 50 to about 6%, which may be acceptable (dynamic bed protection for a smaller rock size D_{50}=0.08 m).

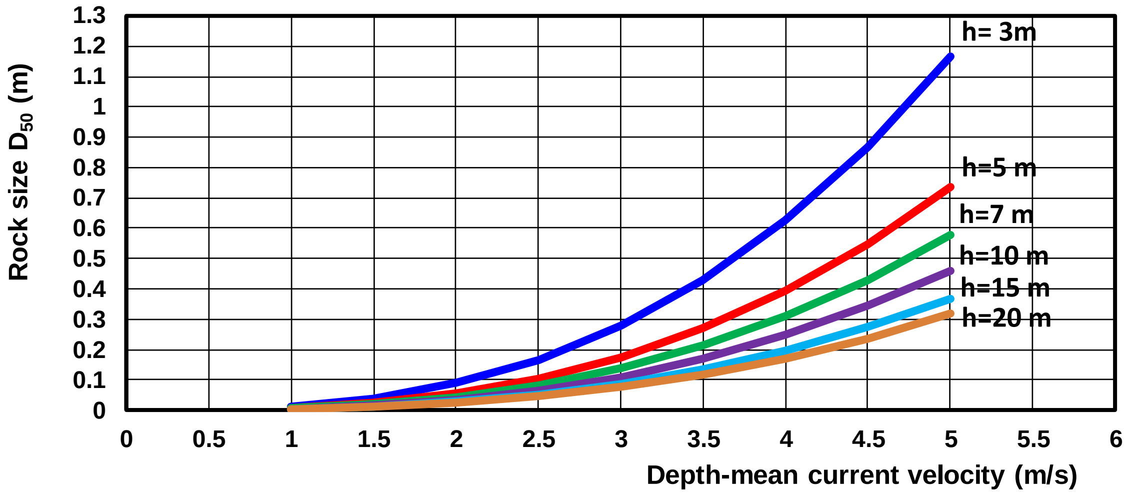


Figure 3.2.6 *Design graph (r= 0.5, γ_{str}= 1) for horizontal bed protection in current conditions*

3.2.4 Stability and damage equations for rocks in waves

Using the available formulae (3.2.3, 3.2.4 and 3.2.5) and $\tau_{b,w}$ = 1/4 ρ_w f_w $\hat{U}^2$, $f_{w,approx} \cong 0.1[\hat{A}/(\alpha D_{n,50})]^{-0.3}$ for large rocks, $\hat{A} = \hat{U}\, T_p/(2\pi)$= orbital amplitude, $\tau_{b,c}$ = 0, θ_{cr}= r $\theta_{cr,shields}$, $\theta_{cr,shields}$ = 0.05 and $\tau_{b,w} = \theta_{cr}(\rho_s-\rho_w)\, g\, D_{n,50}$; the critical diameter can be expressed as:

$$D_{n,50} = \gamma_s \; [\Delta\, g\, K_{\alpha 1}\, K_{\alpha 2}\, r\, \theta_{cr,shields}]^{-1.4}\, [0.045\, (T_p/\alpha)^{-0.3}\, (\gamma_{str}\, \hat{U}\,)^{1.7}]^{1.4} \qquad (3.2.9)$$

with:
$\theta_{cr,shields}$= critical Shields mobility parameter based on Equation 3.2.3; ($\theta_{cr,shields} \cong$ 0.05 for coarse materials);
r = reduction factor=0.5-1 (larger r-value yields smaller diameter);

α = bed roughness coefficient ($k_s=\alpha D_{n,50}$ with α=1 to 2); larger α yields larger diameter;
γ_s = safety factor;
γ_{str}= velocity+turbulence enhancement factor due to the presence of a structure (bed protection around a monopile); γ_{str}=1-1.5, γ_{str} =1 if no structure is present.

Equation (3.2.9) is implemented in the **ARMOUR.xls** model.
Equation (3.2.8b) can be used to get an estimate of the number of moving rocks (damage).
The damage parameter S_d (see **Section 3.3.1**) can be estimated by:
$S_d=A_e/(D_{n,50})^2= \Delta t\ (1-\varepsilon)^{-1} N_{movingrocks}\ V_{1rock}/(D_{n,50})^2= [(\pi/6)/(1-\varepsilon)]\ \Delta t\ N_{movingrocks}\ (D_{n,50})^3/(D_{n,50})^2 \cong \Delta t\ N_{movingrocks}\ D_{n,50}$
with: A_e= eroded area (incl. pores) per unit width in a given time period Δt, Δt= time period considered (usually 5000 to 10000 waves or about 1 day of storm), ε= porosity factor $\cong$ 0.45 and $N_{movingrocks}$ = number of moving rocks per m width and second.

Field data of critical velocity

Crickmore et al. of HR Wallingford (1972) have performed a pebble tracer experiment in the English Channel east of Portsmouth in the period September 1969 to April 1971. The local seafloor consists of natural pebbles/shingle of flint material. The peak tidal velocities are 0.5 m/s during neap tide and 0.8 m/s during spring tide. Radioactive tagged pebbles (D=28 mm, D_{min}=19 mm, D_{max}=38 mm) were placed by divers at three areas (30x60 m^2) with depths of 9, 12 and 18 m (about 1000 tagged pebbles at each area). The site is exposed to waves from south-west to south-east. Wave records were obtained at the Owers light vessel at a depth of about 25 m (about 25 km south-west from the pebble areas). Tracer displacement was measured by towing a detection instrument behind the survey vessel (4 surveys in the period september 1969 to April 1971). The inaccurary of the horizontal positioning system was estimated to be about 5 to 10 m. The thickness of the upper bed layer in which tagged particles were observed, was in the range of 60 to 120 mm. Various storms did occur during the observation period. The maximum significant wave height was about $H_{s,max}$= 5 m in depth of 18 m reducing to $H_{s,max}$=3 m in a depth in 9 m (wave period of about 8 to 10 s). The pebble movement was about 40 m at the site with depth of 9 m, about 15 m at a depth of 15 m and zero at a depth of 18 m.
Table 3.2.4 (Row 1) shows the measured results for the site with depth of 18 m. Based on this, the critical peak orbital velocity of pebbles with D_{50}= 0.028 m is estimated to be $\hat{U}_{cr}\cong$1.2 m/s. Equation (3.2.9) yields $D_{n,50}$=0.028 m for r$\cong$ 0.7, s=2.6, γ_{str}=1 (no structure), T_p=8 s, α=2. This corresponds to a critical mobility number of $\theta_{cr}\cong$0.035, which is much lower than the standard critical Shields value of $\theta_{cr,shields}$= 0.05.

Sediment size		**Type of material**	**Density**	**Water depth**	**Estimated critical velocity for sliding and rolling**
Range (m)	**Mean (m)**		**(kg/m^3)**	**(m)**	**(m/s)**
0.019-0.038	0.028	quartz	2650	9-18	1.2
0.14-0.27	0.2	agglomerate	2360	2-4	2.1
0.16-0.48	0.3	sandstone	2550	2-4	2.2
0.22-0.73	0.45	sandstone	2550	2-4	3.4
0.33-1.25	0.75	agglomerate	2360	2-4	3.7
0.8-1.15	1.0	basalt	3040	2-4	4.9
0.59-1.59	1.1	basalt	3040	2-4	5.5
1.07-2.1	1.5	sandstone	2550	2-4	5.7
0.74-2.51	1.6	basalt	3040	2-4	6.4
1.23-3.05	2.0	basalt	3040	2-4	6.9

Table 3.2.4 *Summary of critical conditions for large size particles/rocks in wave conditions*

Hall (2010) and Hansom et al. (2008) have studied the movement of natural boulders in conditions with breaking waves at the shore platform of East Lothian on the high-energy, macro-tidal North Sea coast of Scotland. Boulders with volumes of more than 0.5 m^3 have been moved landward over extensive areas of the shore platform. Velocity in breaking waves are estimated to have reached values of 3 to 4 m/s on the platform, especially in the sligthly deeper channels eroded in the platform floor. Sliding is the dominant mechanism of movement for irregular shaped (mega) clasts. Rolling and overturning processes occur for platy clasts. Boulder sizes and estimated critical velocities related to boulder sliding are given in **Table 3.2.4** (row 2-10) and in **Figure 3.2.7**.

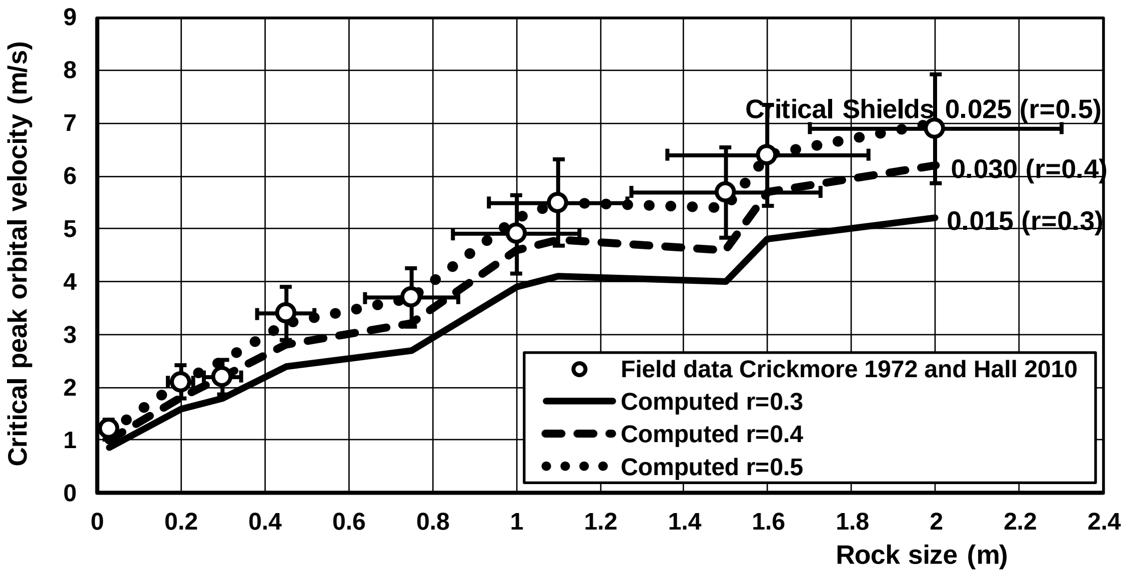


Figure 3.2.7 *Critical peak orbital velocity as function of rock size in waves (γ_{str}=1.2)*

Equation (3.2.9) has been used to compute the critical peak orbital velocity for the boulder sizes of **Table 3.2.4** using: ρ_w= 1020 kg/m^3, α=1=bed roughness coefficient, r=Shields-reduction coefficient=0.3, 0.4 and 0.5 and γ_{str}=1.2= velocity enhancement coefficient. This latter coefficient represents the effect that the boulders at the field site are isolated rocks fully exposed to breaking waves with extreme turbulence levels.
Computed results are shown in **Figure 3.2.7** for r=0.3, 0.4 and 0.5 and γ_{str}=1.2. Fairly good agreement between measured and computed results can be observed for r= 0.4 and r=0.5 in combination with γ_{str}=1.2. These settings also yield the best results for rocks in currents (see **Figure 3.2.7**).

<u>Practical applications bed protection</u>
Equation (3.2.9) has been used to produce a design graph for horizontal bed protections in water depths of h= 3 to 20 m and significant wave heights between H_s= 0.1 and 12 m. The wave period T_p is given by the relationship T_p= 5 $H_s^{0.4}$ (North Sea wave climate). The thickness of the protection layer is set to δ_{bp}= 0.5 m.
The effective water depth is $h_{bp}=h_o-\delta_{bp}$. Other parameters are: density of seawater= 1020 kg/m^3; density of sediment= 2650 kg/m^3; γ_s=1= safety factor, γ_{str}=1, α=2, r= 0.5 and $\theta_{cr,shields}$ = 0.05. The computed results are shown in **Figure 3.2.8**.
For example: h=17 m and H_s= 4.6 m (T_p= 9.2 s) yields: $D_{n,50}$= 0.048 m for r= 0.5 and $N_{moving\ rocks}$= 10 rocks/m/day. This latter parameter can be reduced to N_{mr}=0.033 by using a smaller rock size $D_{n,50}$=0.066 m (r= 0.4).

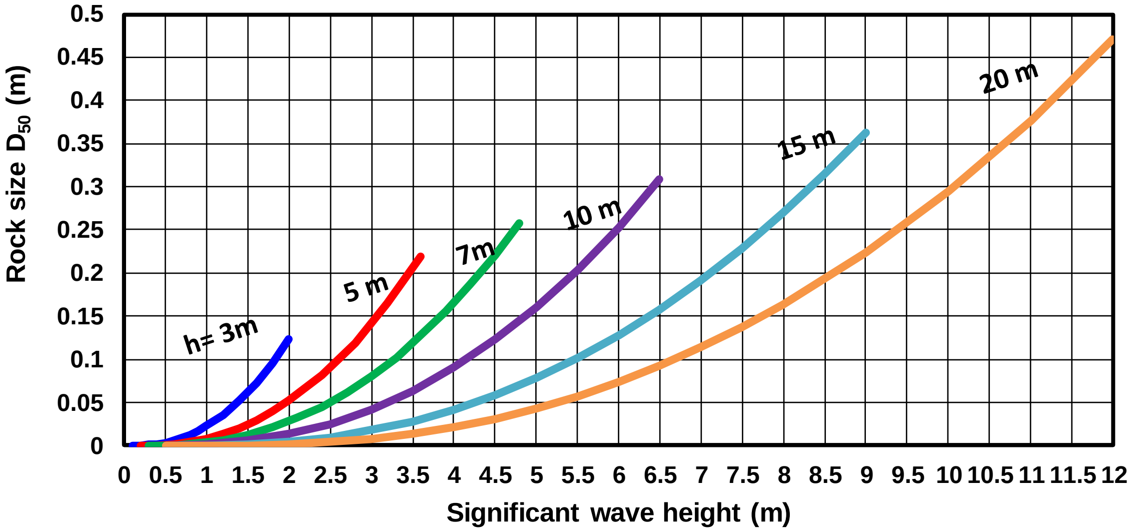


Figure 3.2.8 *Design graph (r= 0.5; γ_{str}=1) for horizontal bed protection in wave conditions*

Practical applications toe protection

The toe structure of a breakwater provides support to the armour layer slope and protects the structure against damage due to scour. Most often, the toe consists of rubble mound of rocks/stones. Usually, the width of the toe varies in the range of 3 to 10 $D_{n,50}$ and the thickness of the toe varies in the range of 2 to 5 $D_{n,50}$ depending on the conditions, see **Figure 3.3.10** (see **Section 3.3.6**).

Figure 3.2.9 shows the computed rock size for increasing depth above the toe protection based on Equation (3.2.9), Equation (3.3.17) of Van der Meer (1998) and Equation (3.3.18) of Van Gent and Van der Werf (2014). The variation of the peak orbital velocity is also shown (right axis). Equation (3.3.18) of Van Gent and Van der Werf (2014) is shown for two wave lengths (deep water and local depth). Using the deep water wave length, the rock size is about 20% larger. Equation (3.2.9) of Van Rijn is applied with α=2, r=0.5, γ_s=1 (no safety factor) and γ_{str}=1.5 (to include effects of breaking, up and downrush and reflecting waves). As Equation (3.2.9) is related to the peak orbital velocity, the rock size decreases markedly for increasing water depth at the toe. The rock size increases by about 20% if a longshore current of 1 m/s is present (Equation 3.2.11 of Van Rijn). Equations (3.3.17) and (3.3.18) are only weakly related to water depth and cannot take the longshore current effect into account. All equations yield very similar results (rock sizes of 0.3 to 0.4 m) for worst-case conditions with breaking waves ($H_{s,toe}/h_{toe} \cong$ 0.5 to 0.6).

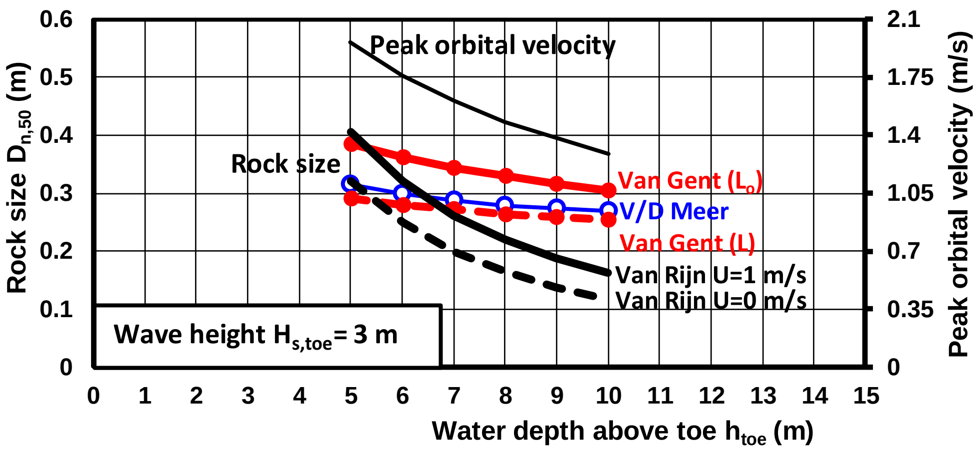


Figure 3.2.9 *Rock size of toe protections*

3.2.5 Stability and damage equations for rocks in currents plus waves

Using the available formulae (3.2.3, 3.2.4 and 3.2.5), the critical diameter can be expressed as:

$$D_{n,50} = \frac{\tau_{b,cw}}{(\rho_s - \rho_w)\, g\, (K_{\alpha 1} K_{\alpha 2}\, r\, \theta_{cr,o})} \qquad (3.2.10)$$

with: $\tau_{b,cw}$ = shear stress at granular material due to currents plus waves (see Equation (3.2.4))
Equation (3.2.10) can be expressed as:

$$D_{n,50} = \frac{\gamma_s\, [0.013\, (h/\alpha)^{-0.3}\, (\gamma_{str}\, \bar{u})^2 + 0.045\, (T_p/\alpha)^{-0.3}\, (\gamma_{str}\, \hat{U})^{1.7}]^{1.4}}{[\Delta\, g\, K_{\alpha 1} K_{\alpha 2}\, r\, \theta_{cr,shields}]^{1.4}} \qquad (3.2.11)$$

$\theta_{cr,shields}$ = critical Shields mobility parameter based on Equation 3.2.3; ($\theta_{cr,shields} \cong 0.05$ for coarse materials);
r = reduction factor = 0.5-1 (larger r-value yields smaller diameter);
α = bed roughness coefficient ($k_s=\alpha D_{n,50}$ with α=1 to 2); larger α yields larger diameter;
γ_s = safety factor;
γ_{str}= velocity+turbulence enhancement factor due to the presence of a structure (bed protection around a monopile); γ_{str}= 1 to 1.5, =1 if no structure is present.

Equation (3.2.11) is implemented in the **ARMOUR.xls** model. Equations (3.2.8b) can be used to get an estimate of the number of moving rocks (damage) during a given period (**ARMOUR.xls**).
The damage parameter S_d (see **Section 3.3.1**) can be estimated by:
$S_d = A_e/(D_{n,50})^2 = \Delta t\, [1/(1-\varepsilon)]\, N_{movingrocks}\, V_{1rock}/(D_{n,50})^2 = \Delta t\, N_{movingrocks}\, (\pi/6)/(1-\varepsilon)\, (D_{n,50})^3/(D_{n,50})^2$
$\cong \Delta t\, N_{movingrocks}\, D_{n,50}$
with: A_e= eroded area per unit width in a given time period Δt, Δt= time period considered (usually 5000 to 10000 waves or about 1 day of storm), ε= porosity factor ($\cong$0.45) and $N_{movingrocks}$ = number of moving rocks per m width and second.
Start of damage during 1 storm with duration of 1 day for $S_d \cong 1$, see **Table 3.3.1**.
Minor damage during 1 storm with duration of 1 day for $S_d \cong 2$, see **Table 3.3.1**.

<u>Practical application of rock bed protection near monopiles</u>
Bed protections around monopiles of wind mills in coastal waters are intensively studied. Equation (3.2.11) can be used to design the rock size of bed protection around the monoplile (see **Figure 3.2.10**) provided that the effect of the structure on the local velocity field is known with sufficient accuracy. This effect is represented by the γ_{str}-coefficient (range of 1 to 1.5) of Equation (3.2.11). **Miles et al. (2017)** have studied the current and wave fields around a monopile at a scale of 1 to 25 in a wave-current basin. The waves are normal to the current. Based on their measured data, it can be concluded that:

- the most significant current-related wake region downstream of the pile has a length of 5D; the total distance of disturbed velocities is about 10D; the maximum turbulent velocities do occur at a distance of 2D downstream of the pile centre; the maximum standard deviation of the instantaneous velocities at that location is about $\sigma_U=0.7U_{c,o}$ with $U_{c,o}$= current velocity upstream of pile;
- the maximum velocity at both sides of the pile is about $U_{c,local}=1.35U_{c,o}$ at 0.75D from the pile centre (normal to main current direction);
- the wave-related influence zone with disturbed orbital velocities is about 3D on both sides of the pile (waves only); the maximum orbital velocity in the influence zone is about $U_{w,local}=1.85U_{w,o}$ with $U_{w,o}$= (undisturbed) near-bed orbital velocity outside influence zone.

De Vos et al. (2012) have performed experimental work (in a wave-current basin) on circular bed protections around a monopile foundation (windmill pile). The overall diameter of the circular bed protection is about $5D_{pile}$ with D_{pile}=pile diameter. The thickness of the bed protection is 2.5 to $3D_{n,50}$. Various sizes of angular protection material (sediment density of 2650 kg/m^3) have been used: $D_{n,50}$= 3.5, 5 and 7.2 mm. The protection material was placed on top of the sand bed (D_{50}= 0.1 mm).

Test	Stone size (mm)	Number of waves (-)	Water depth (m)	Depth-mean velocity u_{mean} (m/s)	Significant wave height H_s (m)	Peak wave period T_p (s)	Dimensionless parameter related to scour of stones S_{3d} (-)	Computed γ_{str} (-)	Computed S_{damage} (-)
6	3.5	5000	0.4	0.08	0.135	1.42	0.60; D.L.=3	1.2	<0.1
14	3.5	5000	0.4	0.164	0.088	1.42	0.24; D.L.=1	1.6	<0.1
19	3.5	3000	0.4	0.163	0.130	1.71	0.94; D.L.=3	1.1	<0.1
50	5.0	5000	0.4	0.224	0.109	1.71	0.64; D.L.=3	1.4	<0.1
52	5.0	3000	0.4	0.315	0.058	1.71	1.21; D.L.=4	1.6	0.15
53	7.2	5000	0.4	0.156	0.145	1.71	0.35; D.L.=2	1.4	<0.1
54	7.2	5000	0.4	0.221	0.121	1.42	0.19; D.L.=2	1.6	<0.1
73	7.2	3000	0.4	0.066	0.151	1.71	0.98; D.L.=3	1.4	<0.1
77	7.2	3000	0.4	0.203	0.122	1.42	0.99; D.L.=3	1.6	<0.1
84	5.0	3000	0.4	0.214	0.135	1.42	0.40; D.L.=2	1.3	<0.1

D.L.=Damage Level; D.L.=1=no movement; D.L.=2= very limited movement of stones (S_{3d}=0.3-0.5); D.L.=3 (S_{3d}=0.5-1); D.L.=4= failure (S_d> 1)

Table 3.2.5 *Stability test results of circular bed protections around a monopile*

Ten test results (**Table 3.2.5**) have been used to calibrate the γ_{str}-parameter of Equation (2.5.1) using: s=2.6, δ_{bp}=0.02 m= thickness protection layer, α=2, r=0.5, $\theta_{cr,shields}$=0.05, and γ_s=1 (safety factor). The γ_{str}-parameter represents the effect of the monopile on the enhancement of the local depth-mean velocity and the additional turbulence generated by the pile structure. The results are shown in **Table 3.2.5** (last two columns). The computed γ_{str}-values vary in the range 1.1 to 1.8 or γ_{str} =1.4$\pm$0.3 (about 20% variation). Miles et al. (2017) have found that the local velocity increase due to the presence of the pile is about 35%, which is in agreement with a velocity enhancement coefficient of γ_{str} =1.4.

The construction of a monopile in the seabed requires the protection of the local bed around the pile by a cover stone layer, see **Figure 3.2.10**. Often, the protection layer consists of a foundation layer and a cover layer. The length of the foundation layer is determined by the acceptable scour depth. The scour depth is of the order of $d_{s,max}\cong D_{pile}$ for $L_{foundation}$=$3D_{pile}$ and $d_{s,max}\cong 0.1D_{pile}$ for $L_{foundation}$=$6D_{pile}$ (Whitehouse et al. 2008).

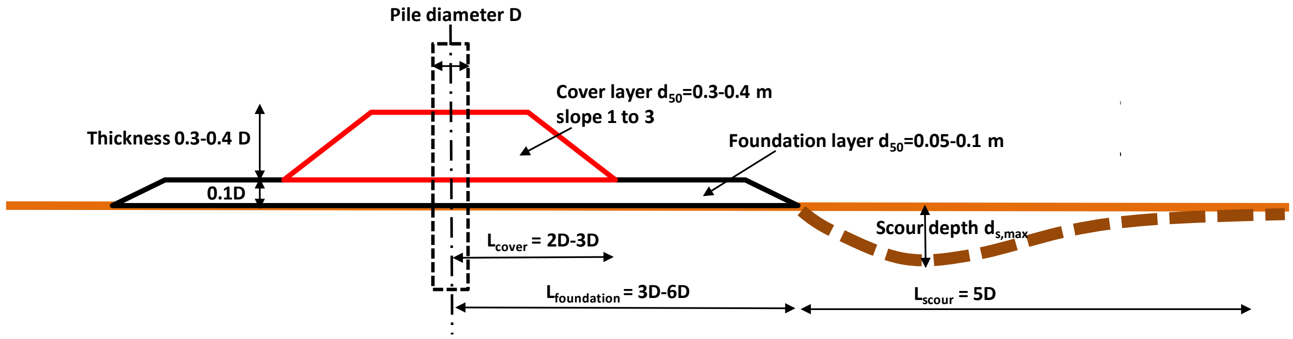


Figure 3.2.10 *Bed protection layers around monopile*

De Vos et al. (2012) have defined a field example case with the following values: D_{pile}= 5 m, water depth h= 20 m, current U_{mean}= 1.5 m/s, significant wave height H_s= 6.5 m, peak wave period T_p= 11.2 s, density seawater ρ_w=1020 kg/m^3, thickness of protection layer δ_{bp}= 2 m.
The expression of the De Vos et al. (2012) yields: $D_{n,50}$= 0.34 m.
Equation (3.2.11) has also been used for this example. Using: effective water depth $h_{bp}=h_o-\delta_{bp}$=20-2=18 m, r= 0.5, $\theta_{cr,shields}$= 0.05 and α= 2, the rock size is: $D_{n,50}$= 0.36 m and $N_{moving\ rocks}$= 0.5 moving rocks/m/day ($S_d \cong 0.2$ for 1 day) for γ_{str}= 1.4. Thus, Equation (3.2.11) yields almost the same result as that of De Vos et al.

3.3 Critical wave height method

3.3.1 Stability equations; definitions

Hudson equation
A classic formula for the stability of rocks/stones under breaking waves at a sloping surface is given by the Hudson formula (Hudson 1958; Rock Manual, 2007), for waves perpendicular to the structure, which reads as:

$$W \geq \frac{\gamma_r H^3}{K_D \Delta \cot(\alpha)} \qquad (3.3.1.)$$

with:
W = weight of unit (= g M);
M = mass of units of uniform size/mass, usually M_{50} for non-uniform rock units (kg/m^3);
M_{50} = mass that separates 50% larger and 50% finer by mass for rock units;
$D_{n,50}$ = nominal diameter of rock unit= $(M_{50}/\rho_r)^{1/3}$,(m);
$D_{n,50} \cong$ 0.8 -0.9 D_{50} for smaller stones (0.05 to 0.15 m; Verhagen and Jansen, 2014);
H = wave height used by Hudson ($\cong$ 1.27 $H_{s,\ toe}$), (m);
$H_{s,toe}$ = significant wave height at toe of structure (m);
K_D = stability coefficient based on laboratory test results (-);
γ_r = specific weight of rock (= g ρ_r)
ρ_r = density of rock ($\cong$ 2700 kg/m^3 for rock and 2300 kg/m^3 for concrete);
ρ_w = density of seawater ($\cong$ 1030 kg/m^3);
α = slope angle of structure with horizontal;
Δ = relative mass density of rock= $(\rho_r-\rho_w)/\rho_w$;
g = acceleration of gravity (9.81 m/s^2).

Equation (3.3.1) can be rearranged into:

$$\frac{H_{s,toe}}{\Delta\ D_{n,50}} \leq 0.8\ [(K_D \cot(\alpha)]^{1/3} \qquad (3.3.2)$$

$$\frac{H_{s,toe}}{\Delta\ D_{n,50}} \leq N_{cr} \qquad (3.3.3a)$$

Using a safety factor (γ_s) and taking oblique waves (γ_{Beta}) into account, it follows that:

$$\frac{H_{s,toe}}{\Delta\ D_{n,50}} \leq N_{cr}/(\gamma_s\,\gamma_{Beta}) \qquad (3.3.3b)$$

with:
γ_s = safety factor (>1),
γ_{Beta} = obliqueness or wave angle factor (= 1 for perpendicular waves and <1 for oblique waves; see Van Gent 2014).

The rock/stone size is given by:

$$D_{n,50} \geq \frac{(\gamma_s\,\gamma_{Beta})\ H_{s,toe}}{\Delta\ N_{cr}} \qquad (3.3.4)$$

with: $N_{cr} = 0.8\ [(K_D \cot(\alpha)]^{1/3}$ = critical value (= stability number) (3.3.5)
Rocks/stones are stable if Equation (3.3.4) is satisfied.
For example: $H_{s,toe} = 5$ m, $N_{cr} = 2$, $\gamma_s = 1$, $\gamma_{Beta} = 1$ and $\Delta = 1.62$, yields $D_{n,50} = 1.55$ m.

The value N_{cr} is a function of many variables, as follows:

N_{cr}= F(*type of unit, type of placement, slope angle, crest height, type of breaking waves, wave steepness, wave spectrum, permeability of underlayers, acceptable damage*)

and can only be determined with sufficient accuracy by using scale model tests of the armour units including the geometry/layout of the whole structure. Many laboratory test results can be found in the Literature. Generally, the N_{cr}-values are in the range of 1.5 to 3. This range of N_{cr}-values yields $D_{n,50}$-values in the range of $D_{n,50} = 0.2$ to $0.4 H_{s,toe}$.

The N_{cr}-value is found to increase (resulting in smaller diameter of the armour units) with:

- decreasing wave steepness ($\approx s^{-0.1}$);
- decreasing crest height (low-crested structure);
- decreasing slope angle;
- larger packing density (more friction between units);
- higher permeability of the underlayer (less reflectivity of the structure);
- more orderly placement (closely fitted).

Two types of stability can be distinguished:

- statically stable: structures designed to survive extreme events with very minor damage;
- dynamically stable: structures designed to survive extreme events with minor damage and reshaping of the outer armour layer.

Damage and safety

An important parameter is the acceptable/allowable damage level in relation to the construction and maintenance costs. If a larger damage level can be accepted, the value of N_{cr} increases resulting in a smaller rock size. This will reduce the construction costs, but it will increase the maintenance cost. The value of N_{cr} may never be taken so large ($\cong 4$) that the armour units are close to failure during design conditions.

A safety factor (γ_s) should be used to deal with the:
- uncertainty of the input variables (boundary conditions);
- uncertainty of the empirical relationships used (often a curve through a cloud of data points, while the envelope of the data is a more safe curve);
- type of structure (single or double armour layer).

The damage is described by various parameters, as follows (see **Tables 3.3.1** and **3.3.2**):
- $S_d = A_e/(D_{n,50})^2$ for rock units with A_e = area of displaced rocks/stones in cross-section of the armour layer (including pores) above and below the design water level after time period Δt (see **Figure 3.3.1**); S_d = non-dimensional parameter; S_d is mostly used for reshaping slopes of rocks; Δt= storm duration (about 5000 waves); volumetric bed load transport (Incl. pores) is defined as: $q_{b,vol}= A_e/\Delta t$ (in m²/s including pores); $q_{b,vol}=1/[(1-p)\ \rho_s]q_{b,mass}$ with p = porosity factor (≅0.45) yielding: $S_d=q_{b,vol}\ \Delta t/(D_{n,50})^2 = q_{b,mass}\ \Delta t/[(1-p)\ \rho_s\ (D_{n,50})^2]$
- N_{od} = number of concrete units displaced over a width of 1 nominal diameter ($D_{n,50}$) along the longitudinal axis of the structure for concrete units after period Δt ($S_d \cong (1-p)N_{od}$ with $p \cong 0.45$);
- N = percentage of damage (%) = N_{od}/n with n= number of total units between crest and toe over a width of 1 nominal diameter after period Δt;
- Number of moving rocks per unit with and time is:
  $N_{mr} = q_{b,mass}/[(\pi/6)\ \rho_s\ D_{n,50}^3]= (1-p)\ \rho_s\ q_{b,vol}/[(\pi/6)\ \rho_s\ D_{n,50}^3]= (1-p)/(0.167\pi)\ q_{b,vol}/D_{n,50}^3$
  $= (1-p)/(0.167\pi)\ S_d\ D_{n,50}^2/(\Delta t\ D_{n,50}^3) \cong S_d/(D_{n,50}\ \Delta t)$.

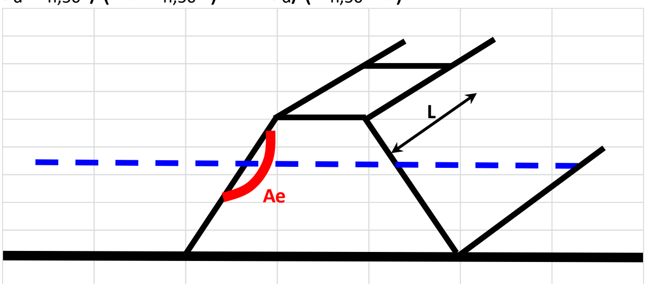


Figure 3.3.1 *Area of displaced stones/rocks*

The total volume of displaced stones for a section with length L is (see **Figure 3.3.1**): $V_e = A_eL$ with A_e is area of displaced stones of cross-shore armour slope (incl. pores).
The volume of displaced stones including porosity is equal to: $V_e = n\ (D_{n,50})^3/(1-p)$ with n = number of displaced stones over section with length L, p = porosity (≅ 0.45).
This yields: $S_d = A_e/(D_{n,50})^2 = V_e/[L(D_{n,50})^2]$ =n $D_{n,50}/((1-p)L) = N_{od}/(1-p)$.
The N_{od} parameter is: $N_{od} = n(D_{n,50}/L)$ and thus N_{od}= n if $L=D_{n,50}$.
Using $S_d = 1$, L= 100 m, p =0.45 and $D_{n,50} = 1$ m, yields: n = 55 displaced units.
N_{od}= 1x55/100 = 0.55 and $N_{od} = (1-p)\ S_d = 0.55$ or S_d=0.55/(1-0.45)=1.

Rock armour slope	Start of damage		Minor damage		Severe damage		Failure
	S_d	N_{od}	S_d	N_{od}	S_d	N_{od}	S_d
1 to 1.5	1	0.5	2	1	4	2	8
1 to 2	1	0.5	2	1	5	2	8
1 to 3	1	0.5	2	1	8		12
1 to 4	1	0.5	3		10		17
1 to 6	1	0.5	3		10		17

Table 3.3.1 *Damage S_d for rock and concrete armour slopes*

An example is shown in **Table 3.3.2** for a breakwater with cubes (n = 20 units between toe and crest) and D_n = 1.85 m.

Damage in number of units over a length of L = 150 m along the axis of the structure	Damage N_{od} (-)	Damage N (%)
n_{150}= 16 units	$(D_n/150)$ x 16= 0.2	0.20/20 x 100% = 1 %
n_{150}= 34 units	$(D_n/150)$ x 34= 0.42	0.42/20 x 100% = 2.1%
n_{150}= 73 units	$(D_n/150)$ x 73= 0.9	0.90/20 x 100% = 4.5%

Table 3.3.2 *Damage example for concrete armour units*

Any damage of an armour layer of ***single*** concrete units (one layer) is not acceptable, as it will lead to exposure of the underlayer and rapidly progressing failure of other units. Therefore, the safety factor for units in a single layer is relatively large (γ_s = 1.3 to 1.5). Failure of a **single** layer of concrete units often is defined as N_{od}= 0.2.

Minor damage ($N_{od}\cong$ 0.5 to 1) of a ***double*** layer of rock units is acceptable (see **Table 3.3.1**), as the failure of an individual unit will not immediately lead to exposure of the underlayer. Generally, a double-layer armour slope will not fail completely, but the slope will be reshaped into a more S-type profile because units from higher up near the crest will be carried toward the toe during extreme events. The damage will gradually increase with increasing wave height until failure. Therefore, the safety factor of a double layer rock slope can be taken as: γ_s= 1.1 to 1.3

Furthermore , it is noted that the safety factor for weight is higher than that for size: $\gamma_{s,weight} = (\gamma_{s,size})^3$.

Thus: $\gamma_{s,size}$= 1.1 means $\gamma_{s,weight}$= 1.33
$\gamma_{s,size}$= 1.3 means $\gamma_{s,weight}$= 2.2
$\gamma_{s,size}$= 1.5 means $\gamma_{s,weight}$= 3.4

The damage S_d is related to the bed load transport of stones (in kg/m/s) by:
$q_b= (1-p)\rho_s A_e/\Delta t= (1-p)\rho_s A_e/(N_{waves}\, T_p) =(1-p)\rho_s\, S_d\, (D_{n,50})^2/(N_{waves}\, T_p)$
with: N_{waves}= number of waves, T_p= wave period.

3.3.2 Stability equations for high-crested conventional breakwaters

A breakwater is high-crested, if $R_c > 4\, D_{n,50}$ with R_c= crest height above still water level, see also **Figure 2.5.1** or **3.3.6**.

Typical features are:
- relatively high crest with minor overtopping;
- relatively steep slopes between 1 to 1.5 and 1 to 2.5;
- permeable underlayers and core;
- relatively high wave heights up to 3 m at the toe;
- mostly used in the nearshore with depths (to MSL) up to 8 m.

Various types of armour units are used:
- randomly used rocks in two layers under water and above water;
- orderly placed rocks in one or two layer above the low water level;
- orderly placed concrete units in one and two layers (cubes and tetrapods);
- concrete units placed in one layer with strict pattern (Accropodes, Core-Locs, Xblocs).

3.3.2.1 Randomply placed rocks in double layer

Some values of critical stability numbers of rock armour units based on laboratory tests (Van der Meer, 1988, 1999; Nurmohamed et al.,2006; Van Gent et al., 2003) are given in **Table 3.3.3**. More information is given in the Rock Manual (2007). The stability values at initiation of damage (movement) vary in the range 1.3 to 1.7 for randomly placed rocks and in the range of 1.7 to 2.2 for orderly placed rocks. The start of damage (S_d= 0,N_{od}= 0) for rocks, cubes and tetrapods in a double layer is almost the same.

The design stability numbers of rocks and concrete cubes randomly placed in a double layer accepting minor damage are $N_{cr,design,minor\ damage} \cong 1.5$ and 2, which is the same as that of rocks and cubes orderly placed in a single layer accepting no damage ($N_{cr,design,no\ damage}$) due to the use of different safety factors (1.1 and 1.5), see **Table 3.3.1**.

Types of units	Placement	Number of layers	Slope of armour layer	N_{cr} no damage (N_{od}=0) (S_d=0)	N_{cr} minor damage (N_{od}=0.5) (S_d=2)	N_{cr} Failure	$N_{cr,design}$ no damage	$N_{cr,design}$ minor damage
Rocks	Randomly Orderly Orderly	2 2 1	1 to 1.5 1 to 1.5 1 to 2	1.3 to 1.7 1.7 to 2.2 2.2	1.7 2.0 -	2.5 to 3.0 3.0 to 3.5 3.0	1.1 (γ_s=1.1) 1.5 (γ_s=1.1) 1.5 (γ_s=1.5)	1.5 (γ_s=1.1) 2.0 (γ_s=1.1) -
Cubes	Randomly Irregularly (n_p=0.7) Orderly (n_p=0.7)	2 1 1	1 to 1.5 1 to 1.5 1 to 1.5	1.5 to 2.2 2.2 to 2.5 3.0	2.2 - -	3.0 3.8 4.5	1.3 (γ_s=1.1) 1.5 (γ_s=1.5) 2.0 (γ_s=1.5)	2 (γ_s=1.1) - -
Tetrapods	Randomly	2	1 to 1.5	1.5 to 2.2	2.2	3.0	1.3 (γ_s=1.1)	2 (γ_s=1.1)
Accropodes	Strict pattern for interlocking	1	1 to 1.33	3.7	-	4.1	2.5 (γ_s=1.5)	-
Core Locs	Strict pattern for interlocking	1	1 to 1.33	4.2	-	4.5	2.8 (γ_s=1.5)	-
Xblocs	Strict pattern for interlocking	1	1 to 1.33	4.2	-	4.5	2.8 (γ_s=1.5)	-

γ_s= safety factor = $N_{cr,start\ damage}/N_{cr,design}$

γ_s= 1.1 to 1.3 for double layer units; γ_s= 1.3 to 1.5 for single layer units; n_p= packing density

Table 3.3.3 *Stability of various armour units (Van der Meer, 1988, 1999; Nurmohamed et al., 2006)*

Various formulae are available to determine the nominal diameter of rock armour units on the seaward side of non-overtopped, high-crested breakwaters (Rock Manual 2007). The formuale should be valid for shallow water, as rock-type breakwaters are mostly built in shallow water. In deep water rock-type breakwaters are not very common; caisson-type breakwaters are more economical in deep water conditions. Waves start breaking at a mild sloping foreland if $H_s > 0.3$ h, with h= local water depth.

Thus, deep water with non-breaking waves can be crudely formulated as h > $3H_{s,toe}$. Using $H_{s,toe}$ = 3 to 5 m, the water depth is 10 to 15 m. Rock-type breakwaters are not very attractive solutions for water depths > 10 m.

Herein, only two formulae based on many laboratory scale tests carried out in The Netherlands, are explained:

- Van Gent et al. 2003;
- Van der Meer 1988.

Van Gent et al. 2003

The formula for ***randomly placed rocks (2 layers) on a slope*** in deep and shallow water reads as

$$N_{cr} = [1.75/(\gamma_{Beta}\,\gamma_s)]\ [\cotan(\alpha)]^{0.5}\ [1 + P_G]\ [S_d/N_w^{0.5}]^{0.2} \qquad (3.3.6)$$

with:

P_G = $D_{n50,core}/D_{n50}$ = permeability factor of structure (P_G=0= impermeable; P_G=1=fully permeable);

$D_{n50,core}$ = nominal diameter of core material (approximately 0.2 to 0.4 m);

D_{n50} = nominal diameter of armour layer (grading D_{85}/D_{15}< 2.5);

α = slope angle of the structure (not foreland slope);
S_d = damage = A_e/D_{n50}^2, S_d = 2 = minor damage (design value), S_d = 10 = failure;
N_w = number of waves during a storm event (1000 to 3000 for storm event of 6 hours);
γ_s = safety factor for deterministic design method (= 1.1 for permeable structures and 1.3 for impermeable structures);
γ_{Beta} = 0.5 + 0.5$(\cos\beta)^2$= reduction factor oblique waves; $\beta = 0^o$ for perpendicular waves (**Fig. 2.5.4**).

Equation (3.3.6) is rather accurate for structures with a permeable core in both deep water and shallow water, but less accurate for impermeable cores. The $D_{n,50}$-value increases with decreasing permeability and decreases for less steep slopes.

According to Van Gent et al. (2003), the effects of wave steepness, wave period and the ratio $H_s/H_{2\%}$ are relatively small and can be neglected.
The validity ranges of Equation (3.3.6) roughly are: wave steepness = 0.01 to 0.06; surf similarity = 1.3 to 15, relative wave height $H_{s,o}/h_{toe}$ = 0.2 to 0.7.
Stability formulae for rock armour layers are usually applied assuming perpendicular wave attack. Often, the effects of oblique waves are neglected. Based on many tests simulating oblique waves on a rubble mound breakwater in relatively deep water (no wave breaking on the foreland), Van Gent (2014) has proposed a size reduction factor (increase of stability) for situations with oblique waves. The effect is found to be relatively small for small wave angles and relatively large for larger wave angles. For $\beta = 90^o$ (wave propagation parallel to axis of structure, see **Figure 2.5.4**), the reduction effect is about 0.35 to 0.45.
To be on the safe side, it is herein proposed to use: $\gamma_{Beta} = 0.5 + 0.5(\cos\beta)^2$, which yields a maximum reduction effect of γ_{Beta}= 0.5 for $\beta = 90^o$ (parallel waves) and no reduction of γ_{Beta} = 1 for $\beta = 0^o$ (perpendicular waves).
Using: P_G= 0.3, S_d= 2 (start of damage) and N_w=2500, it follows that: $N_{cr} = [1.2/(\gamma_s \gamma_{Beta})]\ [\cot(\alpha)]^{0.5}$

This yields the following design values of N_{cr}:
cotan(α) = 1.5 $N_{cr}= 1.45/(\gamma_s \gamma_{Beta})$
cotan(α) = 2.0 $N_{cr}= 1.70/(\gamma_s \gamma_{Beta})$
cotan(α) = 2.5 $N_{cr}= 1.90/(\gamma_s \gamma_{Beta})$

Equation (3.3.6) and also Equation (3.3.7) are related to $S_d^{0.2}$. This means that the rock size is 15% larger, if S_d = 1 instead of S_d = 2 (reduction of factor of 2) and 30% larger if S_d = 0.5 instead of S_d = 2 (reduction of factor of 4).

<u>Van der Meer 1988</u>
The formula for ***randomly placed rocks (2 layers) on a slope*** in deep and shallow water reads as

$$N_{cr} = [C_{plunging}/(\gamma_{Beta}\ \gamma_s)]\ P^{0.18}\ \xi^{-0.5}\ \gamma_H\ [S_d/N_w^{0.5}]^{0.2} \text{ for plunging waves } \xi<\xi_{critical} \text{ and } \cot(\alpha)\geq 4 \quad (3.3.7a)$$

$$N_{cr} = [C_{surging}/(\gamma_{Beta}\ \gamma_s)]\ P^{-0.13}\ \xi^{P}\ \gamma_H\ [\tan(\alpha)]^{-0.5}\ [S_d/N_w^{0.5}]^{0.2} \text{ for surging wave conditions } \xi>\xi_{critical} \quad (3.3.7b)$$

$$\xi_{critical} = [(C_{plunging}/C_{surging})\ P^{0.31}\ [\tan(\alpha)]^{0.5}]^{R} \quad (3.3.7c)$$

with:
P = permeability factor of structure (P = 0.1 for impermeable core; P = 0.4 for rocks on a semi-permeable filter layer; P = 0.5 for permeable core; P = 0.6 for fully permeable structure with uniform rock);
S_d = damage = A_e/D_{n50}^2, S_d = 2 minor damage (design value), S_d =10 = failure;
N_w = number of waves during a storm event (1000 to 3000 for storm event of 6 hours);
R = 1/(P+0.5) = exponent;
α = slope angle of the structure (not the foreland);
ξ = $\tan\alpha/[(2\pi/g)H_{s,toe}/T^2_{mean}]^{0.5}$ = surf similarity parameter based on the T_{mean} wave period;

T_{mean} = mean wave period at toe of the structure ($\cong$0.85 T_p for single peaked spectrum),
γ_H = $H_s/H_{2\%}$ = ratio of wave heights at toe of breakwater (= 0.71 for deep water and 0.85 for shallow water with breaking waves);
γ_s = safety factor for deterministic design method (= 1.1 to 1.3);
γ_{Beta} = 0.5 + $0.5(\cos\beta)^2$ = reduction factor for oblique waves; $\beta = 0^o$ for waves perpendicular and γ_{Beta}=1.

The following steps are required:

- determination of wave conditions and number of waves (offshore and at toe of structure);
- determination of preliminary dimensions of cross-section (slope angle, crest height, core, etc);
- determination of allowable damage (in cooperation with client);
- determination of surf similarity parameter;
- determination of permeability value;
- determination of rock dimensions for different scenarios;
- finalization of design based on iteration and sensitivity analysis;
- verification of design based on scale model tests.

Table 3.3.4 shows the coefficients of the Van der Meer formula 1988.

Equation (3.3.7a) shows that the $D_{n,50}$-value increases with decreasing permeability.
Equation (3.3.7a) shows that: $D_{n,50} \approx H_s\ \xi^{0.5}$ and thus: $D_{n,50} \approx H_s^{0.75}\ T^{0.5}$. If H_s = constant, the rock size increases with increasing wave period T. Assuming an inaccury of 20% for the wave period T, the rock size increases with 10% for a 20%-increase of the wave period. Thus, it is wise to use a conservative estimate of the wave period. Similarly, assuming an inaccury of 20% for the wave height H_s, the rock size increases with 15% for a 20%-increase of the wave height. This can be taken into account by a safety factor of 1.2.

The original formula proposed by Van der Meer (1988) is most valid for conditions with non-breaking waves in relatively deep water ($h > 3H_{s,toe}$). Only, few tests with wave breaking in shallow water were carried out.
The original coefficients are: $C_{plunging}$ = 6.2, $C_{surging}$ = 1 and γ_H = 1 and the wave period is the mean period T_{mean}.
Later it was proposed to use the γ_H-factor (= 0.71 for a single peaked spectrum) resulting in: $C_{plunging}$ = 8.7 and $C_{surging}$ = 1.4 (Van Gent et al., 2003 and Van Gent 2004).
As the γ_H-factor is larger for shallow water ($\cong$ 0.85), this results in a larger N_{cr}-value for shallow water and thus a smaller rock size. The γ_H-parameter can be simply represented as: $\gamma_H = 0.4(H_s/h) + 0.58$ yielding γ_H =0.7 for $H_s/h \leq 0.3$ and $\gamma_H \geq 0.82$ for H_s/h= 0.6.
Van Gent et al. (2003) have recalibrated the coefficients using more test results (using T_{m-1} instead of T_{mean}) including shallow water conditions resulting in: $C_{plunging}$ = 8.4 and $C_{surging}$ = 1.3; these coefficients represent the trendline through the data points. The T_{m-1} period has been used because it better represents the longer wave components of the wave spectrum. This is of importance for the surf zone where the spectrum may be relatively wide (presence of waves with approximately the same height but different periods).
As the recalibrated coefficients are only slightly different, it can be concluded that the Van der Meer formula is generally valid for relatively deep and shallow water. A safety factor should be applied to obtain the envelope of the data points.

Assuming that Equation (3.3.7) is also valid for shallow water, both the original and the modified formula can be compared for a certain location (with constant H_s, P, N_w and S_d), which leads to:
$D_{n50,modified}/D_{n50,original} = (C_{pl,original}/C_{pl,modified})\ (\gamma_{H,original}/\gamma_{H,modified})\ (T_{m-1}/T_{mean})^{0.5}$
In shallow water: $(C_{pl,original}/C_{pl,modified})$= 8.7/8.4 = 1.04 and $(\gamma_{H,original}/\gamma_{H,modified})$= 0.71/0.85 = 0.84, and thus:
$D_{n50,modified}/D_{n50,original} = 0.87\ (T_{m-1}/T_{mean})^{0.5}$
This yields: $D_{n50,modified}/D_{n50,original} > 1$ if $(T_{m-1}/T_{mean}) > 1.3$

Thus: the modified formula will give a larger $D_{n,50}$ value in shallow water if the T_{m-1} period is larger than $1.3T_{mean}$, which may occur if long wave components are important.

Author	$C_{plunging}$	$C_{surging}$	Wave period	Wave height ratio
Van der Meer 1988 original	6.2	1	T_{Mean}	$\gamma_H = 1$
Modified by Van Gent et al. (2003)	8.7	1.4	$T_{m-1,0}$	$\gamma_H = H_s/H_{2\%}$
Recalibrated by Van Gent et al. (2003)	8.4	1.3	$T_{m-1,0}$	$\gamma_H = H_s/H_{2\%}$

Table 3.3.4 *Coefficients of Van der Meer formula*

Figure 3.3.2 shows the stability of randomly placed rocks based on the formulae of Van Gent et al. 2003 and Van der Meer 1988 as function of the surf similarity parameter ξ. Other data used: S_d= 2, N_{od}= 0.5 (minor damage), N_w= 2100, P_M= 0.5, $H_s/H_{2\%}$= 0.71, tan(α)= 0.5, P_G= 0.3, ρ_{rock} = 2700 kg/m^3, γ_s=1.
Both formulae produce approximately the same results for ξ > 2.5. For very small values of ξ <2.5 (large values of the wave steepness or very small slopes), Equation (3.3.7) of Van der Meer 1988 yields systematically higher stability numbers and thus smaller rock sizes.

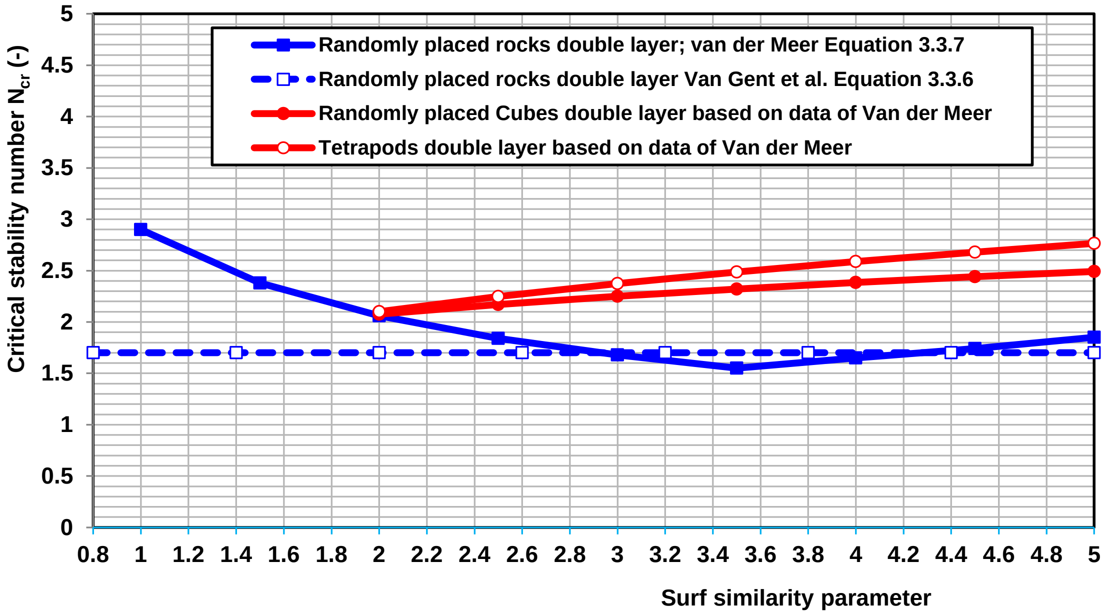


Figure 3.3.2 *Stability values of rocks and concrete armour units at conditions with minor damage (S_d=2, N_{od}=0.5) as function of surf similarity parameter*

Example 1: Rock armour size based on Equations (3.3.6) and (3.3.7)

Armour layer consisting of randomly-placed rocks on a semi-permeable core is exposed to storm events in relatively deep water, see **Table 3.3.5**.
What is the rock size $D_{n,50}$ based on the methods of Van der Meer 1988 and Van Gent et al. 2003 using Equations (3.3.7) and (3.3.6), see ARMOUR.xls(sheet4)?
Here, the parameters $H_{s,toe}$ and $\gamma_H=H_s/H_{2\%}$ are given, but generally wave computations are required to compute these values.
The rock mass can be computed as: $M_{50} = \rho_{rock} (D_{n,50})^3$.
The results (based on a safety factor γ_s = 1 and γ_{Beta}= 1) are shown in **Table 3.3.5**.
The Van der Meer-coefficients are given in **Table 3.3.4**.
Using a larger damage (S_d= 5) for the 100 years storm event, the armour size is about 15% smaller.
Using a safety factor of γ_s = 1.1 , the armour size will be 10% larger.

Using recalibrated coefficients, the rock size of the Van der Meer method is slightly larger (≅10%) due to the larger wave period (T_{m-1} in stead of T_{mean}) yielding a larger surf similarity parameter. The original formula is based on T_{mean}.

Parameters		**Storm event 1**	**Storm event 2**	**Storm event 2**
Return period		25 years	100 years	100 years
Duration		3.5 hours	5.4 hours	5.4 hours
Significant wave height at toe	$H_{s,toe}$	4 m	5 m	5 m
Wave period	T_{mean} ; T_{m-1}	7 s; 8 s	9 s; 10 s	9 s; 10 s
Ratio wave heights	$H_s/H_{2\%}$	0.71	0.71	0.71
Density of rock	ρ_{rock}	2650 kg/m^3	2650 kg/m^3	2650 kg/m^3
Density of seawater	ρ_w	1025 kg/m^3	1025 kg/m^3	1025 kg/m^3
Number of waves	N_w	1800	2160	2160
Permeability factor Van der Meer	P_M	0.4	0.4	0.4
Permeability factor Van Gent	P_G	0.3	0.3	0.3
Damage	S_d	2 (minor)	2 (minor)	5 (severe)
Slope angle	tan(α)	0.33	0.33	0.33
Safety factor	γ_S	1.	1.	1.
Obliqueness/wave angle factor	γ_{Beta}	1.	1.	1.
Computed values				
Surf similarity parameter	ξ	1.44 original 1.65 recalibrated	1.65 1.85	1.65 1.85
Critical surf similarity V.d. Meer	ξ_{cr}	3.0 original 3.1 recalibrated	3.0 3.1	3.0 3.1
Stability number Van der Meer	N_{cr}	2.36 original 2.13 recalibrated	2.17 2.0	2.60 2.38
Rock size Van der Meer	$D_{n,50}$	1.07 m original 1.18 m recalibrated	1.46 m 1.59 m	1.21 m 1.32 m
Rock size Van Gent	$D_{n,50}$	1.17 m	1.49 m	1.24 m

Table 3.3.5 *Rock armour sizes of high-crested breakwaters for storm events; ARMOUR.xls (sheet 4)*

<u>Example 2: Rock armour size based on Equations (3.3.6) and (3.3.7)</u>

What is the effect of wave period on the size of rocks.?
Input data: armour slope 1 to 3 (tanα = 0.33), S_d = 2, N_w = 2160 (6 hours), P_G = 0.3, P_M = 0.4, $\gamma_H = H_s/H_{2\%}$ = 0.71, ρ_s = 2650 kg/m^3, ρ_w = 1025 kg/m^3.
The wave period has been varied in the range of 7 to 20 s. Three wave heights ($H_{s,toe}$ = 3, 4 and 5 m) have been used. A wave height of 3 m in combination with a period in the range of 7 to 10 sec represents short wind waves during storm events, whereas a wave height of 3 m in combination with a period of 15 to 20 s represent large post-storm swell type waves along an open ocean coast (west coast of Portugal, south-west coast of France).
Figure 3.3.3 shows the rock size $D_{n,50}$ as function of the wave period based the formulae of Van der Meer 1988 (original, Equation 3.3.7) and Van Gent et al. 2003 (Equation 3.3.6). The formula of Van Gent et al. 2003 is not dependent on the wave period resulting in constant values of $D_{n,50}$. The formula of Van der Meer 1988 shows an increasing trend for increasing wave periods up to the critical surf similarity parameter and a decreasing trend for larger wave periods. The rock size based on Van der Meer 1988 is significantly larger than that of Van Gent et al. 2003 for waves of 4 to 5 m and wave periods in the range of 14 to 20 s.

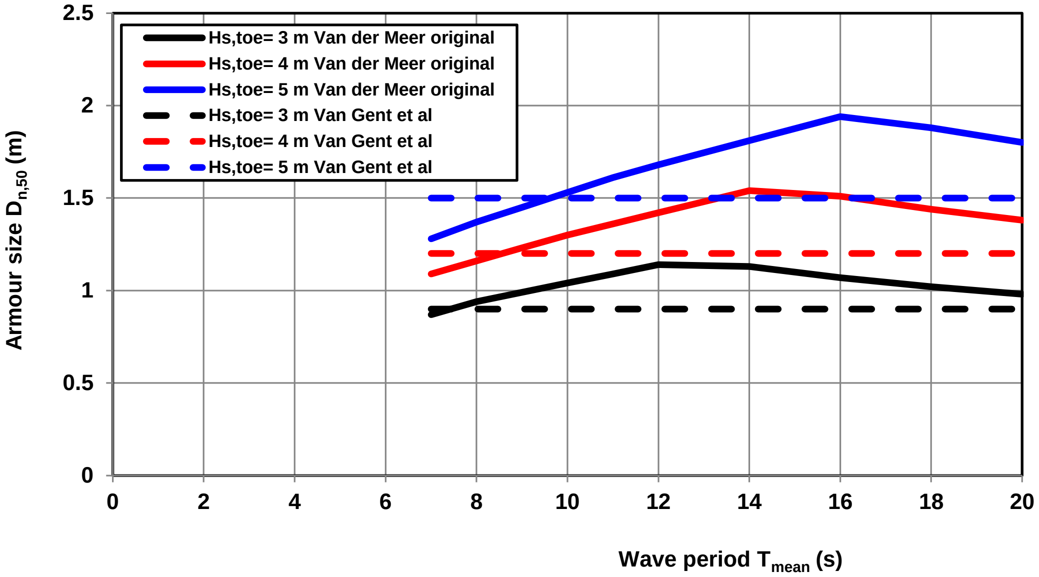


Figure 3.3.3 *Rock size $D_{n,50}$ as function of wave period and wave height*

3.3.2.2 Orderly placed rocks in single layer

In Norway, many breakwaters have been constructed with a ***single*** layer of large rocks placed individually (orderly) by a crane (Hald, 1998).

Nurmohamed et al. (2006) have studied the stability of orderly placed rocks (initial damage) in a **single** layer. The experimental range of the stability numbers is shown in **Figure 3.3.4**. The data refer to slopes in the range of 1 to 1.5 and 1 to 3.5 with permeable underlayers in most cases.

Based on the data of Nurmohamed et al. (2006), the mean trendline of N_{cr}-values can be described by:

$N_{cr} = [4.8/(\gamma_s \gamma_{Beta})]\ \xi^{-0.8}$ for $\xi < 3$ (plunging breaking waves) (3.3.8a)
$N_{cr} = [1.0/(\gamma_s \gamma_{Beta})]\ \xi^{0.6}$ for $\xi \geq 3$ (surging waves) (3.3.8b)

with: ξ = surf similarity parameter and γ_s= safety factor for deterministic design (= 1.5 for orderly placed rocks in a **single** layer; = 1.1 for orderly placed rocks in a **double** layer).
Equation (3.3.8) is shown as the mean trendline in **Figure 3.3.4**.

Using a safety factor of 1.5, the lower envelope of the experimental range is obtained. It can be seen that orderly placed rocks are significantly more stable (15% to 25%) than randomly placed rocks as expressed by the equations of Van Gent et al. 2003 and Van der Meer 1988 based on: S_d= 2, N_w= 2100, P_M= 0.5, $H_s/H_{2\%}$= 0.71, tan(α)= 0.5, P_G= 0.3.

Equation (3.3.8) can also be used for orderly placed rocks in a **double** layer. In that case the safety facor can be reduced to γ_s = 1.1.

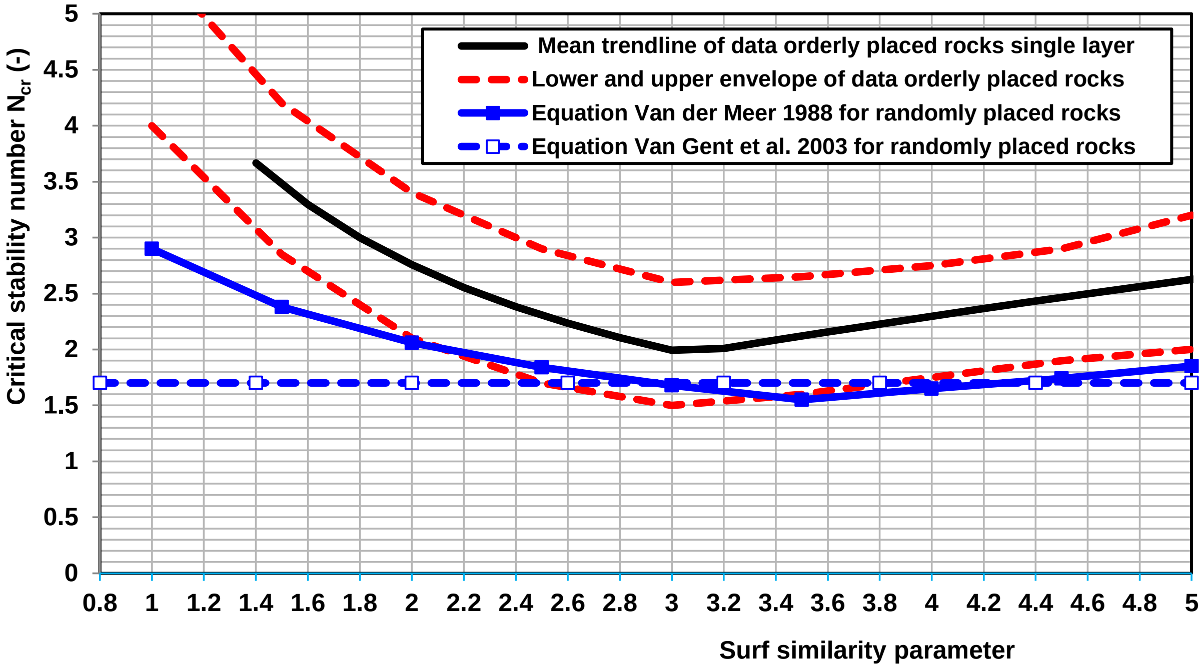


Figure 3.3.4 *Critical stability of orderly placed rocks in a single layer after Nurmohamed et al. (2006)*

3.3.2.3 Randomly placed concrete units in double layer

The critical stability numbers of cubes and tetrapods in a double layer are given by Van der Meer (1988, 1999). His results for wave steepness values in the range of 0.01 to 0.06 and N_{od}= 0.5 (minor damage) and N_w= 2100 are shown in **Figure 3.3.2**. The stability numbers of randomly placed concrete cubes and tetrapods are slightly higher (0 to 15%) than those of randomly placed rocks. The stability of tetrapods is slightly higher than that of cubes. The N_{cr}-values of concrete cubes and tetrapods randomly placed in two layers on a permeable underlayer can be roughly described for preliminary design by ($N_{od} \leq 2$):

Cubes: $N_{cr} = [1/(\gamma_{Beta}\, \gamma_s)]\, [1.1 + N_{od}^{0.5}]\, \xi^{0.2}$ for $0 < N_{od} < 2$ (3.3.9)

$D_{n,50} = [(\gamma_{Beta}\, \gamma_s/\Delta)]\, [1.1 + N_{od}^{0.5}]^{-1}\, \xi^{-0.2}\, H_{s,toe}$

Tetrapods: $N_{cr} = [1/(\gamma_{Beta}\, \gamma_s)]\, [1 + N_{od}^{0.5}]\, \xi^{0.3}$ for $0 < N_{od} < 2$ (3.3.10)

$D_{n,50} = [(\gamma_{Beta}\, \gamma_s/\Delta)]\, [1 + N_{od}^{0.5}]\, \xi^{-0.3}\, H_{s,toe}$

The effects of the armour slope and the wave period are taken into account by the ξ-parameter. The stability number increases slightly with increasing ξ-value (see **Figure 3.3.2**). This means that a steeper armour slope will result in a higher N_{cr}-value and thus a slightly smaller size $D_{n,50}$. A steeper angle of the armour slope yields more friction between the side planes of the cubes (due to gravity).
Equations (3.3.9) and (3.3.10) are only valid for $N_{od} \leq 2$ ($N_{od} > 1$ means severe damage).
Using N_{od} = 0.5 (minor damage) and a safety factor of γ_s= 1.1 (double layer) as acceptable for design, the $N_{cr,design}$ is approximately 2 to 2.4 for cubes and approximately 2 to 2.6 for tetrapods in a double armour layer (ξ in the range of 2 to 6).
Cubes are more stable than rocks due to i) the additional friction forces between the side planes, ii) larger uniformity resulting in a 'smoother' surface. Tetrapods are more stable than cubes and rocks due to the additional interlocking forces.

3.3.2.4 Orderly placed concrete units in single layer

Cubes and cubipods

The stability of cubes in a ***single layer*** strongly depends on the placement pattern and packing density n_p (n_p= area of the blocks in a control area divided by the control area in plan view; the packing density is approximately equal to $n_p \cong 1\text{-}p$ with p= porosity; porosity is the volume of the spaces between the blocks in a control volume divided by the total control volume).

Figure 3.3.5 *Randomly placed (left) and ordely placed (right) in horizontal rows*

Two placement patterns can be distinguished for cubes in a single layer (see **Figure 3.3.5**):

- irregularly placed by dropping the cubes from a crane (packing density in range of 0.65 to 0.75); the cubes should be dropped with the sides making an angle of 45 degrees with the breakwater axis (Verhagen et al., 2002);
- orderly placed at some (small) distance in horizontal rows (packing density of 0.7 to 0.8).

Relatively high stability values can be obtained using an orderly placement pattern, see **Table 3.3.3**.
The packing density should be about 70% (open space of about 30%) to obtain the highest stability. Orderly placed cubes with packing density of about 70% at a slope of 1 to 1.5 are found to be stable up to N_{cr} = 4.8 (no damage; Van Buchem, 2009).
If the packing density is too high (80%) the stability reduces, because the cubes can be more easily pushed out by large overpressure forces under the blocks. The cubes at a slope of 1 to 1.5 are slightly more stable than cubes at a slope of 1 to 2, because the friction forces at the side planes are smaller at a slope of 1 to 2.
The stability reduces if the packing density is relatively small (<70%). Furthermore, relatively small packing densities may result in relatively large gaps at the transition between the slope and the crest due to settlements of the cubes under wave action.
High-density cubes (up to 4000 kg/m^3) can be obtained by using magnetite as aggregate material (Van Gent et al., 2002). If the cube density can be increased to 4000 kg/m^3, the relative density increases with a factor of 2 and the cube size reduces with a factor of 2. This reduces the weight of an individual high- density concrete unit by a factor of 5.

Concrete interlocking units (single layer)

Various types of interlocking concrete units in a single layer have been developed: Accropodes, Core-locs, Xblocs. The stability numbers are given in **Table 3.3.3**. The safety factor is recommended to be γ_s =1.5 (high value for single layer).

3.3.2.5 Example of high-crested conventional breakwater of rock and concrete armour units

A high-crested breakwater is exposed to storm events in relatively deep water, see **Table 3.3.6**.
Water depth to MSL at the toe of the structure = 7 m.
Three storms are considered:
Storm 1: Return period= 25 years, $H_{s,toe}$*= 4 m;* h_{toe} *to MSL= 7 m; Max. water level above MSL= 3 m.*
Storm 2: Return period= 100 years, $H_{s,toe}$*= 6 m;* h_{toe} *to MSL= 7 m; Max. water level above MSL= 4 m.*
Storm 3: Return period= 100 years, $H_{s,toe}$*= 6 m;* h_{toe} *to MSL= 7 m; Max. water level above MSL= 4 m.*

The wave overtopping should be smaller than 10 l/m/s during the 25 years storm event and smaller than 100 l/m/s during the 100 years storm event.

The transmitted wave height in the harbour should be smaller than 0.5 m during the 25 years storm event and smaller than 1 m during the 100 years storm event.

The results (based on the spreadsheet-model ARMOUR.xls) are shown in **Table 3.3.6** (γ_s = 1.1 and γ_{Beta}= 1 = perpendicular waves).

The crest height should be at 10 m above MSL to reduce the wave overtopping rate to less than 100 l/m/s. The maximum transmitted wave height is slightly larger than 1 m.

Accepting minor damage, the rock size is of the order of 2.4 m (36 tonnes) to withstand a storm with a design wave height of 6 m at the toe.

Accepting severe damage for the 100 years storm event, the armour rock size is about 2.1 m (25 tonnes), which is a size reduction of about 10%.

Using a double layer of cubes yields a size of about 2.25 m (minor damage) or 1.95 m (more damage).

Cubes are more stable than rocks due to the additional friction forces between the side planes.

Using a double layer of tetrapods yields a size of about 2.1 m (minor damage) or 1.8 m (more damage).

Tetrapods are slightly more stable than cubes and rocks due to the additional interlocking forces.

Using a single layer of cubes (safety factor= 1.5) yields a size of 2.4 m (32 tonnes; $\rho_{concrete}$= 2.3 t/m^3).

Parameters			Storm 1	Storm 2	Storm 3
Input values:	*Column*				
Density of seawater	F47	ρ_w	1025 kg/m^3	1025 kg/m^3	1025 kg/m^3
Density of rock	F48	ρ_{rock}	2650 kg/m^3	2650 kg/m^3	2650 kg/m^3
Density of concrete	F49	$\rho_{concrete}$	2300 kg/m^3	2300 kg/m^3	2300 kg/m^3
Kinematic viscosity coefficient	F50	ν	10^{-6} m^2/s	10^{-6} m^2/s	10^{-6} m^2/s
Crest width armour	F52	B_c	5 m	5 m	5 m
Total crest width	F53	B_t	15 m	15 m	15 m
Berm	F54	-	0 (none)	0 (none)	0 (none)
Slope angle front	F55	tan(α)	0.5	0.5	0.5
Slope angle rear	F56	tan(α)	0.5	0.5	0.5
Rougness front slope	F58	γ_r	0.45	0.45	0.45
Permeability factor Van Gent	F60	P_G	0.3	0.3	0.3
Permeability factor Van der Meer	F62	P_M	0.4	0.4	0.4
Critical stability number concrete units	F64	N_{cr}	3	3	3
Safety factor runup	F66	γ_s	1.2	1.2	1.2
Safety factor wave overtopping	F67	γ_s	1.5	1.5	1.5
Safety factor wave transmission	F68	γ_s	1.2	1.2	1.2
Safety factor stone size double layer	F69	γ_s	1.1	1.1	1.1
Safety factor stone size single layer	F70	γ_s	1.5	1.5	1.5
Water depth in front of toe to MSL	B82	h	7 m	7 m	7 m
Crest height to MSL	C82	R_c	10 m	10 m	10 m
Max. water level due to tide+surge to MSL	D82	SSL	3 m	4 m	4 m
Flow velocity (parallel) at toe of structure	E82	U	0 m/s	0 m/s	0 m/s
Significant wave height at toe of structure	F82	H_s	4 m	6 m	6 m
Wave period	G82	T_{mean} (T_{m-1})	10 s (11 s)	14 s (15 s)	14 s (15 s)
Wave angle at toe of structure	H82	β	0 deg. normal	0 deg. normal	0 deg. normal
Damage parameter	I82	S_d	2	2	4
Damage parameter	J82	N_{od}	0.5 (minor)	0.5 (minor)	1 (severe)
Number of waves	K82	N_w	1800	1800	1800
Computed values					
Ratio $H_s/H_{2\%}$ (van der Meer)	T82	γ_H	0.81	0.82	0.82
Surf similarity parameter	U82	ξ	3.1	3.6	3.6
Runup height	Y82,Z82	$R_{2\%}$	6.8 m	10.4 m	10.4 m
Wave overtopping rate	AI82	q_{ow}	1 l/m/s (<1%)	75 l/m/s (7%)	75 l/m/s (10%)
Transmitted wave height	AL82	$H_{s,Tr}$	0.48 m	1.08 m	1.08 m
Rock size front slope based on Van Gent	AY82	$D_{n,50}$	1.59 m	2.36 m	2.07 m
Critical surf similarity Van der Meer	BA82	ξ_{cr}	3.8	3.8	3.8
Rock size front slope based on Van der Meer	BG82	$D_{n,50}$	1.51m	2.37m	2.08 m
Rock size rear slope	CS82,CU82	$D_{n,50}$	0.8 m	1.28 m	1.14 m
Rock size first underlayer front slope	CX,CZ,DB82	$D_{n,50}$	0.8 m	1.18 m	1.03 m
Cubes randomly front slope in double layer	CJ82	$D_{n,50}$	1.56 m	2.25 m	1.95 m
Cubes orderly front slope single layer above LW	CD82	$D_{n,50}$	1.61 m	2.39 m	2.40 m
Tetrapods front in double layer	CO82	$D_{n,50}$	1.47 m	2.10 m	1.80 m

Table 3.3.6 *Rock and concrete armour sizes of high-crested breakwaters for storm events (ARMOUR.xls)*

3.3.3 Stability equations for high-crested berm breakwaters

The presence of a berm in the seaward slope of a rubblemound breakwater can be very effective to reduce wave overtopping.
A berm in the seaward slope can also increase the stability of the rock in the armour layer resulting in smaller rock sizes (or less damage for the same rock size).
Two types of berm breakwaters can be distinguished:

- non-reshaping berm breakwaters: berm is designed to be stable (non-reshaping berms);
- reshaping berm-breakwater: dynamically-stable berms; berm is initially unstable but will reshape during normal and more severe conditions into more stable gentle S-curved slopes which change/adjust to the various sea states; a flatter S-profile for smaller stones.

Typical features of a berm breakwater are:

- relatively high crest with less overtopping;
- relatively steep slopes between 1 to 1.5 and 1 to 2;
- mostly used in somewhat deeper water with depths (to MSL) between 8 and 10 m and relatively high wave heights between 3 and 7 m at the toe.

The stability of non-reshaping breakwaters with a berm has been studied by Van Gent (2013).
Based on experimental work in a wave flume, he has proposed various correction factors for the rock size of the upper slope above the berm and the lower slope below the berm.
Van Gent (2013) shows that Equation (3.3.7a) of Van der Meer (1988) can also be used for a breakwater with a berm just above the design water level. The permeability P-factor should be taken as P= 0.5 to 0.6.
Van Gent has proposed various correction factors ($1/\gamma_{pos}$ and $1/\gamma_{berm}$) to Equation (3.3.7a) of Vermeer (1988) for plunging breaking waves.
The correction factors depend on the following parameters:

- angle (α) of the seaward upper or lower slope;
- surf similarity parameter $\xi = \tan\alpha/[(2\pi/g)H_{s,toe}/T^2_{mean}]^{0.5}$ based on T_{mean} wave period;
- ratio of $h_b/H_{s,toe}$ with h_b= position of the berm with respect to the still water level (SWL); h_b>0 for berm above SWL, h_b< 0 for berm below SWL; $H_{s,toe}$= significant wave height at toe;
- ratio of $B/H_{s,toe}$ with B= berm width normal to the breakwater.

The conclusions of Van Gent (2013) are:

upper slope

- rock size in the upper slope can be significantly smaller than for a straight slope without a berm;
- rock stability increases and depends on the level of the berm, the width of the berm, the surf-similarity parameter;
- influence of a berm is more significant for a steep slope than for a gentle slope;

lower slope

- berm at SWL: no reduction in the damage level;
- berms below SWL: reduction in the damage level due to reduced wave run-down;
- influence of an emerged berm is more significant for a steep slope than for a gentle slope;

berm

- damage occurs at the seaward side of the berm;
- no reduction of rock size if berm is at SWL;
- reduction of rock size if berm is lower or higher than SWL depending on berm level and berm width;
- landward of the position at the berm where damage ends, a smaller rock size may be applied.

Andersen et al. (2012) have found that the stability of reshaping berm breakwaters can be best computed using the formula for plunging breaking waves only. The presence of the berm leads to plunging breaking waves rather than to surging breaking waves. The waves effectively feel a flatter slope than present.

Various formulae are available in the Literature to compute the recession at the edge of the berm or the new reshaped S-type profile of the armour layer (see also Van der Meer 1988, Van Gent 2013, Burcharth 2013 and Rock Manual 2007).
Tests in a wave bassin/flume are highly recommended for breakwaters with a berm.

3.3.4 Stability equations for low-crested, emerged breakwaters and groins

A breakwater has a low crest if $0 < R_c \leq 4\ D_{n,50}$ with R_c = crest height between crest and still water level ($R_c > 0$ for emerged breakwaters and $R_c < 0$ for submerged breakwaters), see **Figures 2.5.1** and **3.3.6**.

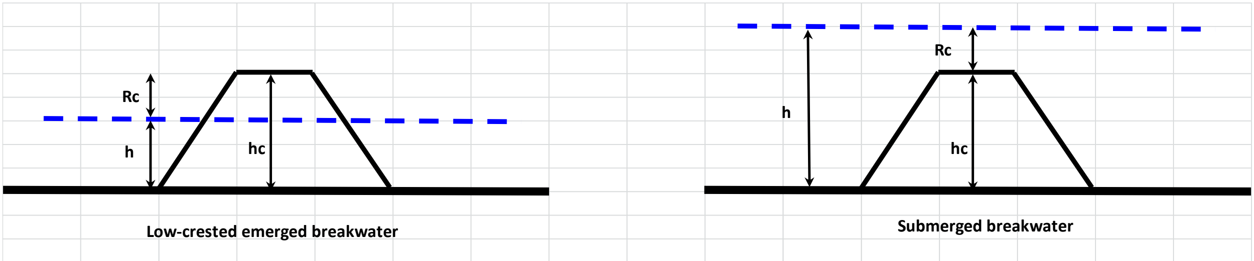


Figure 3.3.6 *Low-crested breakwaters*

The crest width is approximately 3 to $10D_{n,50}$; wide-crested breakwaters (having a width equal to 0.5 the local wave length) are known as reef-type breakwaters.

Typical features of low-crested, emerged breakwaters are:
- shore-parallel (breakwaters) and shore-connected structures (groins);
- relatively low crest above the design water level;
- significant wave overtopping; relatively mild slopes between 1 to 2 and 1 to 3;
- permeable underlayers and core;
- wave heights between 2 and 4 m at the toe;
- mostly used in the nearshore with depths (to MSL) up to 8 m.

Mostly, rock armour units are used:
- randomly placed rocks in two layers under water;
- orderly placed rocks in one or two layers above water.

3.3.4.1 Randomly placed rocks in two layers

Van der Meer (1990) and Van der Meer et al. (1996) have analysed scale model tests for rock-type low-crested breakwaters with slopes of 1 to 1.5 and 1 to 2. The effect of rock shape and grading (up to D_{85}/D_{15} = 2.5) was found to be small. Relatively flat and elongated rocks were as stable as more uniform rocks. Angular and round rocks had the same stability values. The measured N_{cr}-values (minor damage) of Van der Meer et al. (1996) are shown in **Figure 3.3.7A** and **Table 3.3.7**. **Figure 3.3.7A** is based on data with wave steepness in the range of 0.01 to 0.05; crest widths in the range of 5 to $8D_{n,50,front}$ and seaward front slopes of 1 to 1.5 and 1 to 2. The rock size of a toe/bed protection can also be expressed in terms of N_{cr} and $R_c/D_{n,50}$. Equation (3.3.18) of Van Gent and Van der Werf and Equation (3.2.10) of Van Rijn are shown in **Figure 3.3.7A**. The rocks of the front slope have an increasing stability for decreasing crest levels. The stability number of the front slope should have a smooth transition to the stability of the toe/bed protection in the case of a submerged structure.

The rocks of the crest zone and the rear slope have the lowest stability for crest levels ($R_c/D_{n,50,front}$) in the range of 0 to 3 when overtopping is relatively large. The stability of rocks of the crest and the rear slope increases significantly for relatively high crest levels and thus relatively small wave overtopping.
Based experimental results, Van der Meer (1990) and Van der Meer and Daemen (1994) have proposed a correction method to determine the stability of rock slopes of low-crested, emerged structures ($0 < R_c/D_{n,50,front} < 4$). The correction factor can be applied to the stability formula of Van der Meer 1988 (Equation 3.3.7) for rock slopes of high-crested structures, as follows:

$$N_{cr,lowcrested} = \gamma_{cor}\ N_{cr,front,\ highcrested} \qquad (3.3.11)$$
$$\gamma_{cor} = 1.25 - 4.8[R_c/H_{s,toe}][s/(2\pi)]^{0.5} \text{ and } \gamma_{cor} \geq 1 \qquad \text{for } 0 < R_c/D_{n,50,front} < 4$$

with: $N_{cr,front,highcrested}$ = stability number based on Equation (3.3.7), R_c = crest height ($R_c > 0$), $s = H_{s,toe}/L_o$= wave steepness, L_o = wave length deep water. The P_M-value is about 0.4.
Equation (3.3.11) yields about $\gamma_{cor} = 1$ for $R_c \geq H_{s,toe}$ and about $\gamma_{cor} = 1.25$ for $R_c= 0$. Thus, the correction factor of Equation (3.3.11) is in the range of 1 to 1.25 for $0 < R_c/D_{n,50,front} < 4$. Equation (3.3.11) is NOT valid for submerged breakwaters ($R_c < 0$).
The correction factor can also be applied to the stability formula of Van Gent et al. (2003) for rock slopes of high-crested structures.
The stability number N_{cr} of low-crested structures, as shown in **Figure 3.3.7A** can be expressed as a correction to the stability number of the front slope of a high-crested structure, as follows:
$N_{cr,lowcrested} = \gamma_{cor}\ N_{cr,front,highcrested}$. The correction factor γ_{cor} can be derived from the data of **Figure 3.3.7A**.
Figure 3.3.7B shows the dimensionless correction factor $\gamma_{cor} = N_{cr}/N_{cr,front,highcrested}$ with $N_{cr,front,highcrested}$=1.3 (based on the data of **Figure 3.3.7A**) for $R_c/D_{n,50} \geq 4$. Using this approach, the stability number of a low-crested (submerged or emerged) breakwater can be computed as: $N_{cr} = \gamma_{cor}\ N_{cr,front,highcrested}$ with γ_{cor} = correction factor based on **Figure 3.3.7B** for the front slope, the crest zone and the rear slope.

Relative crest level above water level $R_c/D_{n,50,front}$	**Front armour slope $N_{cr,design}$** Minor damage (S_d=1)	**Crest armour $N_{cr,design}$** Minor damage (S_d=0.5)	**Rear armour slope $N_{cr,design}$** Minor damage (S_d=0.5
0	$1.6/(\gamma_s\,\gamma_{Beta})$	$1.5/(\gamma_s\,\gamma_{Beta})$	$2.2/(\gamma_s\,\gamma_{Beta})$
1	$1.5/(\gamma_s\,\gamma_{Beta})$	$1.4/(\gamma_s\,\gamma_{Beta})$	$2.0/(\gamma_s\,\gamma_{Beta})$
2	$1.4/(\gamma_s\,\gamma_{Beta})$	$1.5/(\gamma_s\,\gamma_{Beta})$	$1.9/(\gamma_s\,\gamma_{Beta})$
3	$1.35/(\gamma_s\,\gamma_{Beta})$	$1.9/(\gamma_s\,\gamma_{Beta})$	$1.9/(\gamma_s\,\gamma_{Beta})$
4	$1.3/(\gamma_s\,\gamma_{Beta})$	$2.6/(\gamma_s\,\gamma_{Beta})$	$2.7/(\gamma_s\,\gamma_{Beta})$

R_c= crest height above still water level; γ_s= safety factor
Table 3.3.7 *Stability of randomly-placed rocks for low-crested, emerged breakwaters*

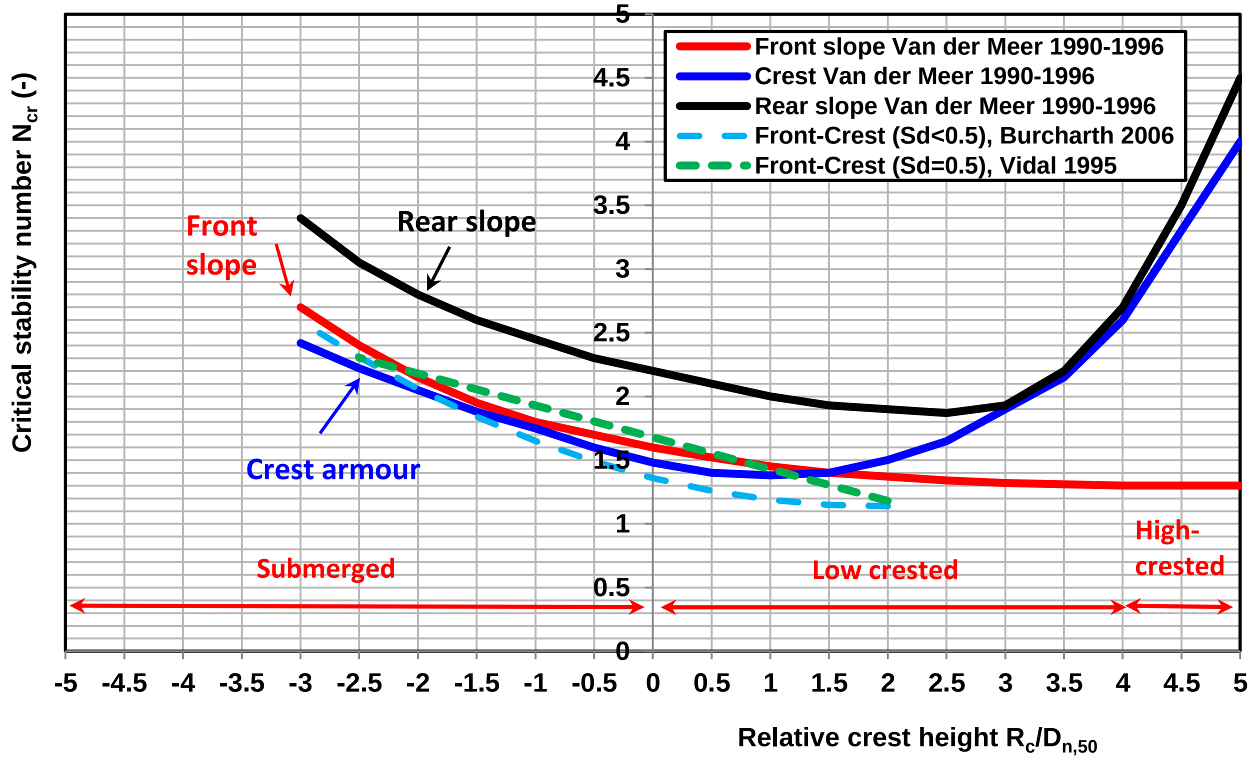


Figure 3.3.7A *Stability (minor damage) of low crested, emerged and submerged rock breakwaters*

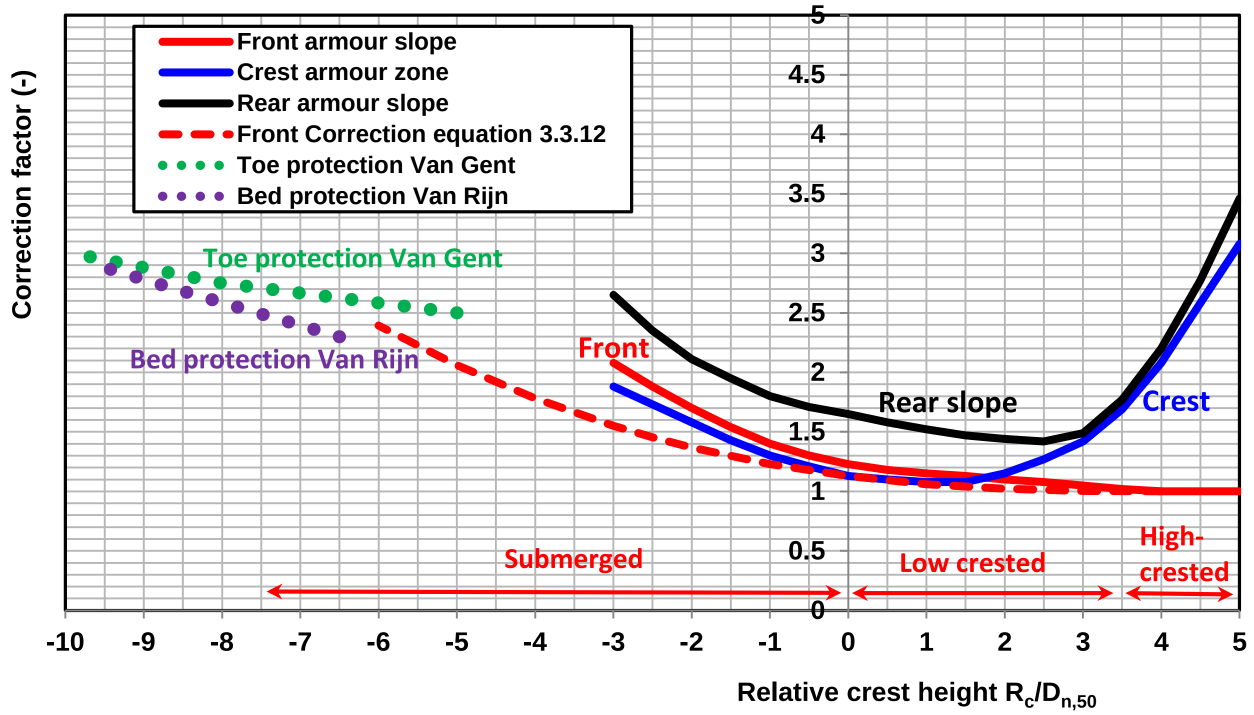


Figure 3.3.7B *Correction factor for stability of low-crested, emerged and submerged rock breakwaters*

The correction factors of **Figure 3.3.7B** can be roughly represented as:

Front slope: $N_{cr,lowcrested} = \gamma_{cor}\, N_{cr,\,front\,highcrested}$ (3.3.12)
$D_{n,50,\,lowcrested} = (1/\gamma_{cor})\, D_{n,50,front,\,highcrested}$
$\gamma_{cor} = 1$ for $R_c/D_{n,50,front} > 4$
$\gamma_{cor} = 0.0035\,(|R_c/D_{n,50,front} - 4|)^{2.6} + 1$ for $-8 < R_c/D_{n,50,front} < 4$
$\gamma_{cor} = 3$ for $R_c/D_{n,50,front} < -8$

Crest zone $D_{n,50,crest} = (1/\gamma_{cor})\, D_{n,50,front,\,highcrested}$ (3.3.13)
$\gamma_{cor} = 0.5(R_c/D_{n,50,front})$ for $R_c/D_{n,50,front} > 2$
$\gamma_{cor} = 0.0035\,(|R_c/D_{n,50,front} - 4|)^{2.6} + 1$ for $-8 < R_c/D_{n,50,front} < 2$
$\gamma_{cor} = 3$ for $R_c/D_{n,50,front} < -8$

Rear slope: $D_{n,50,rear} = (1/\gamma_{cor})\, D_{n,50,front,highcrested}$ (3.3.14)
$\gamma_{cor} = 0.5(R_c/D_{n,50,front})$ for $R_c/D_{n,50,front} > 3$
$\gamma_{cor} = 1.5$ for $0 < R_c/D_{n,50,front} < 3$
$\gamma_{cor} = -0.4(R_c/D_{n,50,front}) + 1.5$ for $-3 < R_c/D_{n,50,front} < 0$
$\gamma_{cor} = 3$ for $R_c/D_{n,50,front} < -3$

with: $N_{cr,front,highcrested}$ = stability number based on Equation (3.3.6 or 3.3.7) for rocks or Equation (3.3.9) for cubes, R_c = crest height (positive or negative).
Equation (3.3.12) is based on the absolute value of the parameter $|R_c/D_{n,50,front} - 4|$ and yields a smooth transition to a bed protection for submerged breakwaters, see **Figure 3.3.7B**.
The computation of $D_{n,50,front}$ requires an iteration procedure as its value is a priori unknown. Given the accuracies involved, two iterations generally are sufficient, taking $D_{n,50,front,\,highcrested}$ as the start value.
A safety factor γ_s =1.2 to 1.3 should be used for deterministic design of randomly-placed rocks in a double layer for low-crested breakwaters. The uncertainty is somewhat larger as that for high-crested breakwaters.
Equation (3.3.12) yields a size reduction of about 10% for $R_c/D_{n,50} = 0$ and about 50% for $R_c/D_{n,50} = -4$ with respect to a high-crested structure ($R_c/D_{n,50} \geq 4$)
In nearshore breaking wave conditions with $H_{s,toe} = \gamma_{br}$ h and γ_{br}= 0.6 to 0.8 and $N_{cr,design} \cong 1.4$, it follows that: $D_{n,50,front} \cong 0.27$ to 0.35 h for low-crested, emerged breakwaters.
Vidal et al. (1995) and Burcharth et al. (2006) have carried carried out laboratory tests of low-crested (emerged and submerged) breakwaters with crest widths in the range between $3D_{n,50,front}$ and $8D_{n,50,front}$ and slopes in the range between 1 to 1.5 and 1 to 2. The stability numbers (minor damage) of the trunk section are in the range of 1.2 and 2 and those of the roundheads are in the range of 1.4 to 2.0. The stability decreases slightly with increasing wave steepness. The crest width has no effect on stability.
Based on their data, the N_{cr}-value of the front and crest of the trunk and roundhead of low-crested (submerged and emerged) breakwaters is proposed (by the present author) to be described by the following expression (see **Figure 3.3.7A**):

$N_{cr} = [1/(\gamma_s\,\gamma_{Beta})]\,[-0.25(R_c/D_{n,50,front}) + 1.8\,(S_d)^{0.1}]$ for $-3 < R_c/D_{n,50,front} < 2$ and $0.5 < S_d < 2$ (3.3.15a)
$D_{n,50} = [0.35\,(\gamma_s\,\gamma_{Beta})H_{s,toe} + 0.14\,R_c]\,(S_d)^{-0.1}$ for $-3 < R_c/D_{n,50,front} < 2$ and $0.5 < S_d < 2$ (3.3.15b)

with: S_d= damage (S_d= 0.5 = start of damage and S_d = 2 = minor damage).
Equation (3.3.15a) requires iterative equations, as the value of $D_{n,50,front}$ is a priori unknown. If $R_c/D_{n,50,front} < -3$, the N_{cr}-value (outside the validity range of Equation (3.3.15a) should be kept constant.
Burcharth et al. have reanalyzed all available data and proposed as underenvelope to all data (**Figure 3.3.7A**):

$N_{cr} = [1/(\gamma_s\,\gamma_{Beta})]\,[0.06\,(R_c/D_{n,50,front})^2 - 0.23(R_c/D_{n,50,front}) + 1.36]$ for $-3 < R_c/D_{n,50,front} < 2$ (3.3.15c)

3.3.4.2 Randomly placed concrete units in double or single layer

Stability numbers are given in **Table 3.3.3**.

Equations (3.3.9) and (3.3.10) in combination with the correction factor of Equation (3.3.12 to 3.3.14) for low crests can be used for cubes and tetrapods in a double layer.

3.3.5 Stability equations for submerged breakwaters

Typical features are:

- relatively low crest below the design water level;
- crest width between $3D_{n,50,front}$ and $10D_{n,50,front}$;
- relatively steep slopes of 1 to 1.5 in non-breaking wave conditions (deeper water);
- relatively mild slopes between 1 to 2 and 1 to 3 (in nearshore breaking wave conditions);
- all waves are overtopping;
- permeable underlayers and core;
- mostly used in the nearshore with depths (to MSL) up to 8 m.

Rock or concrete armour units can be used:

- randomly placed rocks in two layers;
- randomly placed cubes in two layers.

In nearshore waters it is common practice to use the same armour size for the whole structure, whereas in deeper water it may be more economic to use different armour sizes for the seaward slope, the crest zone and the rear slope.

3.3.5.1 Randomly placed rocks in double layer

Van der Meer et al. (1996) have carried out scale model tests for submerged breakwaters.
The N_{cr}-values based on their results are shown in **Figure 3.3.7A** and **Table 3.3.8**.
The N_{cr}-values of the front slope are largest for a relative freeboard $R_c/D_{n,50,front}$= -3.
The lower limit of the the data is about $R_c/D_{n,50,front} \cong$ -4.

Relative crest level below water level $R_c/D_{n,50}$	**Front slope** $N_{cr,design}$ (minor damage)	**Crest** $N_{cr,design}$ (minor damage)	**Rear slope** $N_{cr,design}$ (minor damage)
-3	$2.5/(\gamma_s \gamma_{Beta})$	$2.4/(\gamma_s \gamma_{Beta})$	$3.3/(\gamma_s \gamma_{Beta})$
-2	$2.2/(\gamma_s \gamma_{Beta})$	$2.0/(\gamma_s \gamma_{Beta})$	$2.7/(\gamma_s \gamma_{Beta})$
-1	$1.8/(\gamma_s \gamma_{Beta})$	$1.7/(\gamma_s \gamma_{Beta})$	$2.4/(\gamma_s \gamma_{Beta})$
0 (crest at SWL)	$1.6/(\gamma_s \gamma_{Beta})$	$1.5/(\gamma_s \gamma_{Beta})$	$2.2/(\gamma_s \gamma_{Beta})$

R_c= crest height above still water level; γ_s= safety factor
Table 3.3.8 *Stability of rocks for low-crested, submerged breakwaters*

The formulae of Van Gent et al. 2003 (Equation 3.3.6) and Van der Meer 1988 (Equation 3.3.7) can also be used for submerged breakwaters in combination with a correction factor (Equations 3.3.12 to 3.3.14) with R_c = (negative) crest height.
The armour size of the crest may be slightly larger than that of the front slope for submerged conditions.

Equations (3.3.12 to 3.3.14) can be tentatively applied to both rock and concrete (cubes) units.
A safety factor γ_s = 1.2 to 1.3 should be used for deterministic design (double layer of rocks) of submerged breakwaters.

Equations (3.3.15a,b,c) are also valid for submerged breakers.
The incoming design waves will be breaking In the case of submerged breakwaters in shallow water.

A relatively simple expression can be derived (Burcharth et al. 2006):

$D_{n,50,front} \cong 0.3\, h_{crest}$ with h_{crest} = height of crest above the bottom

A double layer of rocks with $D_{n,50} > 0.5\, h_c$ (see **Figure 3.3.6**) in nearshore shallow water requires that part of the structure should be placed below the bed level, which requires the dredging of a trench. The $D_{n,50}$ can be reduced by using a milder slope than 1 to 2 in the surf zone. A conventional structure with core and filter layers above the bed requires $D_{n,50} < 0.2\, h_c$. In most cases this is not feasible in shallow water, see also Burcharth et al. (2006).

3.3.5.2 Concrete armour units

Randomly placed cubes in double layer
Stability numbers are given in **Table 3.3.3**.

Equation (3.3.9) can be used for cubes in a double layer.

The correction Equation (3.3.12 to 3.3.14) can also be used for concrete cubes.

Concrete interlocking units in single layer
Muttray et al. (2012) have tested a ***single*** layer of interlocking Xblocs on the slope and crest of low-crested, emerged and submerged breakwaters.
The lower envelope of their basic data (start of damage) can be represented as:

$$N_{cr} = 3.5/(\gamma_s\, \gamma_{Beta}) \quad \text{for} \quad R_c/H_{s,toe} > 1 \qquad (3.3.16a)$$
$$N_{cr} = 3.0/(\gamma_s\, \gamma_{Beta}) \quad \text{for} \quad -0.5 < R_c/H_{s,toe} < 1 \qquad (3.3.16b)$$
$$N_{cr} = 3.5/(\gamma_s\, \gamma_{Beta}) \quad \text{for} \quad R_c/H_{s,toe} < -0.5 \qquad (3.3.16c)$$

with: γ_s = safety factor= 1.3 to 1.5.

Xblocs have relatively low stability for $-0.5 < R_c/H_{s,toe} < 1$ due to the gap-effect at the transition from slope to horizontal crest. This behaviour is opposite to that of rocks, which show an increasing stability for decreasing crest height. Interlocking units under water require special care during placement (divers) to ensure sufficient interlocking.

3.3.5.3 Example case 1: Low-crested breakwater in shallow water

Two water levels are considered: high water level (high tide) and low water level (low tide).

Emerged case: $H_{s,toe}$= 3 m; crest level= 0 m above mean sea level (MSL); Tide level= -2 m below MSL.
Submerged case: $H_{s,toe}$= 4 m; crest level= 0 m above mean sea level (MSL); Tide level= +2 m above MSL.

The return period = 25 years; the storm duration= 4 hours.
The waves generally are higher during high tide (larger water depth).
The input data and results based on the tool **ARMOUR.xls** (Sheet 2) are given in **Table 3.1.1**.
Water depth (to MSL) in front of structure= 8 m to MSL.
Crest width of armour= 5 m; total crest width= 15 m; no berm
The significant wave heigth at the toe of the structure is given. In most cases, only the offshore wave height is known. The tool WAVEMODELS.xls can be used to compute the wave height at the toe of the structure.

The results of **Table 3.3.9** show that the armour size is slightly larger for the submerged case.

The maximum size of randomly placed rocks in a double layer is about 1.15 to 1.2 m.

Orderly placed rocks in a single layer (above water) have a size of about 1.4 m.
If the breakwater is emerged during low tide, the rock units above the low water level can be placed orderly which increases the stability and gives a more aesthetical view.

Randomly placed cubes in a double layer have a (maximum) size of about 1.15 m.

Orderly placed cubes in a single layer also have a (maximum) size of about 1.2 m.

Parameters			Storm event Emerged case	Storm event Submerged
Input values:	***Column***			
Density of seawater	F47	ρ_w	1025 kg/m^3	1025 kg/m^3
Density of rock	F48	ρ_{rock}	2650 kg/m^3	2650 kg/m^3
Density of concrete	F49	$\rho_{concrete}$	2300 kg/m^3	2300 kg/m^3
Kinematic viscosity coefficient	F50	ν	0.000001 m^2/s	0.000001 m^2/s
Crest width armour	F52	B_c	5 m	5 m
Total crest width	F53	B_t	15 m	15 m
Berm	F54	-	0 (none)	0 (none)
Slope angle front	F55	tan(α)	0.5	0.5
Slope angle rear	F56	tan(α)	0.5	0.5
Rougness front slope	F58	γ_r	0.45	0.45
Permeability factor Van Gent	F60	P_G	0.3	0.3
Permeability factor Van der Meer	F62	P_M	0.4	0.4
Critical stability number concrete units	F64	N_{cr}	3	3
Safety factor runup	F66	γ_s	1.2	1.2
Safety factor wave overtopping	F67	γ_s	1.5	1.5
Safety factor wave transmission	F68	γ_s	1.2	1.2
Safety factor stone size double layer	F69	γ_s	1.1	1.1
Safety factor stone size single layer	F70	γ_s	1.5	1.5
Water depth in front of toe to MSL	B82	h	8 m	8 m
Crest height to MSL	C82	R_c	0 m	0 m
Maximum water level due to tide+surge to MSL	D82	SSL	-2 m	+2 m
Flow velocity (parallel) at toe of structure	E82	U	0 m/s	0 m/s
Significant wave height at toe of structure	F82	H_s	3 m	4 m
Wave period	G82	T_{mean} (T_{m-1})	8 s (9 s)	10 s (11 s)
Wave angle at toe of structure	H82	β	0 degrees normal to structure	0 degrees normal to structure
Damage parameter	I82	S_d	2	2
Damage parameter	J82	N_{od}	0.5	0.5
Number of waves	K82	N_w	1800	1800
Computed values				
Ratio $H_s/H_{2\%}$ (van der Meer)	T82	γ_H	0.73	0.78
Surf similarity parameter	U82	ξ	2.9	3.1
Runup height	Y82,Z82	$R_{2\%}$	5.05 m	-
Wave overtopping rate	AI82	q_{ow}	73 l/m/s (21%)	5000 l/m/s (100%)
Transmitted wave height	AL82	$H_{s,Tr}$	0.9 m	2.9 m
Rock size front slope based on Van Gent	AY82	$D_{n,50}$	1.16 m	1.21 m
Critical surf similarity Van der Meer	BA82	ξ_{cr}	3.8	3.8
Rock size front slope based on Van der Meer	BG82	$D_{n,50}$	1.17 m	1.19 m
Rock size orderly placed single front above LW	BN82	$D_{n,50}$	1.34 m	1.45 m
Rock size orderly placed double front above LW	BT82	$D_{n,50}$	0.98 m	1.07 m
Rock size rear slope	CS82,CU82	$D_{n,50}$	0.75 m	0.79 m
Rock size first underlayer front slope	CX,CZ,DB82	$D_{n,50}$	0.58 m	0.63 m
Cubes randomly front slope in double layer	CJ82	$D_{n,50}$	1.15 m	1.18 m
Cubes orderly front slope single layer above LW	CD82	$D_{n,50}$	1.17 m	1.22 m

Table 3.3.9 *Rock and concrete armour sizes of low-crested breakwaters for storm events*

3.3.5.4 Example case 2: Low-crested breakwater in shallow water

The crest height above bottom is 5 m (bottom at 4 m below MSL). The crest level is at 1m above MSL.
Seven cases with varying water levels (in the range of -1.5 to +4.5 m) have been considered.
The water depth varies between 2.5 m and 8.5 m.
As a result of the increasing water level, the crest height (to the still water level) decreases.
The lowest water level (-1.5 m) yields an emerged breakwater and the highest water level (+4.5 m) yields a submerged breakwater. The significant wave height at the toe is $H_{s,toe}$= 0.6 h_{toe} resulting in values between 1.5 m and 5.1 m. The data are given in **Table 3.3.10**.

The results based on the spreadsheet-model **ARMOUR.xls** are shown in **Figure 3.3.8**. The armour size ($D_{n,50}$) of the front slope and the crest increases with increasing wave height and increasing water level. Equation (3.3.6) of Van Gent et al. (2003) and Equation (3.3.7) of Van der Meer (1988) have been applied in combination with a correction factor (Equation (3.3.12) to account for the varying values of $R_c/D_{n,50}$. The computed rock sizes are in the range of 0.6 to 1.7 m.The results of the method of Van Gent et al. (2003) without correction factor are also shown, yielding values in the range of 0.6 to 2.1 m. The correction factor yields a size reduction of about 35% for the most submerged case with the largest water depth.
A size reduction (10% to 20%) can be obtained by using orderly placed rocks (in a double layer above low water level) instead of randomly placed rocks (in a double layer). The armour size of rocks orderly placed in a single layer is largest, because of the use of a high safety factor of 1.5.

Parameters		Values
Maximum water level incl. tide to MSL	SSL	-1.5, -0.5, 0.5, 1.5, 2.5, 3.5, 4.5 m
Significant wave height	H_s	1.5, 2.1, 2.7, 3.3, 3.9, 4.5, 5.1 m
Water depth in front of toe to MSL	h	2.5, 3.5, 4.5, 5.5, 6.5, 7.5, 8.5 m
Wave period	T_{mean}	5, 6, 7, 8, 9, 10, 11 s
Density of rock	ρ_{rock}	2650 kg/m^3
Density of concrete	$\rho_{concrete}$	2300 kg/m^3
Density of seawater	ρ_w	1025 kg/m^3
Number of waves	N_w	2500
Permeability factor Van der Meer	P_M	0.4
Permeability factor Van Gent	P_G	0.3
Damage	S_d; N_{od}	2; 0.5
Crest height above MSL	R_c	+1 m
Total crest width	B_t	15 m
Berm	-	none
Slope angle front	tan(α)	0.5
Slope angle rear	tan(α)	0.5
Rougness front slope	γ_r	0.45
Safety factor runup	γ_s	1.2
Safety factor wave overtopping	γ_s	1.5
Safety factor wave transmission	γ_s	1.2
Safety factor stone size double layer	γ_s	1.1
Safety factor stone size single layer	γ_s	1.5
Wave angle at structure	β	90 degrees

Table 3.3.10 *Armour sizes of low-crested submerged and emerged breakwaters*

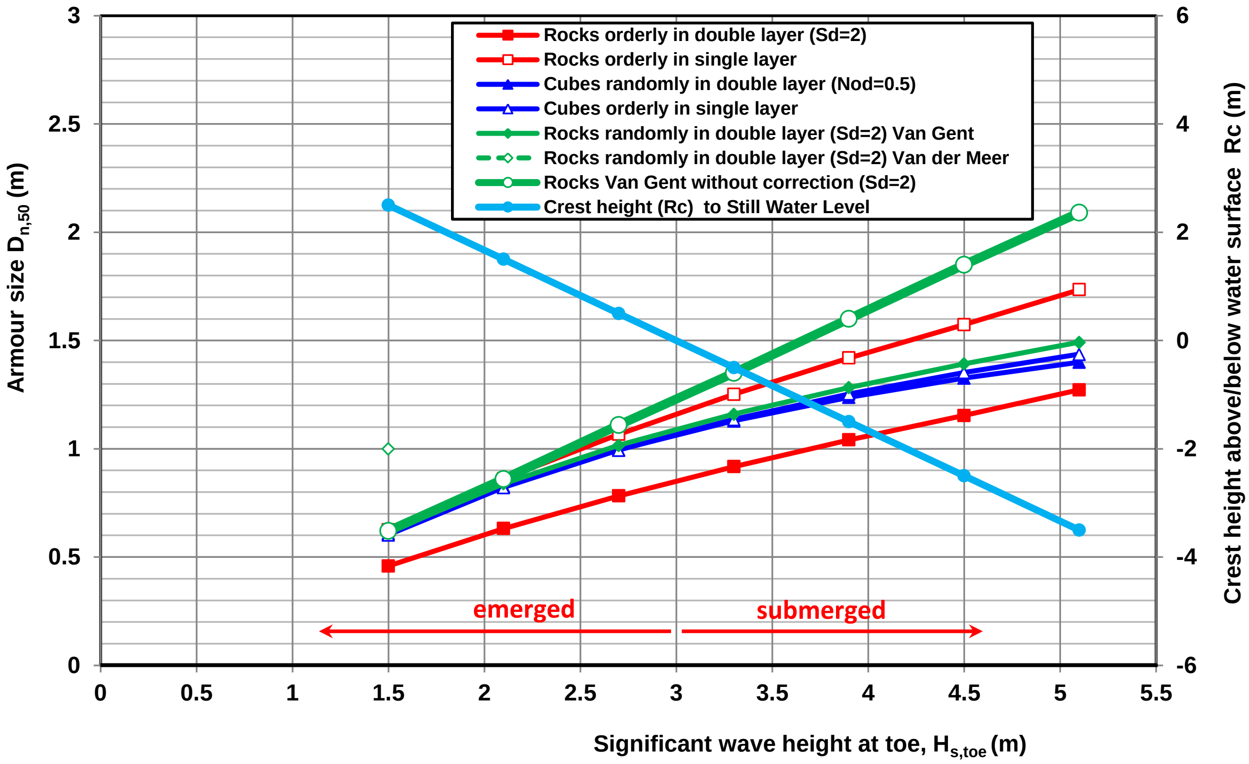


Figure 3.3.8 *Armour size as function of wave height for low-crested emerged and submerged breakwaters*

3.3.5.5 Example case 3: Low-crested breakwater in shallow water.

The water depth is constant at h= 6 m and the significant wave height at the toe also is constant at $H_{s,toe}$= 3.6 m. The mean wave period is T_p= 10 s. Ten cases with varying crest heights (in the range of -5 to +6 m) have been considered. The lowest crest level (-5 m) yields a submerged breakwater and the highest crest level (+6 m) yields an emerged breakwater. The data are given in **Table 3.3.11**.

The results based on the spreadsheet-model **ARMOUR.xls** (sheet 2) are shown in **Figure 3.3.9**. The armour size ($D_{n,50}$) of the front slope and the crest (minor damage S_d = 2) increases with increasing crest height. Equation (3.3.6) of Van Gent et al. (2003) and Equation (3.3.7) of Van der Meer (1988) have been applied in combination with a correction factor (Equation (3.3.12)) to account for the varying values of $R_c/D_{n,50}$. The computed rock sizes are in the range of 0.5 to 1.5 m. The emerged breakwater cases have the largest armour sizes.

The results of the method of Van Gent et al. (2003) without correction factor are also shown, yielding a constant rock size of $D_{n,50}$ = 1.48 m for S_d = 2 and $D_{n,50}$ = 1.95 m (30% larger) for S_d = 0.5. The correction factor yields a size reduction of about 60% for the submerged case with the lowest crest. The correction factor is 1 (no reduction) for R_c = 6 m ($R_c/D_{n,50} \geq 4$).

The rock sizes according to Equation (3.3.15a) based on the data of Vidal et al. (1995) and Burcharth et al. (2006) are shown for S_d = 0.5 (start of damage) and S_d = 2 (minor damage). The results for S_d = 2 are in good agreement with those of Van Gent et al. 2003 (Equation 3.3.6) in combination with the correction factor of Van Rijn (Equation 3.3.12). Equation (3.3.15c) given by Burcharth et al. (2006) yields relatively large rock sizes for crests higher than -2 m, which is caused by the fact that this expression is the underenvelope of all available data (almost no damage), whereas the other expressions are trendlines through the data points (see Van der Meer et al., 1996).

The rock sizes of a toe protection layer (γ_s = 1.1) at -5 m and -5.5 m below the water level (water depth of 5 m and 5.5 m above the toe) are also shown in **Figure 3.3.9**. The rock size of a submerged breakwater with a crest level at -5 m is slightly smaller than that of a toe protection layer at -5 m of a high-crested breakwater. The rock size of the toe protection at -5 m is expected to be somewhat larger as it experiences both the incoming wave and the downrush of breaking waves.

Parameters		Values
Maximum water level incl. tide to MSL	SSL	0 m
Significant wave height and period	H_s; T_{mean}	3.6 m; 10 s
Water depth in front of toe to MSL	h	6 m
Density of rock	ρ_{rock}	2650 kg/m^3
Density of concrete	$\rho_{concrete}$	2300 kg/m^3
Density of seawater	ρ_w	1025 kg/m^3
Number of waves	N_w	2500
Permeability factor Van der Meer	P_M	0.4
Permeability factor Van Gent	P_G	0.3
Damage	S_d; N_{od}	2; 0.5 (**minor damage**)
Crest height above MSL	R_c	-5, -4, -3, -2, -1, 0, 1, 2, 3, 6 m
Total crest width	B_t	15 m
Slope angle front and rear	$\tan(\alpha)$	0.5
Rougness front slope	γ_r	0.45
Safety factor runup	γ_s	1.2
Safety factor wave overtopping	γ_s	1.5
Safety factor wave transmission	γ_s	1.2
Safety factor stone size double layer	γ_s	1.1
Safety factor stone size single layer	γ_s	1.5
Wave angle at structure	β	90 degrees

Table 3.3.11 *Armour sizes of low-crested submerged and emerged breakwaters*

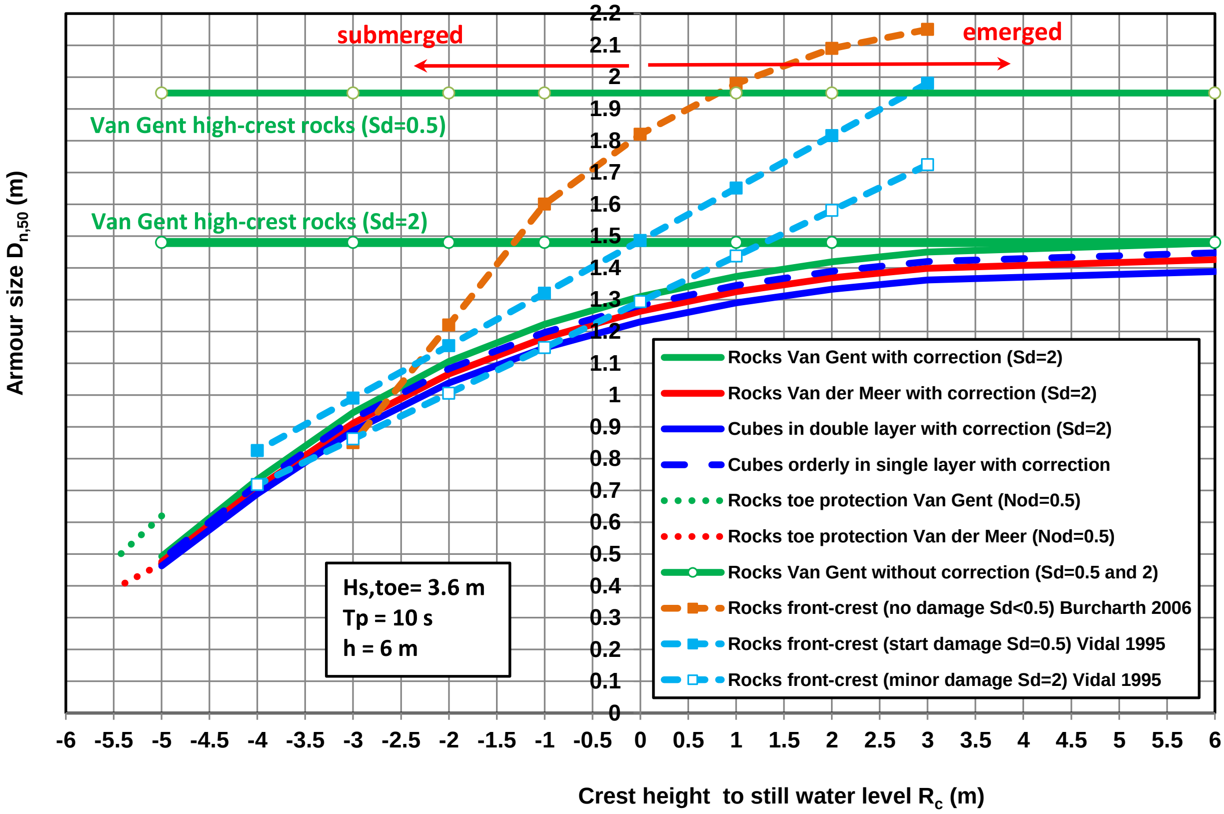


Figure 3.3.9 *Armour size as function of crest height for low-crested emerged and submerged breakwaters*

3.3.6 Stability equations for toe protection of breakwaters

Typical features of toe protections are:

- almost horizontal armour layer (randomly placed rocks/stones under water);
- no underlayers (armour is placed on geotextile).

The toe structure of a breakwater provides support to the armour layer slope and protects the structure against damage due to scour at the toe. Most often, the toe consists of randomly placed rocks/stones. Usually, the width of the toe varies in the range of 3 to 10 $D_{n,50}$ and the thickness of the toe varies in the range of 2 to 5 $D_{n,50}$ depending on the conditions, see **Figure 3.3.10**. The maximum toe thickness used is of the order of 2 to 2.5 m. (De Meerleer et al., 2013). The toe needs to be wider (about $3H_{s,toe}$) and thicker in strong scouring conditions. The toe protection should be designed such that almost no damage occurs. Damage will on the long term lead to undermining of the stucture due to scouring processes.
If the rock/stone size of the toe is the same as the armour slope, then the toe generally is stable, but this is not a very economic solution. In deeper water the rocks/stone size can be reduced as the wave forces are smaller.
A small ratio of h_{toe}/h in the range of 0.3 to 0.5 means that the toe is relatively high above the bed in shallow water. The toe may then be seen as a berm. In shallow water ($h_{toe}/h < 0.4$) the slope of the foreland also is important as it determines the type of breaking (Baart et al., 2010).

Some N_{cr}-values based on flume tests (h= water depth in front of toe, h_{toe}= water depth above toe), are:

$N_{cr}= 3.3$ for $h_{toe}/h = 0.5$
$N_{cr}= 4.5$ for $h_{toe}/h = 0.6$
$N_{cr}= 5.5$ for $h_{toe}/h = 0.7$
$N_{cr}= 6.5$ for $h_{toe}/h = 0.8$

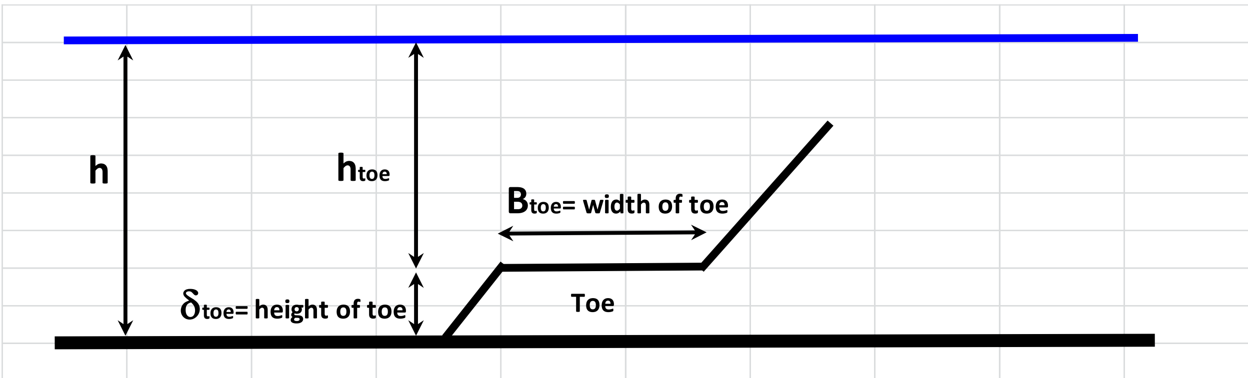


Figure 3.3.10 *Toe dimensions*

Based on laboratory tests in a wave flume, **Van der Meer (1998)** has proposed:

$$D_{n,50} = \gamma_s \, [6.2(h_{toe}/h)^{2.7} + 2]^{-1} \, \Delta^{-1} \, N_{od}^{-0.15} \, H_{s,toe} \quad \text{for } 0.4 < h_{toe}/h < 0.9 \qquad (3.3.17)$$

with:
N_{od} = 0.5 to 1 = start of damage; N_{od} = 2 = severe damage and N_{od} = 4 = failure;
γ_s = safety factor (=1.5).

Based on many laboratory tests in a wave flume (non-overtopped rock slope of 1 to 2; permeable core; foreland of 1 to 30; no severe wave breaking at foreland), Van Gent and Van der Werf (2014) have proposed the following formula (N_{cr}-values in the range of 2 to 6):

$$D_{n,50}= (0.32\ \gamma_s)\ [H_{s,toe}/(\Delta\ N_{od}^{0.33})]\ (B_{toe}/H_{s,toe})^{0.1}\ (\delta_{toe}/H_{s,toe})^{0.33}\ [U_{max}/(g\ H_{s,toe})^{0.5}]^{0.33} \quad (3.3.18)$$

with:

B_{toe} = width of toe; δ_{toe} = height of toe;
N_{od} = damage (N_{od} = 0.5 for small toe width and N_{od} = 1 for large toe width);
U_{max} = $\pi\ H_{s,toe}/(T_{m-1,0} \sinh(kh_{toe}/L_o))$ = peak orbital velocity at toe based on deep water wave length;
K = $2\pi/L$ = wave number; L_o = wave length at deep water = $(g/2\pi)\ (T_{m-1,0})^2$;
h_{toe} = water depth above toe; h= water depth in front of toe;
γ_s = safety factor (= 1.1 for double layer; 1.5 for single layer), $\Delta=(\rho_s - \rho_w)/\rho_w$.

Equation (3.3.18) is valid for h_{toe}/h = 0.7 to 0.9 or δ_{toe}/h = 0.1 to 0.3. The peak orbital velocity (U_{max}) is based on the deep water wave length (L_o) which leads to relatively large U_{max}-values in shallow water and hence relatively large $D_{n,50}$-values for shallow depths.

Baart et al. (2010) have studied the stability of toe protections in very shallow water on a sloping bottom (foreland). The N_{cr}-value is related to the surf similarity parameter and decreases with increasing ξ-value. The formula reads, as:

$$N_{cr} = (3/\gamma_s)\ \xi^{-0.5}\ (N_{\%})^{0.33} \qquad \text{for } 0.3 < \xi < 0.9 \text{ and } h_{toe}/h < 0.4 \quad (3.3.19)$$

with:

ξ = surf similarity parameter = $\tan(\alpha_{bottom})/s^{0.5}$; minimum value of ξ = 0.3 for relatively flat slopes;
S = $H_{s.toe}/L_o$= wave steepness;
$\alpha_{foreland}$ = slope angle of foreland in shallow water (between 1 to 10 and 1 to 50);
L_o = wave length in deep water $((g/(2\pi))\ T_p^2)$;
$N_{\%}$ = 100 n $(D_{n,50})^3/((1-p)\ V_T)$ = damage as a percentage of the total volume of stones per unit length of the structure ($N_{\%}$ = 5 should be used as start of damage);
V_T = total volume of stones per unit length of structure;
N = number of stones displaced per unit length of structure;
p = porosity factor; $\Delta=(\rho_s - \rho_w)/\rho_w$;
γ_s = safety factor (= 1.3 to 1.5); should be relatively large to prevent failure at the toe.

Muttray (2013) has proposed:

$$N_{cr}= (2.4/\gamma_s)\ (N_{od})^{0.33}\ [1.4 - 0.4h_{toe}/H_{s,toe}]^{-1} \quad (3.3.20)$$

$$D_{n,50}= \gamma_s\ [(1.4 - 0.4h_{toe}/H_{s,toe})\ H_{s,toe}]/(2.4\ \Delta\ N_{od}^{0.33})$$

with:

N_{cr} = $H_{s,toe}/(\Delta D_{n,50})$;
N_{od} = damage (N_{od} = 0.5 for small toe width and N_{od} = 1 for large toe width);
h_{toe} = water depth at toe;
$H_{s,toe}$ = significant wave height at toe.

Equation (3.2.11) of **Van Rijn** can be used for rock toe stability in a regime with waves plus currents. The effect of the structure on the velocity field can be taken into account by the γ_{str}-coefficient. A reasonable estimate is γ_{str}=1.2 to 1.3.

Parameters			Case 1	Case 2
Input values	***Columns***			
Density of seawater	E21	ρ_w	1030 kg/m^3	1030 kg/m^3
Density of rock	E22	ρ_{rock}	2700 kg/m^3	2700 kg/m^3
Kinematic viscosity coefficient	E23	ν	0.000001 m^2/s	0.000001 m^2/s
Thickness of bed protection layer	G82	δ	1 m	1 m
Length of bed protection normal to waves	B82	B	3 m	3 m
Critical Shields parameter	E28/E29	θ_{cr}	0.02	0.02
Tan of longitudinal bed slope	E30	tan (α_1)	0.04	0.04
Tan of lateral (side) side bed slope	E33	tan (α_1)	0	0
Tan of angle of repose	E35	tan (α_{repose})	0.3	0.3
Velocity enhancement coefficient	E27	γ_{str}	1.2	1.2
Safety factor	E37	γ_{safety}	1.5	1.5
Maximum water level incl. tide and surge to MSL	C46	SSL	1 m	1 m
Water depth at toe to MSL	B46	h_{toe}	6 m	6 m
Significant wave height	E46	$H_{s,toe}$	3 m	3 m
Wave period	F46	T_{mean} (T_{m-1})	10 s (10 s)	10 s (10 s)
Flow velocity	D46	U_o (m/s)	0 m/s	1 m/s
Damage parameter	G46	N_{od}	1	1
Damage parameter	H46	N	1	1
Computed values				
Delta parameter	H22	Δ	1.62	1.62
Wave length at toe	M47	L	68 m	68 m
Wave length deep water	N47	L_o	156 m	156 m
Peak orbital velocity based on L	P47	U_{max}	1.96 m/s	1.96 m/s
Rock size Van der Meer; Equation (3.3.17)	S47	$D_{n,50}$	0.48 m	0.48 m
Rock size Van Gent; Equation (3.3.18) based on L	Y47	$D_{n,50}$	0.44 m	0.44 m
Rock size Baart; Equation (3.3.19) based on L	V47	$D_{n,50}$	0.51 m	0.51 m
Rock size Muttray; Equation (3.3.20)	AA47	$D_{n,50}$	0.85 m	0.85 m
Rock size van Rijn; Equation (3.2.11)	AF47/AI47	$D_{n,50}$	0.41 m	0.51 m

Table 3.3.12 *Rock sizes of bed protection based on tool ARMOUR.xls (sheet 3)*

Example 1:

Protection layer of stones a on sloping sea bottom of 1 to 25 (tan(α_{bottom}= 0.04)).
Case 1: only waves with $H_{s,toe}$= 3 m at toe of bed protection.
Case 2: waves $H_{s,toe}$ = 3 m plus current of U_o=1 m/s (current normal to waves).
What is the stone size of the bed protection layer?
The input and output data data of the tool ARMOUR.xls (Sheet 3) are given in **Table 3.3.12**.
Equation (3.3.17) yields: $D_{n,50}$ = 0.48 m.
Equation (3.3.18) yields: $D_{n,50}$ = 0.44 m based on local wave length L; $D_{n,50}$ = 0.59 m based on L_o.
Equation (3.3.19) yields: $D_{n,50}$ = 0.51 m with s = $H_{s,toe}/L_o$ = 0.019, ξ = tan(α_{bottom})/s$^{0.5}$ = 0.31, N_{cr}= 3.6.
Equation (3.3.20) yields: $D_{n,50}$ = 0.85 m (much larger value than other values).
Equation (3.2.11) yields: $D_{n,50}$ = 0.41 m with $\theta_{cr,shields}$ = 0.02 and γ_{str}=1.2; mean velocity= 0 m/s.
Equation (3.2.11) yields: $D_{n,50}$ = 0.51 m with $\theta_{cr,shields}$ = 0.02 and γ_{str}=1.2; mean velocity= 1 m/s.
Equation (3.2.11) can take current velocity into account.

Example 2

The stability equations for toe protections have been used to compute the stone size of the toe protection as function of the depth above the toe (based on spreadsheet-model ARMOUR.xls).

The original bottom has a slope of 1 to 25. Other data are:

H_s = significant wave height in front of the toe = 3 m, T_p = peak period = 10 s,

δ_{toe} = thickness of toe above the original bottom= 1 m, B_{toe}= length of toe = 3 m,

Δ = 1.62, $\tan\alpha_1$ =0.04,

N_{od} = damage parameter = 1, $N_{\%}$ = damage parameter= 1,

$\theta_{cr,shields}$ = 0.02, γ_{str}=1.2,

γ_s = safety factor= 1.5.

Figure 3.3.11 shows the results for depth-values (h_{toe}) in the range of 5 to 15 m. The expressions given by Van der Meer 1998 and Van Gent et al.2014 show a weakly decreasing trend with increasing depth-values.

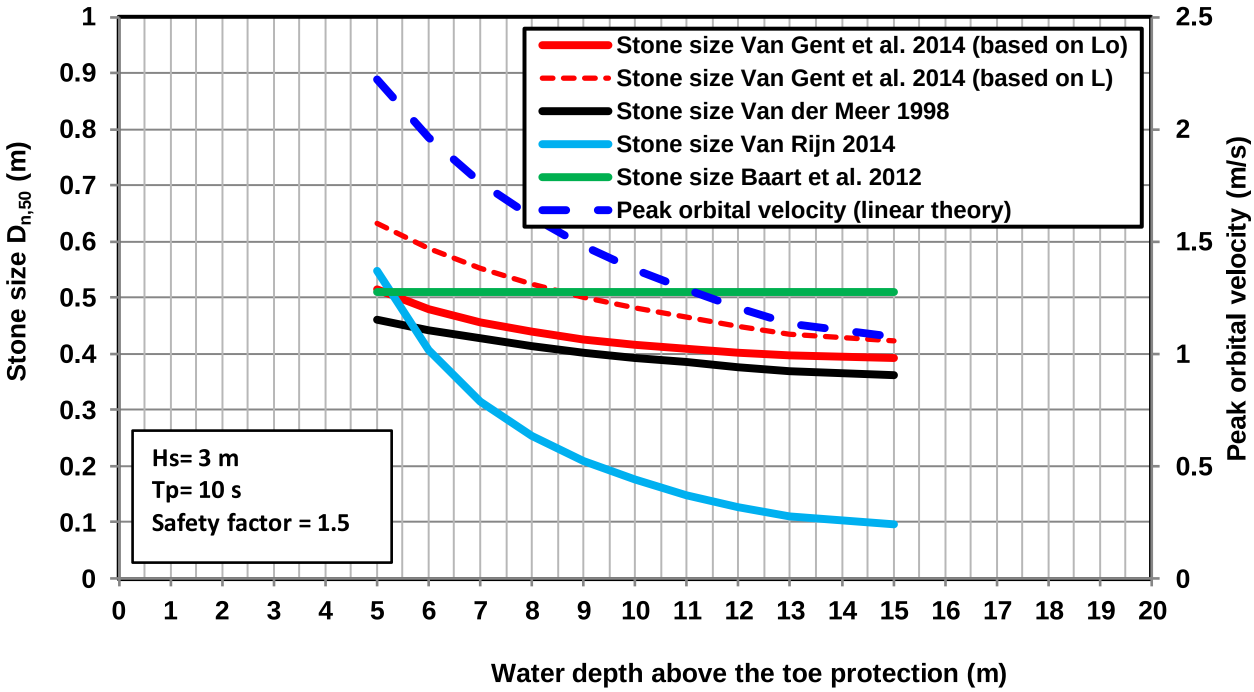


Figure 3.3.11 *Stone size of toe protection as function of depth above the toe*

The stone sizes of Van Gent et al. 2014 are significantly smaller if the local wave length is used in stead of the deep water wave length.The expression of Baart et al. 2010 is only dependent on the bottom slope and the wave height, but not on the water depth above the toe. The expression of Van Rijn (Equation 3.2.10) based on the critical shear stress-method shows a strong effect of the water depth as a result of the decreasing peak orbital velocity for increasing depth. In shallow depth (≅ 5 to 6 m) with breaking waves the stone size is in the range of 0.4 to 0.6 m.

3.3.7 Stability equations for rear side of breakwaters

The rock armour units on the rear side of a structure that can be overtopped by waves is exposed to the downrush of the overtopping waves. The downrush velocities just below the crest can be relatively high in the range of 3 to 5 m/s and the layer thickness of the flow of water is also relatively large. The velocity decreases in downward direction due to friction and lateral spreading.

Van Gent and Pozueta (2004) have given a formula for the $D_{n,50}$ of the rear side rocks of high-crested breakwaters, which reads as:

$$D_{n,50\,rear} = 0.008\,\gamma_{Beta}\,(S_d/N_w^{0.5})^{-0.167}\,(U_{1\%}\,T_{m-1,o}/\Delta^{0.5})\,(\tan\alpha_{rear})]^{0.417}\,[1 + 10\exp(-R_{c,rear}/H_{s,toe})]^{0.167} \quad (3.3.20a)$$

$$U_{1\%} = 1.7\,(g\,\gamma_{r,crest}/\gamma_{r,slope})^{0.5}\,(R-R_c)^{0.5}\,(1+0.1B_{total}/H_{s,toe})^{-1} \quad (3.3.20b)$$

with:
$U_{1\%}$ = maximum velocity at rear side of the crest due to wave overtopping;
R = runup height avove still water level (m);
R_c = crest height above still water level (m);
$R_{c,rear}$ = crest height above still water level at rear side (m);
B_{total} = total crest width (m);
α_{rear} = slope angle of rear side (degrees);
S_d = damage level parameter;
N = number of waves;
Δ = $(\rho_{rock}-\rho_w)/\rho_w$ = relative density of rock;
$\gamma_{r,slope}$ = roughness factor of seaward slope (= 0.55 for rock slopes; = 1 for smooth, impermeable slope);
$\gamma_{r,crest}$ = roughness factor of crest (= 0.55 for rock crest; = 1 for smooth, impermeable crest),
γ_{beta} = obliqueness or wave angle factor (see Van Gent, 2014).

Table 3.3.13 shows the required dimensions of the rock armour units on the rear side based on a graph in Rock Manual 2007. The results can be represented by:

$$D_{n,50\,rear}/D_{n,50,front\,armour} = -0.67\,(R_c/H_{s,toe}) + 1.1 \qquad \text{for } R_c/H_{s,toe} > 0.3 \quad (3.3.21)$$

If the crest is relatively high ($R_c/H_{s,toe} > 1$), the armour layer of the rear side generally is made of randomly placed rocks/stones of smaller size than on the front slope.
If the crest is relatively low ($R_c/H_{s,toe} < 0.5$), the upper part of the rear side generally consists of similar, but somewhat smaller armour units as those of the front side. The armour units of the lower part of the rear side can be made of randomly placed rocks of smaller size.

Relative crest height $R_c/H_{s,toe}$	**Ratio of stone size of rear layer and front layer $D_{n,50\,rear}/D_{n,50,front}$**
< 0.3	1.0
0.3	0.9
0.6	0.7
0.9	0.5
1.0	0.4
1.2	0.3

Table 3.3.13 *Rock/stone size of rear armour layer (slope of 1 to 1.5 or 1 to 2)*

3.3.8 Stability equations for seadikes and revetments

Typical features are:
- relatively mild slope of 1 to 4;
- relatively high crest (almost no overtopping);
- relatively low wave heights at the toe (1.0 to 2.0 m);
- impermeable underlayer.

Various types of armour units are used:

- randomly placed rocks in two layers under water;
- orderly placed rocks in one or two layers above water;
- closely-fitted (pitched) rocks in one layer above water;
- closely-fitted concrete units (Basalton) in one layer above water;
- gabions (cage-type boxes filled with stones);
- bituminous/asphalt layers.

The hydrodynamic loads exerted on a slope of a seadike consisting of a sloping armour layer and almost impermeable underlayers, are:

- wave impact forces;
- wateroverpressure loads under the sloping armour layer due to wave forces;
- friction forces along the slope due to water flow.

3.3.8.1 Randomly placed rocks

The stability of randomly placed rocks in two layers on a slope of a seadike or revetment with an impermeable underlayer can be described by Equations (3.3.6) of Van Gent et al. (2003) and Equation (3.3.7) of Van der Meer (1988).

3.3.8.2 Orderly placed and closely-fitted rocks and concrete blocks

Pilarczyk (1990) introduced an empirical formula for various types of armour layers (**Table 3.3.14**), as follows:

$$N_{cr} = (2.7/\gamma_s)\ \phi\ \xi^{-0.67} \cos(\alpha) \qquad (3.3.22)$$

with: ϕ = empirical stability factor, see **Table 3.3.14**, ξ = surf similarity factor, α = slope angle (slope in the range between 1 to 3 and 1 to 8; slope of 1 to 4 has angle of 15 degrees).

Type of armour material	Relative density Δ	Stability factor ϕ
Stones/rocks (placed in 2 layers)	1.6	1.0
Stones/rocks (regular shape, closely-fitted)	1.6	1.3
Basalt blocks (closely-fitted)	1.6	1.5
Concrete blocks (closely-fitted)	1.3	1.5
Concrete blocks (connected to each other)	1.3	2.0
Concrete block mattrass on geotextile	1.3	1.5
Gabions filled with stones/rocks (closely fitted)	1.6	2.0

Table 3.3.14 *Empirical stability factors*

Nurmohamed et al. (2006) have studied the stability of orderly placed rocks and closely fitted (pitched) rocks in a single layer with sizes in the range of 0.3 to 0.5 m (grading D_{85}/D_{15} in the range of 1.2 to 1.7).
Based on the work of Nurmohamed et al. (2006), the N_{cr}-values can be described by:

$$N_{cr} = (4.8/\gamma_s)\ \xi^{-0.8} \quad \text{for } \xi < 3 \text{ (plunging breaking waves)} \qquad (3.3.23a)$$
$$N_{cr} = (1/\gamma_s)\ \xi^{0.6} \quad \text{for } \xi \geq 3 \text{ (surging waves)} \qquad (3.3.23b)$$

with: ξ = surf similarity parameter and γ_s= safety factor for deterministic design (= 1.5 for orderly placed rocks in a single layer). Pitched rocks are somewhat more stable than orderly placed rocks.

Closely-fitted concrete units (Basalton, $\rho_{concrete}$= 2300 kg/m^3; www.holcim.nl) in a single layer with granular space filling placed on a dike slope has been tested in the large-scale Deltaflume of Deltares.
Based on these results, the N_{cr}-values of concrete Basalton blocks can be described by:

$$N_{cr} = (6.5/\gamma_s)\ \xi^{-0.67} \quad \text{for } 1.5 < \xi < 2.5 \text{ (plunging breaking waves)} \qquad (3.3.24)$$

with: ξ = surf similarity parameter, γ_s= safety factor for deterministic design (= 1.3 to 1.5 single layer).

3.3.8.3 Ordely placed Gabions

Equation (3.3.23) and the data of Table 3.3.11 can be used to determine the stability of gabions (filled with rocks/stones). The porosity (p) of the rocks/stones (gage filling) should be taken into account.
Thus: $N_{cr} = H_{s,toe}/((1-p)\ \Delta\ D_{n,50})$.

3.3.8.4 Bituminous layers

Armour layer of stones can be made more stable by using bituminous or cement mixtures (as bonding material). This will result in an almost impermeable and strong layer. These types of armour layers should only be used on an impermeable layer of clay or on a rather impermeable filter layer (wide graded filter material) to prevent the generation of high wateroverpressure loads at the bottom side of the armour layer. The thickness of a fully bituminous (asphalt) layer should be about 0.15 m. Usually, bituminous layers are only used near the design water level (berm used as maintenance road)

3.3.8.5 Example case 1

Input data:
$H_{s,toe}$ = 2 m, T_p = 6 s, $\alpha \cong 17$ degrees,
tan(α) = 0.3 (1 to 3.3), cos(α) = 0.95,
ρ_{rock} = 2700 kg/m^3, $\rho_{concrete}$ = 2300 kg/m^3, p = porosity = 0.4, ρ_{water} = 1025 kg/m^3, Δ_{rock} = 1.63 (saline water), $\Delta_{concrete}$ = 1.24,
γ_s = 1.5 (single layer), γ_s = 1.1 (double layer),
s = H/L_o = 0.036, γ_H= 0.8, S_d = 2, N_w = 3600, P_M= 0.1, P_G= 0, ξ = 1.6.
The results are given in **Table 3.3.15**.

Type of armour layer	N_{cr}	$D_{n,50}$
Randomly placed rocks double Van Gent et al. 2003; ARMOUR.xls (sheet 4)	1.5	0.81 m
Van der Meer 1988; ARMOUR.xls (sheet 4)	1.7	0.71 m
Ordely placed rocks in single layer; Equation (3.3.23);	2.20	0.56 m
Closely-fitted concrete blocks single layer; Equation (3.3.22)	1.87	0.86 m
Closely-fitted concrete blocks single layer (Basalton); Equation (3.3.24)	3.17	0.51 m
Gabions filled with rocks/stones (single)	2.5	0.81 m

Table 3.3.15 *Armour size of seadike or revetment*

3.3.8.6 Example case 2

Seadike (no berm) with mild, smooth slope of 1 to 4: tan(α)= 0.25,
ρ_{rock} = 2700 kg/m^3, $\rho_{concrete}$ = 2300 kg/m^3, ρ_w = 1025 kg/m^3.
Water depth at toe = 3 m (to MSL); crest height is not important for the rock size in the wave attack zone.
Maximum water level (storm surge level, SSL) is in the range of 0 to 3 m above mean sea level (MSL).
Wave heights and wave periods are: $H_{s,toe}$ = 1, 2, 2.5, 3, 3.5 and 4 m and T_p = 6, 8, 9, 10, 11, 12 s.

Waves perpendicular to seadike: $\beta = 0°$.
Safety factor armour $\gamma_s = 1.2$ (double layer) and 1.5 (single).
Van der Meer: $P_M = 0.4$, $S_d = 2$, $N_w = 2160$
Van Gent: $P_G = 0.3$, $S_d = 2$, $N_w = 2160$

Figure 3.3.12 shows the armour size as function of the significant wave height using various types of armour units and placement methods. The formulae of Van Gent and Van der Meer yield about the same results for rocks. The size of closely-fitted concrete blocks (Basalton) are slightly smaller (10%) than randomly-placed rocks. Basalton in a single layer has a smaller density than rock, but the safety factor is larger (1.5 instead of 1.2). The size can be reduced (15%) by using orderly placed rocks in a double layer (smaller safety factor 1.2).

Table 3.3.16 shows wave overtopping rates for various crest levels. The crest should be at +17 m (to MSL) to reduce the wave overtopping rate to about 1 l/m/s. The crest can be reduced to +15 m if roughness elements (10% of the local surface area) are placed on the seaward slope below the crest.

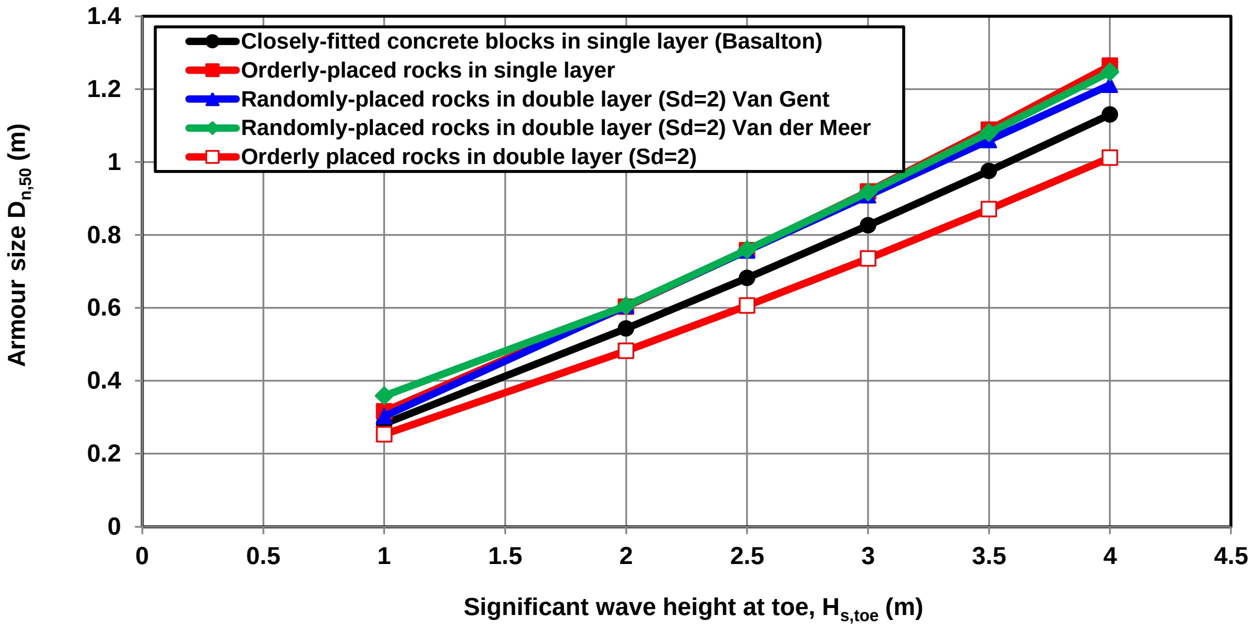


Figure 3.3.12 *Armour size as function of significant wave height*

Wave height (m)	Maximum water level (m)	Wave overtopping rate (l/m/s)			
		Crest = 10 m (γ_r= 0.9)	Crest = 15 m (γ_r= 0.9)	Crest = 17 m (γ_r= 0.9)	Crest = 15 m + roughness (γ_r= 0.8)
1	0	0	0	0	0
2	1	0.04	0	0	0
2.5	1.5	0.47	0	0	0
3	2	5.35	0.08	0.01	0.02
3.5	2.5	25.7	0.66	0.16	0.21
4	3	85.8	3.51	0.98	1.3

Table 3.3.16 *Wave overtopping rates*

4 PRACTICAL DESIGN OF SEADIKES, REVETMENTS AND BED PROTECTIONS

4.1 Types of structures and armouring

4.1.1 General

Seadikes and embankments are built as flood protection structures along coastal sections where natural defences such as sand dunes, cliffs or rock formations are absent, see **Figure 4.1.1**.
Generally, these types of structures have a smooth, impermeable surface slope at the seaside in the range between 1 to 3 and 1 to 5. Milder slopes reduce the wave runup. Berms and roughness elements are often constructed on the upper part of the slope to reduce wave runup and wave overtopping. The maximum nearshore wave heights at the toe of the dike are of the order of 2 to 3 m during storm events due the limited water depths.

Figure 4.1.1 *Seadike between Camperduin and Petten, The Netherlands*
(Crest = 12.8 m above MSL, Berm at 5.5 m above MSL, Slopes between 1 to 4 and 1 to 8)

The volume of the dike body can be reduced by using a relatively mild slope of 1 to 4 and a berm (see **Figure 4.1.2**), which reduces the wave runup and thus the crest height. **Figure 4.1.2** shows examples of various dike profiles, all having the same wave runup level.

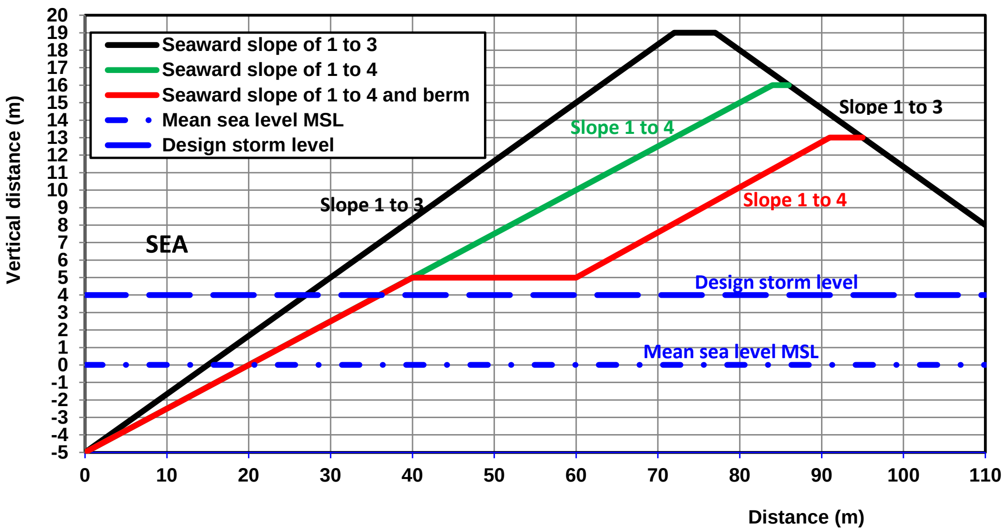


Figure 4.1.2 *Different dike profiles and dike height with the same wave runup level*

The armouring of the seaward slope of a seadike or revetment generally consists of the following layers:

- ***subsoil of sand and/or clay***; subsoil should be sufficiently compacted to prevent settlement under loading conditions;
- ***geotextile filter***; artificial permeable fabrics made of polyester or polypropylene to prevent the erosion of particles from the subsoil; a geotextile filter is necessary between two layers of different granular materials if a significant fraction of the finer-grained layer cannot be restrained by the coarser layer under the expected pore water flow (if $D_{50,upperlayer}/D_{50,subsoil} > 5$); special sinkable geotextiles and mattrasses are available for underwater applications (double layer geotextiles with and without granular filling);
- ***filter layers***; sublayers consisting of granular materials to spread the load over a larger area; to reduce the erosion of particles from lower layers and to reduce water overpressures under high loading conditions; the permeability of the upper layer should always be larger than that of the lower layer; one layer of gravel with grain sizes of 10 to 30 mm placed on geotextile on subsoil of sand generally is sufficient (see also **Figures 4.1.3 and 4.1.5**); the layer thickness depends on the loading zone (thicker layers under high loads; 0.4 to 0.8 m); the filter layer is often covered with a geotextile and a thin granular levelling layer (narrow-graded 20 to 40 mm) on top of it; filter layers are not required if firm clay is used as subsoil;
- ***top armour layer*** consisting of rocks or crushed rock/concrete blocks, asphalt, etc. (**Table 4.1.1**);

Figure 4.1.3 *Armouring made of concrete blocks (Basalton) on thin granular levelling layer and geotextile*

The type of materials used for armour slopes of seadikes and revetments are given in **Table 4.1.1**.

Type of armouring	Description	Applications
1. Crushed rocks/stones	A. Randomly dumped (2 layers); least stable B. Orderly placed (2 layers); reasonably stable C. Closely fitted (1 layer; pitched position); most stable Various size/weight classes are available but most often the vertical size (thickness) is between 0.3 m and 0.5 m for exposed seadikes (Class IV stones); Smaller stones are used as filter layers (10 to 100 mm); Stones for underwater layers should be dumped through a pipe (to preserve the right size grading)	Filter layers; Toe and bed protection under water; Top layer (only orderly-placed rocks)
2. Closely-fitted blocks of basalt and concrete (basalton)	Closely-fitted blocks with thickness of 0.3 to 0.5 m placed above the LW tide level up to the berm level (low to high wave loads); spaces between blocks should be filled with fine granular materials (16-32 mm) to increase the resistance of the blocks against vertical movements due to wave forces; larger blocks in upper zone to reduce runup	Permeable top layer above LW tide level up to the berm level (2 to 4 m above MSL); zone of low to high wave loads; very aesthetic appearance
3. Crushed rocks/stones impregnated with cement and bituminous mixtures	Strong impermeable armour layer (two layers) with large resistance againt wave forces; wave runup increases over impermeable surface; water overpressure under armour layer due to high loads may lead to local breakouts at weak spots (damage); easy maintenance ; the maximum stone range is 5 to 50 kg; otherwise the spaces are too large	Impermeable top layer above LW tide level up to the berm level (2 to 4 m above MSL); zone of low to high wave loads
4. Asphalt layer	Very smooth and impermeable layer; wave runup is relatively large; roughness elements are often required to reduce runup; water overpressure under armour layer which may lead to local breakouts at weak spots (damage); easy maintenance;	Impermeable top layer from just below berm level to crest; zone of high wave loads
5. Cage-type boxes of wire filled with crushed rock (gabions)	Wire material should be resistant (galvanised or coated) against corrosion by salt water; easy construction and practical use in developing countries	Permeable armour units for toe protection of nearshore groins
6. Block mats and mattrasses	Concrete blocks are attached to each other by wire or by geotextile material	Semi-permeable top layer under water and in lower zone of dike up to HW tide level

Table 4.1.1 *Armour materials for seadikes/revetments*

4.1.2 Traditional seadike

The cross-section of a traditional seadike generally consists of the following zones (see also **Figure 4.1.5** and **Table 4.1.1**):

- core of sand and clay;
- fully submerged toe protection (up to low water level LW), consisting of two or three layers of loose materials (crushed stones/rock of 0.2 to 0.4 m) on geotextile or on granular filter layer of finer crushed materials (0.5 to 1 m thick); see **Figure 4.1.5**;
- lower armour zone with slope of 1 to 4 between LW and berm level (about 2 to 4 m above MSL), mostly exposed to low daily waves and high waves during storm events, consisting of closely fitted basalt or concrete blocks on foundation layer of finer crushed materials (0.5 to 1 m thick);
- berm of 10 to 20 m wide (with maintenance access road) just above the design water level (50 to 100 year return period), consisting of an asphalt layer on sublayer of fine crushed materials;
- upper armour zone with slope of 1 to 4 exposed to high waves and wave runup, consisting of closely fitted blocks on sublayer of fine crushed materials (0.5 to 1 m thick); roughness elements can be used to reduce wave runup and wave overtopping;
- crest with superstructure (vertical wall, road, width of 3 to 10 m), consisting of asphalt layer;
- rear side with slope of 1 to 3 consisting of grass on sublayer of clay (0.5 to 1 m thick).

If the land surface behind the dike is below mean sea level, the penetration of salt water through the dike body may be a problem and mitigating measures using impermeable sublayers of clay may be required on the seaward side (see also **Table 4.1.2**).

Many old seadikes in The Netherlands, built in the period 1800 to 1900, have an outer armour layer consisting of pitched stones with sizes in the range of 0.3 to 0.5 m (weight 100 to 400 kg; grading < 1.7). These dikes are mostly situated along the Wadden Sea coast (less exposed with $H_{s,toe}$ of about 2 m). An armour layer of pitched stones consists of a single layer of permeable, very closely packed natural stones/rocks, often with their longest axis perpendicular to the dike surface and has a very aesthetic appearance, see **Figure 4.1.4**. The pitched pattern in a single layer is much more stable than a randomly dumped pattern of 2 layers. The dikes have survived about 150 years with almost no damage (Van de Paverd, 1993 and Nurmohamed et al., 2006).
Nowadays, the armour layer is made of closely fitted concrete units (Basalton, www.holcim.nl), see **Figure 4.1.3**. These concrete units are the modern alternative for closely fitted stones as used in old seadikes.

Figure 4.1.4 *Armour layer consisting of pitched stones (left) and randomly dumped stones at toe (right)*

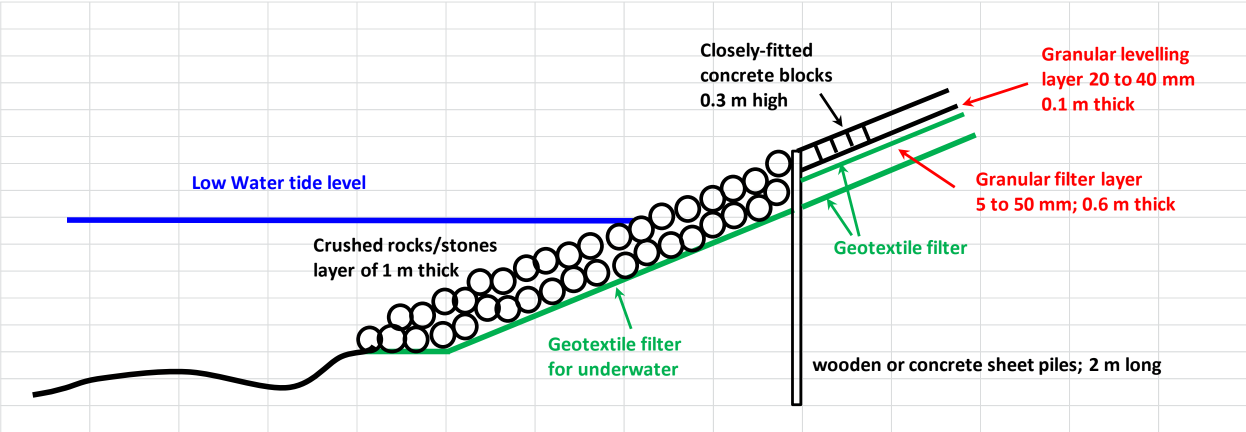


Figure 4.1.5 *Armouring of toe and lower zone of conventional seadike/revetment in The Netherlands*

Subsoil of clay	**Subsoil of sand**
1. geotextile on clay 2. thin filter layer (0.1 to 0.2 m) of granular material (20-40 mm); D_{15} of filter material should be larger than 20 mm to prevent erosion through the spaces between armour blocks 3. closely-fitted basalt blocks or concrete blocks (Basalton) 4. space filling of very angular crushed rock of high density (2900 kg/m^3) with sizes in the range of 4-32 mm or 16-32 mm depending on maximum space size between armour blocks (largest space size of basalton is about 40 mm)	1. geotextile on sand 2. thick foundation layer (0.6 m) of crushed rock (2-60 mm) of wide grading to reduce permeability 2. thin filter layer (0.1 to 0.2 m) of granular material (20-40 mm); D_{15} of filter material should be larger than 20 mm to prevent erosion through spaces between armour blocks 3. closely-fitted basalt blocks or concrete blocks (Basalton) 4. space filling of very angular crushed rock of high density (2900 kg/m^3) with sizes in the range of 4-32 mm or 16-32 mm depending on maximum space size between armour blocks (largest space size of Basalton is about 40 mm)

Table 4.1.2 *Armour and underlayers of conventional seadike/revetment in The Netherlands*

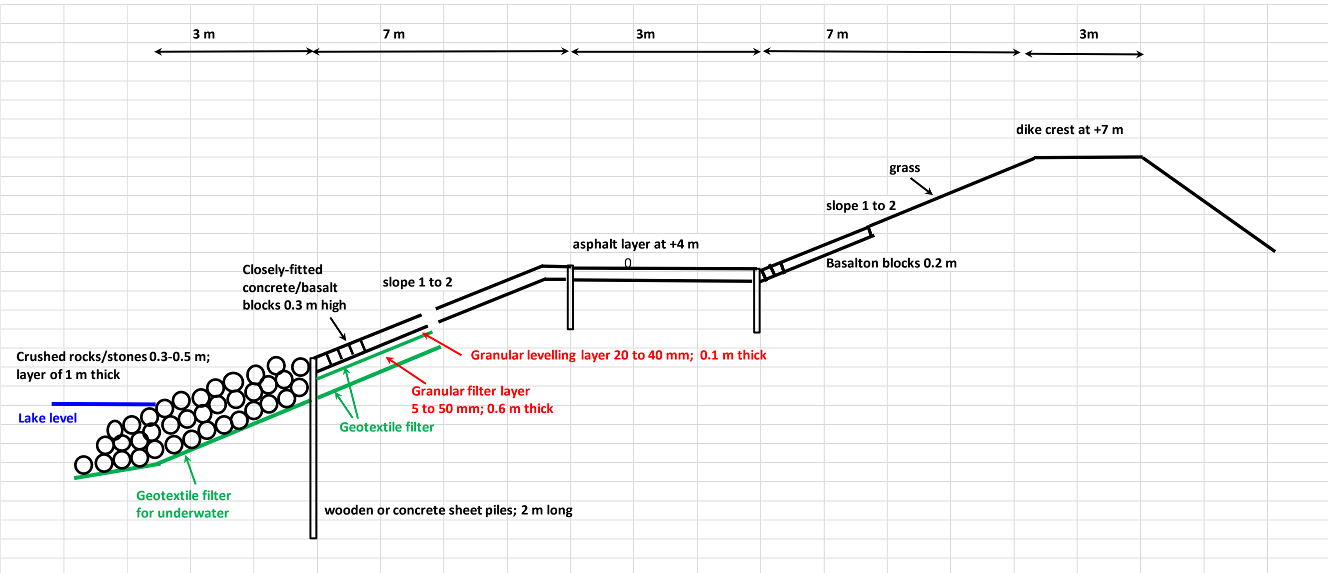


Figure 4.1.6 *Armouring of toe and lower zone of conventional lake dike in The Netherlands*

Figure 4.1.6 shows a typical cross-section of a dike along a large-scale lake (IJsselmeer lake) in the Netherlands. The dike crest is at +7 m. The armouring consists of: crushed rocks at the lower base; closely-fitted basalt blocks (or basalton) at the lower slope; asphalt layer (road) at the berm and grass at the upper slope above the +5 m.

4.1.3 Modern seadefence

A modern seadefence (length of 3.5 km) for a land reclamation with land surface at 5 m above mean sea level (sea defence Maasvlakte 2, The Netherlands) consists of (see **Figure 4.1.7A**);

- core of sand of 0.25 to 0.35 mm (1);
- detached breakwater with crest at about +2 m above MSL consisting of concrete blocks (5) of 2.5x2.5x2.5 m^3 and mass of 40 tonnes on foundation layers of crushed stones (4) of 150-800 kg and 5-70 kg and a filter layer of gravel/pebbles (3 to 35 mm); the toe protection (6) consists of rock blocks of 1 to 10 tons; sandy bed is fully protected down to the -6 m depth contour to prevent erosion by waves plus longshore currents;
- armour slope of 1 to 7.5 consisting of cobbles (4) in the range of 20 to 135 mm between the breakwater and the crest of the dike;
- crest at about 14 m above mean sea level;
- rear side of the dike consists of a layer of clay and grass as protection against erosion due to overtopping waves during extreme events.

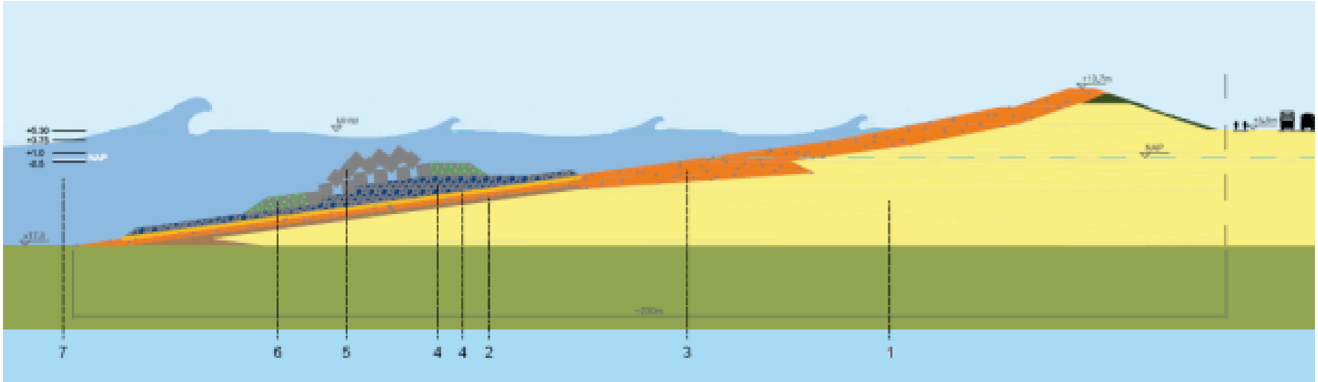


Figure 4.1.7A *Example of sea defence along land reclamation (Maasvlakte 2, The Netherlands)*

Figure 4.1.7B shows some photographs of the hard sea defence of the Maasvlakte 2 at the transition point from sandy beach defence to hard dike defence. The length of the hard defence zone is about 3.5 km and is located on the south side of the entrance to the Rotterdam Waterway.

The hard defence consists of:

- shore-parallel breakwater of concrete cubes (2.5x2.5x2.5 m^3) with crest at about +2 m above MSL;
- a layer of angular crushed rock/cobbles (size 20 to 135 mm) up to the crest of the dike at about +9 m above MSL.

Various wind screens are placed to prevent the wind-induced erosion of sand into the hard defence zone.

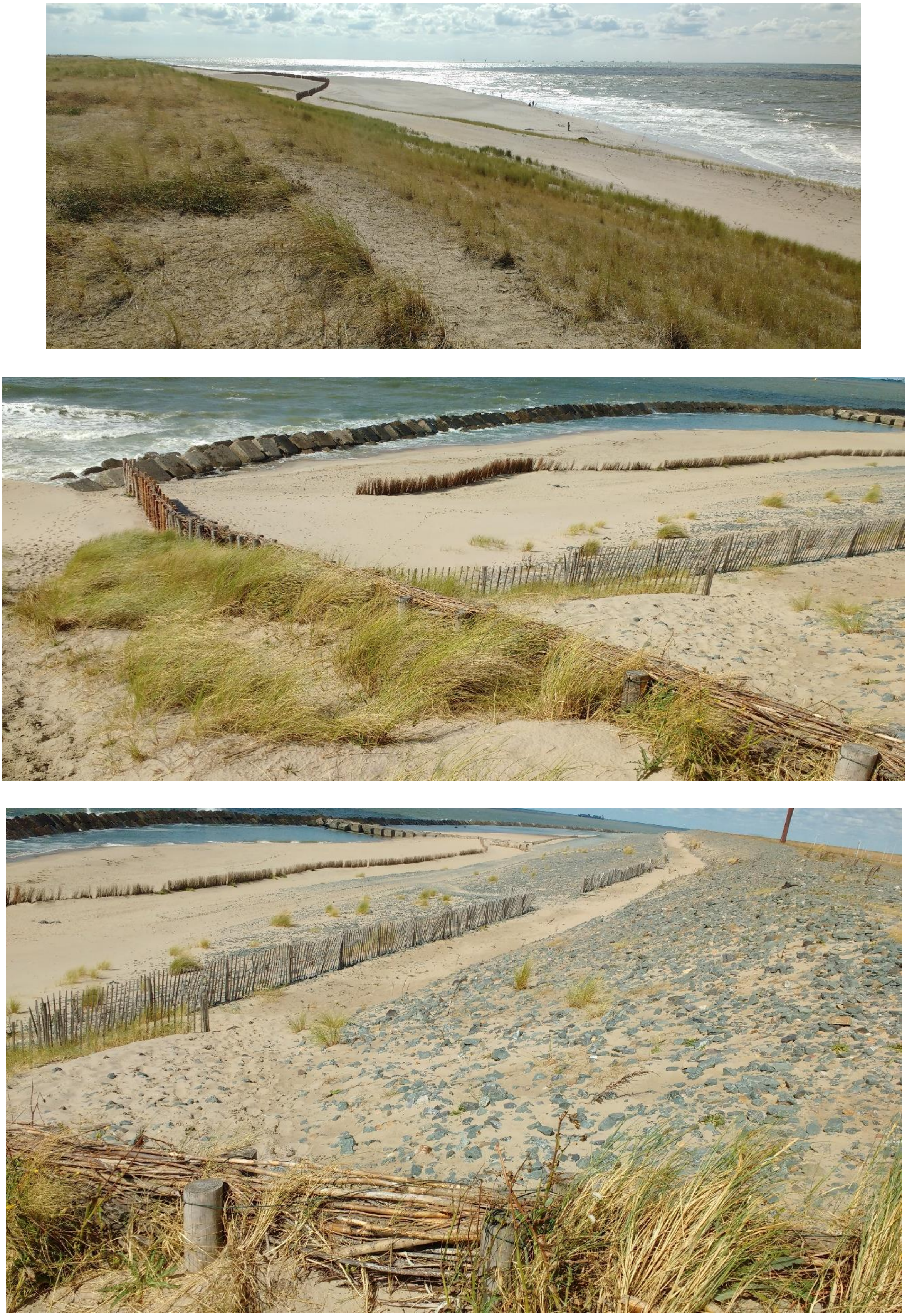


Figure 4.1.7B *Sea defence of Maasvlakte 2 at transition from sandy beach to hard defence (August 2018)*
Upper: *sand beach;* ***Middle and Lower:*** *hard sea defence with concrete cubes and cobbles*

4.1.4 Landward side of seadike

Generally, the landward side of a seadike consists of grass on a thick layer of clay (1 m). During storm events with significant overtopping, the landward side may be endangered by erosion of the top layer due to overtopping waves. The strength of the grass cover of a particular dike under overtopping waves can be tested using an in-situ wave overtopping simulator (Van der Meer, 2011), which is a 4 m wide, high-level mobile box with a water storage capacity of 22 m^3 and a maximum discharge rate of 125 l/m/s simulating overtopping waves in the range of 1 to 3 m during a period of 6 hours.
In-situ tests using the wave overtopping simulator (Van der Meer 2011) have shown that wave overtopping rates as large as 30 l/m/s over a period of 6 hours (storm event duration) donot significantly damage the top layer of grass on a sublayer of clay.
If the wave overtopping rate is larger than 30 l/m/s, the landward side can be eroded and should be protected with an armour layer. A large overtopping rate implies that large volumes of water have to be pumped out at the toe of the dike during or shortly after a storm event, which requires a relatively large pumping capacity for rare events. It may be better (safer) to take measures to reduce the wave overtopping rate by raising the dike crest level. The foreshore on the seaward side of the dike can also be raised and armoured over a length of about 100 m to reduce the incoming wave height.

4.2 Wave runup and wave overtopping of seadikes

The crest height of a seadike depends on:
- extreme water levels (return period > 1000 years; 10,000 years is used in The Netherlands);
- sea level rise and subsidence during the life time of the dike (100 years);
- wave runup and oscillation margin (to account for water level oscillations due to wind gusts).

The crest height of smooth sloping and impermeable structures is primarily based on the maximum runup and overtopping rate during design conditions. Minor wave overtopping in the range of 0.1 to 1 l/m/s may be allowed in extreme events with a large return period (> 100 years). The water depth at the toe of seadikes and revetments often is relatively small in the range of 3 to 5 m during storm events. Hence, the maximum wave height at the toe is of the order of 2 to 3 m with periods periods in the range of 10 to 15 s. At open ocean coasts large swell waves (2 to 3 m) may occur with periods in the range of 15 to 20 s. The wave runup and wave overtopping rate for smooth slope structures can be computed by using the spreadsheet-model **ARMOUR.xls**.

4.3 Armour size of seaward dike slope

The spreadsheet-model **ARMOUR.xls** (based on the equations of Chapter 2 and 3) can be used to determine the wave overtopping rates and the armour sizes. Examples are given in **Section 3.3.8**.

4.4 Scour protections around structures on the seabed

4.4.1 Scour protection around monopile in seabed

To prevent the gradual burial of a structure paced on the seabed, a scour protection around the structure is required. Two basic types of scour protection structures can be distinguished:

- loose stones/rocks; protection layer generally consists of geotextile placed on seabed; filter layer of smaller stones with thickness of 2 times the mean stone size and upper armour layer of larger stones with thickness of 3 times the mean stone size (Figure 4.4.1);
- mattress of concrete blocks or rock-filled wire-cage (gabion); gabion type mattress generally consists of a lower mattress with geotextile in it and smaller stones and an upper mattress with larger stones; mattress thickness in range of 0.2 to 0.3 m (Figure 4.4.1).

Possible bed protection solutions for a monopile are shown in **Figure 4.4.1**. Based on scour data of field cases, the horizontal diameter of the scour protection is found to be about 3 to $5D_{pile}$. Beyond the scour protection, edge scour will occur with maximum depth of about 2 m. Edge scour depth will be smaller for larger scour protection diameters.

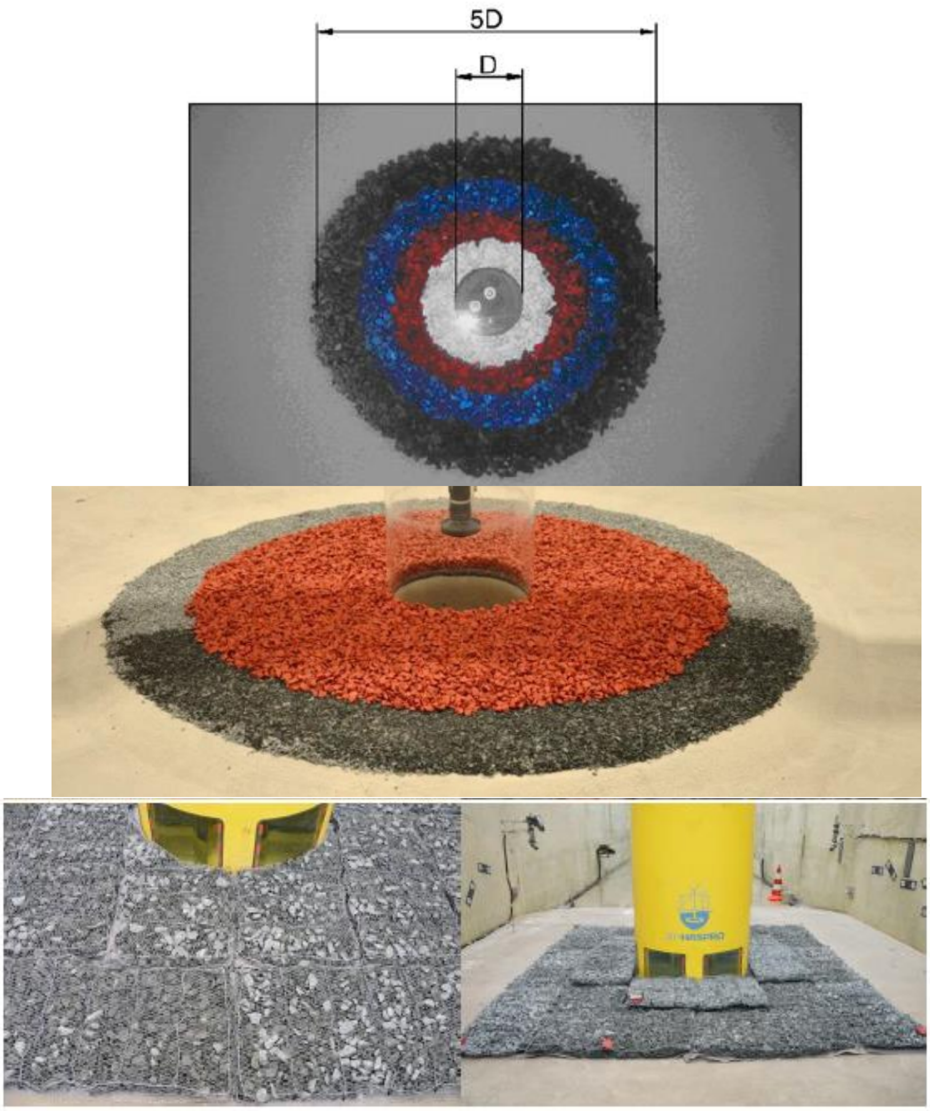


Figure 4.4.1 *Example of rock protection of wind mill pile*

The technical requirements for a gravel/rock-type protection layer (see **Figure 4.4.2**) are:

- external stability; each gravel or rock element of the layer should be stable under the hydraulic loads during storm events; static stability accepting no movement requires relatively large gravel/rock sizes, whereas dynamic stability accepting limited movement and some damage yields smaller sizes.
- internal stability, which can obtained by selecting the proper size grading (D_{85}/D_{15} <10) and filter layers(s) so that the erosion of the smaller sizes and sand from the seabed is minimum.
- flexibility, which is required so that the edges of the protection layer can adjust to seabed lowering on the stoss-side of migrating sand waves and to edge scour which may occur at the transition from protection layer to the sandy seabed; flexibility can be obtained by increasing the length of the filter layer.
- robustness; the protection layer should be sufficiently thick and long to survive the design lifetime of the structure.
- low construction and maintenance costs.

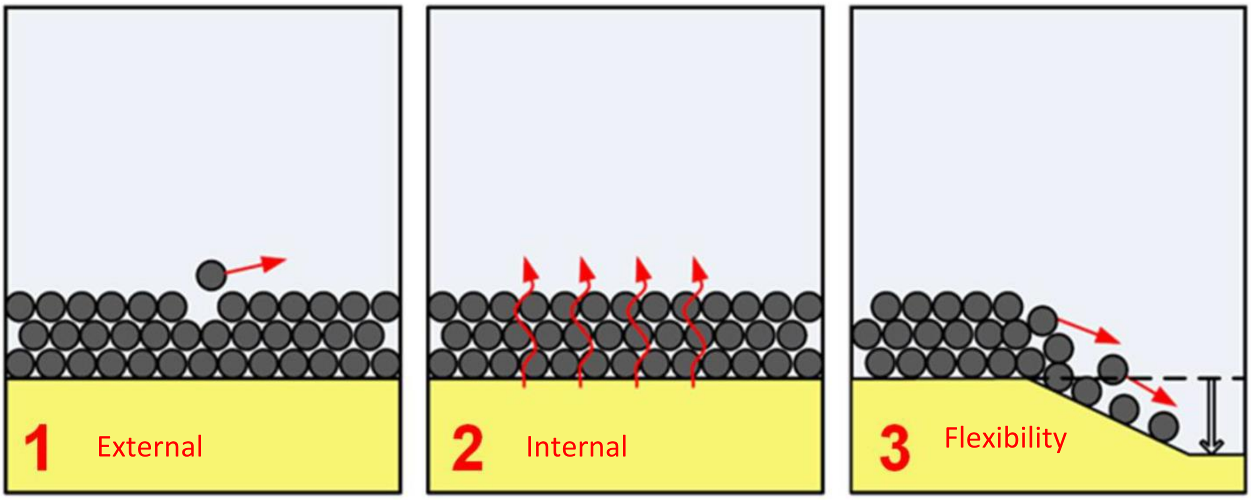


Figure 4.4.2 *Stability requirements (Deltares 2017, 2020)*

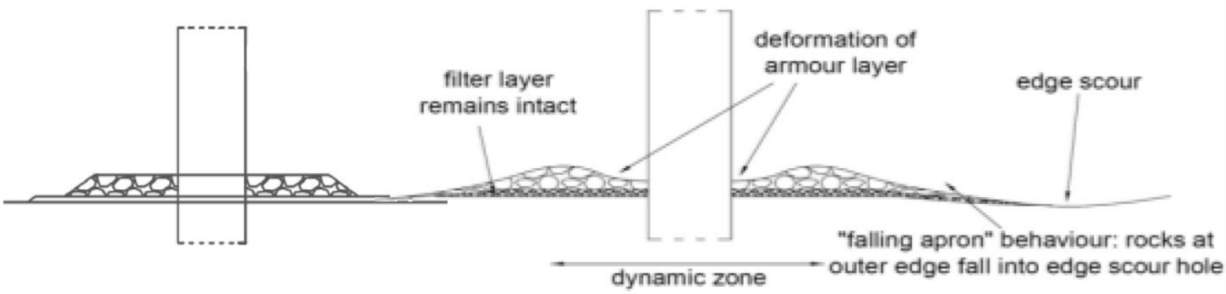


Figure 4.4.3 *Statically stable protection (left) and dynamically stable protection (right), (Deltares 2017)*

The size of loose rocks/stones strongly depend on the acceptable damage level, as follows:

- statically stable protection layer; the classical design consists of a lower filter layer of smaller sizes and an upper armour-layer of larger sizes which are stable under all extreme conditions (waves with return period of 50 years) with minimum damage (losses); horizontal diameter is about $5D_{pile}$ for armour-layer and about $7D_{pile}$ for filter layer (see **Figures 4.4.3** and **4.4.4**); length of filter layer should be sufficient ($7D_{pile}$) to give a smooth transition to the slope of the edge scour pit; length of filter layer can be extended in the direction of the most severe conditions; edge scour will be relatively large if armour layer consists of very large and rough rocks;
- dynamically stable protection layer consisting of a lower filter layer (0.5 m) and an upper armour-layer (1 m) with rock sizes allowing limited deformation of the upper layer (0.5 m); stone movement is allowed during storm conditions as long as a minimum layer is maintained (filter layer should not be exposed); smaller rock sizes can be used;
- dynamically stable protection layer consisting of a single layer with a relatively wide grading ($D_{85}/D_{15} \cong 5$ to 10) which is mostly applied in milder wave conditions to reduce damage (water depth > 25 m); the thickness and volume of rocks should be sufficiently large to deal with deformations (thickness of at least 1.5 m); regular monitoring and maintenance are required; rocks of higher density can be used (eclogite with density of 3000 kg/m³ in stead of granite with density of 2650 kg/m³); commonly used gradings for North Sea conditions are: 3 to 9 inch/0.08-0.25m HD (high density) or 10-200 kg ND (normal density); see **Table 4.4.1**.

The total thickness of the protection layer is in the range of 1 to 1.5 m depending on conditions. The wave parameters should be determined at the top level of the protection layer ($h_{effective} = h_{water}$ - layer thickness).
In conditions with large migrating sand waves, the seabed around the structure may lower substantially (2 to 5 m; passage of sand wave trough) over a period of 25 to 50 years. The edge of the bed protection should be sufficiently long and flexible to partly follow this lowering (falling apron).
An edge scour pit will be generated at the end of the protection layer (**Figure 4.4.4**). The pit depth will be smaller for a longer protection layer. The filter layer will be partly undermined by scour and redistributed over the slope of the scour pit during storm events (known as falling apron; layer thickness will vary in the range of 0.2 to 0.5 m).
Assuming a pit slope of 1 to 2 (about 30°), the slope length will be about $L_{slope}=2.5\, d_{s,edge}$.

The downstream edge scour can be reduced substantially by a longer filter layer. At some piles of the wind park Egmond aan Zee in a depth of 18 m (The Netherlands), the dumping activities of filter material (< 0.1 m) had resulted (unintentionally) in a longer protection layer which remained in tact over 7 years and the maximum edge scour depth of the medium fine sand bed (0.25 mm) was found to be much lower (0.1 to $0.2D_{pile}$), (Petersen et al., 2015). This proves that the smaller stones (<0.1 m) of the filter layer are quite stable over a long period of time in conditions with waves up to 6 m.

If sand waves are present on the seabed, the local sea bed may lower (d_{lb}) in future due to sand wave migration. Maximum lowered bed values (d_{lb}) are about 2 to 5 m.
Taking this into account, then the slope length of the edge scour pit will be $L_{slope}=2.5\,(d_{s,e} + d_{lb})$.

Using: D_{pile}= 7 m, $d_{s,e}$ = 2 m and d_{lb}=5 m; the $L_{slope}=2.5x(2+5) \cong 18$ m, which is about 2 to 3 D_{pile}.

Thus, the length (diameter) of the lower protection layer (in case of a double layer) is approximately:
$L_{protection}=3\,D_{pile} + L_{slope} \cong 3\,D_{pile} + 2.5\,(d_{s,e} + d_{lb}) \cong 5$ to 6 D_{pile}

The bed protection beyond the cover layer will gradually sink into the edge scour pit as a falling apron during storm events, see **Figure 4.4.5**. The smaller stones of the filter (under) layer are mobile during storm conditions and will be transported into the edge scout pit. Furthermore, the edge of the filter layer may

become partly undermined. Both will result in a falling apron at the slope of the pit. Most damage of the scour protection occurs on the flanks of the pile, see **Figure 4.4.5**.
In the case of a single layer with a larger thickness (≅1.5 m) over the full protection length, more material is available for redistribution over the pit slope and thus the additional length can be reduced slightly (say 20%; maximum $8D_{pile}$).

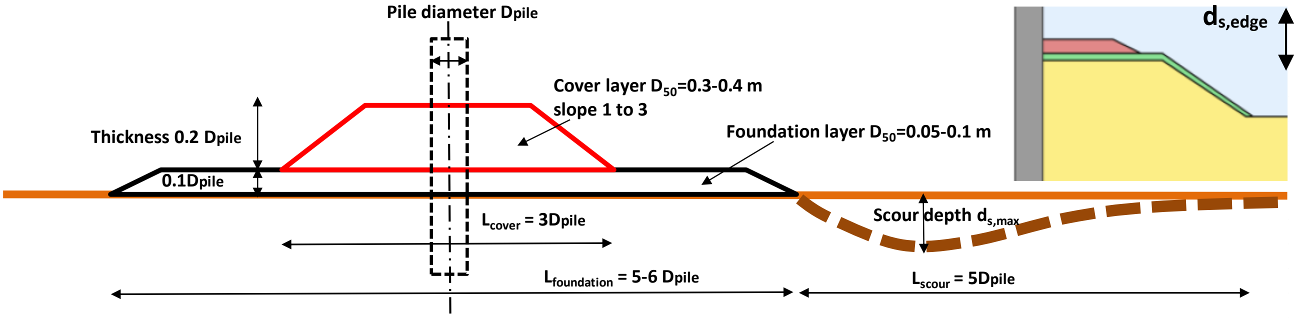


Figure 4.4.4 *Scour protection of monopile in seabed*

Typical solutions for scour protection around a monopile with diameters in the range of 5 to 10 m are:

- water depth < 25 m (relatively shallow water): double layer of rocks (filter layer ND3-9 or HD2-8 with thickness of 0.5 m) with upper layer of grading 10-200 kg and thickness of 1 m;
- water depth > 25 m (relatively deep water): single layer of rocks of grading ND3-9 (or HD2-8) with thickness of 1.5 m.

Model testing of dynamically stable scour protections in a wave-current tank is highly recommended.

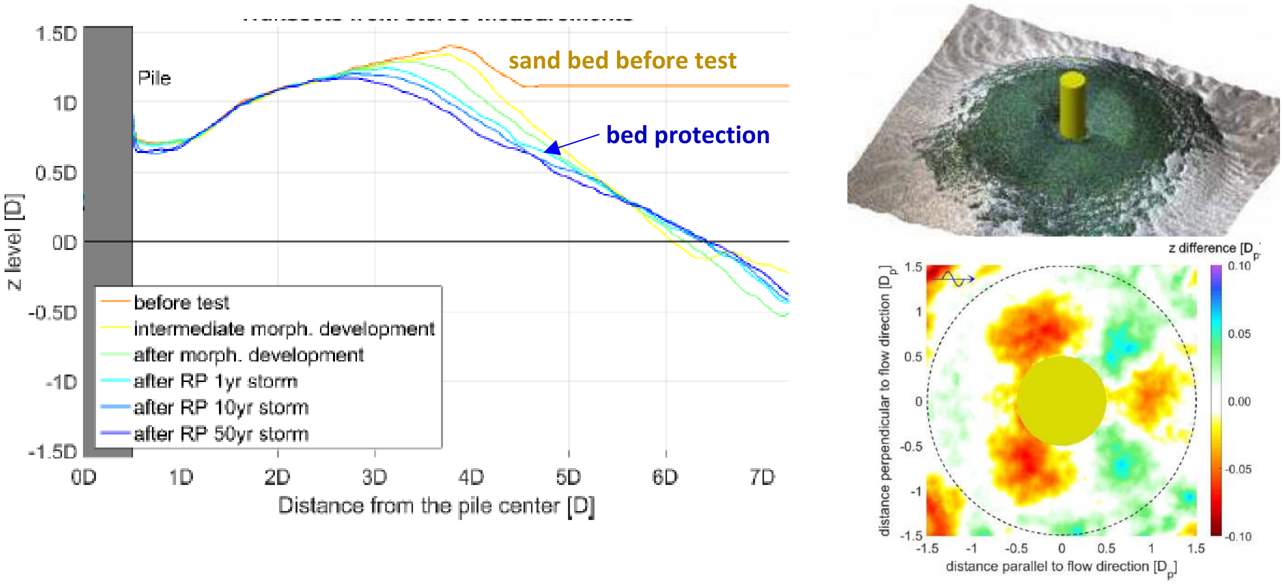


Figure 4.4.5. *Scour test wit bed protection around pile (blue line represents surface of protected bed with falling apron); Riezebos et al. (2016)*

Rock grading	Median size (mm)	Density (kg/m³)	Type of layer
60-300 kg ND	470 (350-600)	2650	armour layer
40-200 kg ND	400 (300-520)	2650	armour layer
10-200 kg ND	350 (190-520)	2650	armour layer
10-60 kg ND	260 (190-350)	2650	armour layer
3-9 inch ND	125 (75-230)	2650	filter layer or single layer
3-9 inch HD	125 (75-230)	3000	filter layer or single layer
2-8 inch ND	100 (50-200)	2650	filter layer
2-8 inch HD	100 (50-200)	3050	filter layer

Table 4.4.1 *Rock gradings (ND=normal density; HD=high density); rock sizes differ slightly for each quarry*

4.4.2 Other scour protections for structures on the seabed

Other types of protection materials are:

- block mattress (**Figure 4.4.6**) consisting of concrete blocks connected by nylon-rope or geotextile material; mostly used for river bank protection; very flexible; relatively small obstruction height (less edge scour); filter layer or geotextile underlayer is required against washing out of sand; precise installation without gaps is required (double layer with staggered pattern can also be used; underlayer can have smaller rock sizes to act as filter layer); special block-type mattresses can be made fit to the structure shape and planform (www.subcon.com; **Figure 4.4.7**);
- gabion mattress (**Figure 4.4.8**) consisting of steel nets filled with gravel, stones or rock; mostly used for pipeline cover; very flexible; small obstruction height (less edge scour); filter layer or geotextile underlayer is required against washing out of sand; precise installation without gaps is required (double layer can also be used);
- large mesh bags filled with rocks: nylon mesh bags with diameter of 2 to 3 m; filled with rocks of 50 to 200 kg; placed by crane.

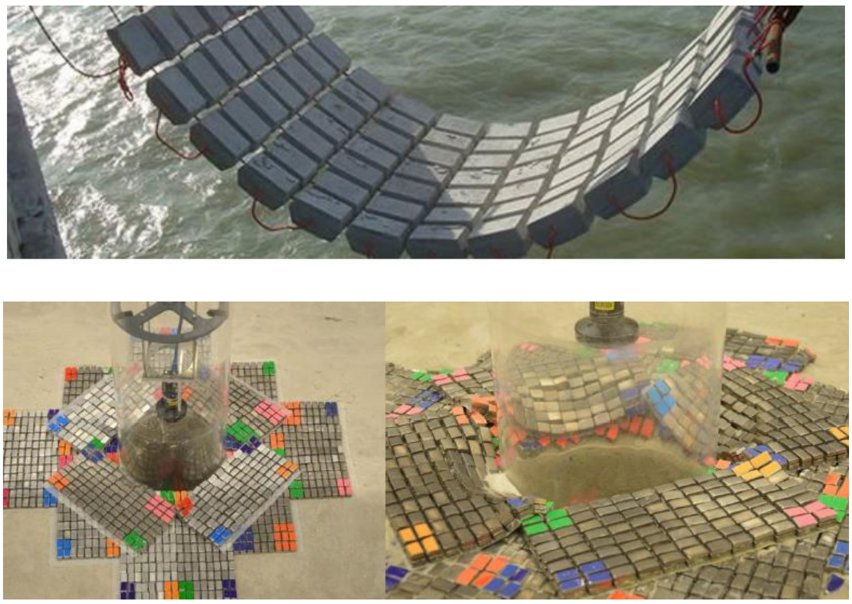


Figure 4.4.6 *Mattress of concrete blocks (Deltares 2017, 2020)*

Figure 4.4.7 *Mattress of bloks fit to structure (www.subcon.com)*

Figure 4.4.8 *Mattress of gabions as single or double layer (Deltares 2017, 2020)*

5 PRACTICAL DESIGN OF ROCK-TYPE BREAKWATERS, GROINS AND REVETMENTS

5.1 Types of structures and armouring

Rubble mound breakwaters are high- and low-crested, permeable structures of rock and crushed rock covered by a steep sloping outer armour layer of rock blocks or concrete units/blocks, see **Figures 5.1.1** and **5.1.2**. The outer slopes generally are relatively steep in the range between 1 to 1.5 and 1 to 2 (minimum construction costs.

A revetment (see **Figures 5.1.3, 5.1.4**) is a local armour protection layer at the toe of relatively small dunes or urban boulevards at the end of the beach against scour due to wave action during storm events. Revetments consist of closely-fitted rocks, stones, block-works, in-situ cast concrete slabs, underlying filter layer and toe protections. To reduce scour due to wave action and wave reflection at the toe of the structure, the slope of the revetment should not be steeper than 1 to 2. Preferably, a mild slope should be used. The crest of the revetments should be well above the highest storm surge level resulting in a crest level at +5 m above mean sea level along open coasts and upto +7 m at locations with extreme surge levels.
Figure 5.1.5 shows a massive coastal revetment of 18 m in deep water protecting a large-scale land reclamation near Lagos in Nigeria (EKO Atlantic city). It consists of a rock core, several layers of rock and two layers of armor units. Basically, it is a breakwater with armor units on the sea side and sand fill (reclamation) on the land side.

Groins (see **Figure 5.1.3**) are low-crested structures approximately perpendicular to the shore and constructed to reduce beach erosion by deflecting nearshore currents. Seabed protection may be necessary in case of strong tidal currents passing the head of the structure.


Figure 5.1.1 *Examples of rock (left) and concrete armour blocks (Dolos and X-blocks)*

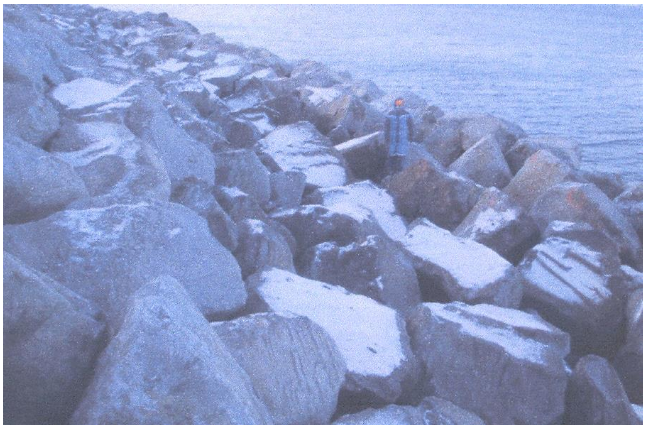



Figure 5.1.2 *Icelandic-type berm breakers at Sirevag (Norway) and Husavik (NW Iceland)*

Figure 5.1.3 *Coastal revetments and groins of orderly rocks at Caparica, South of Lisbon, Portugal*

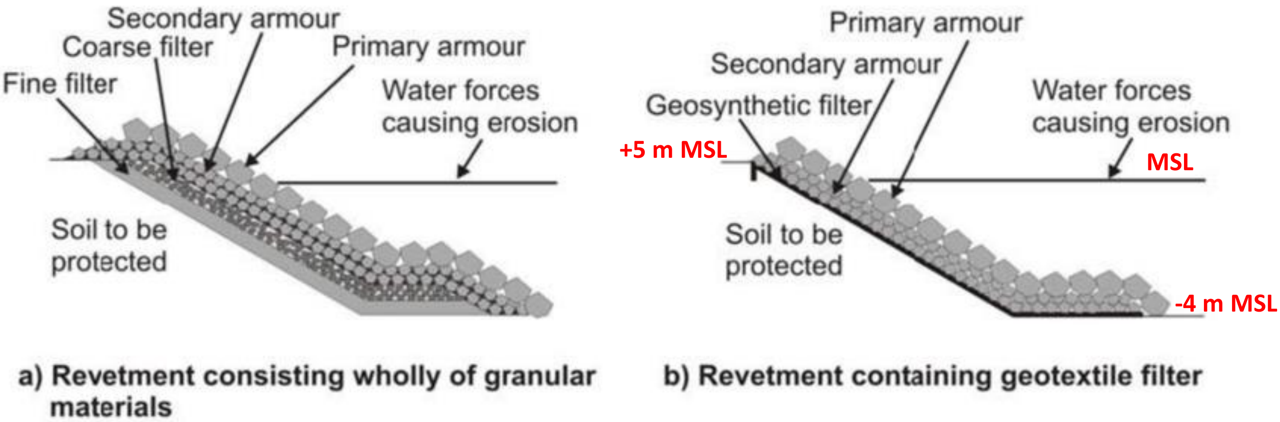


Figure 5.1.4 *Cross-sections of coastal revetments*

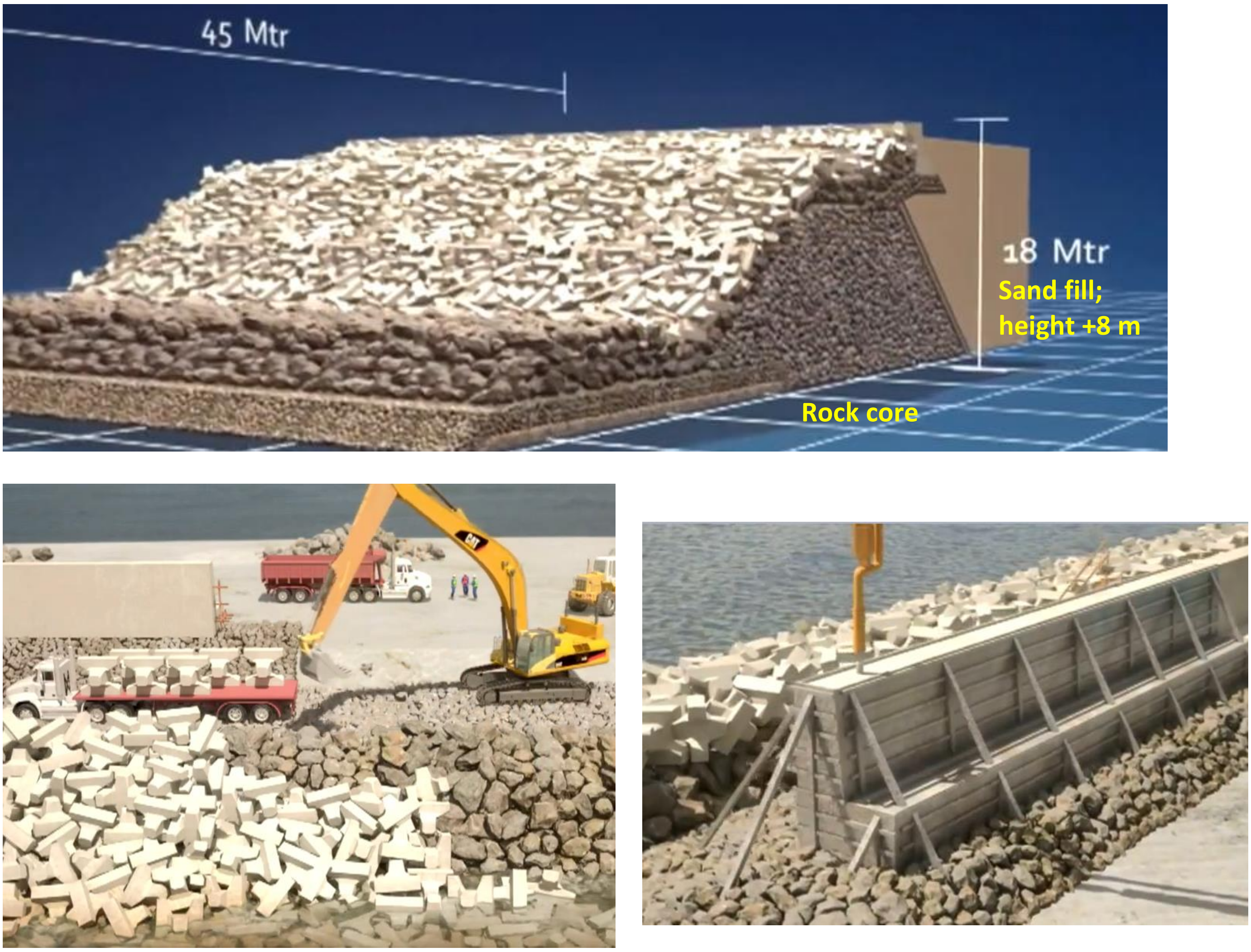


Figure 5.1.5 *Coastal revetment Lagos wall protecting EKO Atlantic City, Lagos, Nigeria*

Two major types of breakwaters can be distinguished (see **Table 5.1.1**):

- conventional, high-crested and low-crested or fully submerged breakwaters of rocks or concrete units;
- berm breakwaters of large rocks; massive high-crested breakwaters also known as Icelandic berm breakwaters, which can withstand incoming significant wave heights at the toe ($H_{s,toe}$) in the range of 3 to 6 m without significant movement of individual rock blocks.

Types of breakwaters		**Waterdepth= 4 to 6 m** $H_{s,toe}$**= 2 to 3.5 m**	**Waterdepth= 6 to 10 m** $H_{s,toe}$**= 3.5 to 6 m**
Conventional breakwaters	High-crested	Rock Class III	Interlocking concrete blocks, Orderly placed cubes (1 layer)
	Low-crested	Rock Class III	Orderly placed cubes (1 layer)
	Submerged	Rock Class III and IV	Randomly placed cubes (2 layers)
Berm breakwaters		*Not used in low wave conditions*	Orderly placed rocks, Class I and II (only, if special equipment is available)

Table 5.1.1 *Classification of breakwaters*

Sigurdarson and Van der Meer (2013) have proposed a classification of breakwaters (see **Table 5.1.2**) based on the stability of the armour layer. The recession (Rec) at the edge of the berm (see also **Figure 5.3.1**) and the damage (S_d) are also shown. The damage S_d is defined as $S_d = A_e/(D_{n50})^2$ with $A_e = L_e$ x 1= damage or erosion area near the water level per unit length of structure, L_e = damage or erosion length along the slope of the structure.

Behaviour of outer armour layer	Recession at edge of berm Rec	Damage S_d	$H_s/(\Delta D_{n50})$	D_{n50} for H_s= 3 m	D_{n50} for H_s= 5 m
Hardly reshaping Icelandic armoured HR-IC	0.5 to 2 D_{n50}	2-8	1.7 to 2	0.9-1.1 m	1.5-1.8 m
Partly reshaping Icelandic armoured PR-IC	1 to 5 D_{n50}	10-20	2 to 2.5	0.7-0.9 m	1.2-1.5 m
Partly reshaping mass armoured PR-MA	1 to 5 D_{n50}	10-20	2 to 2.5	0.7-0.9 m	1.2-1.5 m
Fully reshaping mass armoured FR-MA	3 to 10 D_{n50}		2.5 to 3	0.6-0.7 m	1.0-1.2 m

Table 5.1.2 *Classification of berm breakwaters*

5.2 Cross-section of conventional breakwaters

5.2.1 Definitions

The cross-section of a conventional breakwater (**Figures 5.2.1** and **5.2.2**) has a trapezoidal shape and mostly consists of:

- **crest**: crest width is often determined by the dimensions of the superstructure (vertical wall, crown wall/cap, access road, construction road), the minimum crest width is $B_c = 3D_{n50}$;
- **outer sloping armour layer** consisting of rock or concrete blocks (see **Table 5.1.1**) to resist and dissipate the wave energy (through breaking and friction) without significant movement; rock/stone classes and concrete armour units are shown in **Table 5.2.2, Figures 5.2.1** and **5.2.2;**
- **core** of quarried rock fill (crushed rock);
- **underlayers** (foundation/filter layers) of crushed rock;
- **geotextile layer** to prevent the erosion of sand from the soil surface.

Geometrical definitions are shown in **Figure 5.2.1**:

- crest height (R_c) is the distance between the still water level and the crestpoint where overtopping water cannot flow back to the sea through the permeable armour layer (also known as freeboard);
- armour crest height (A_c) is the distance between the still water level and the crest of the outer armour layer;
- armour slope is the slope of the outer armour layer (generally between 1 to 1.5 and 1 to 3);
- toe berm is the horizontal transition between the foreshore and the armour slope with a length of maximum 0.25 L_i (with L_i = incident wave length); toe berm should support the armour layer and reduce the undermining effect of toe scour.

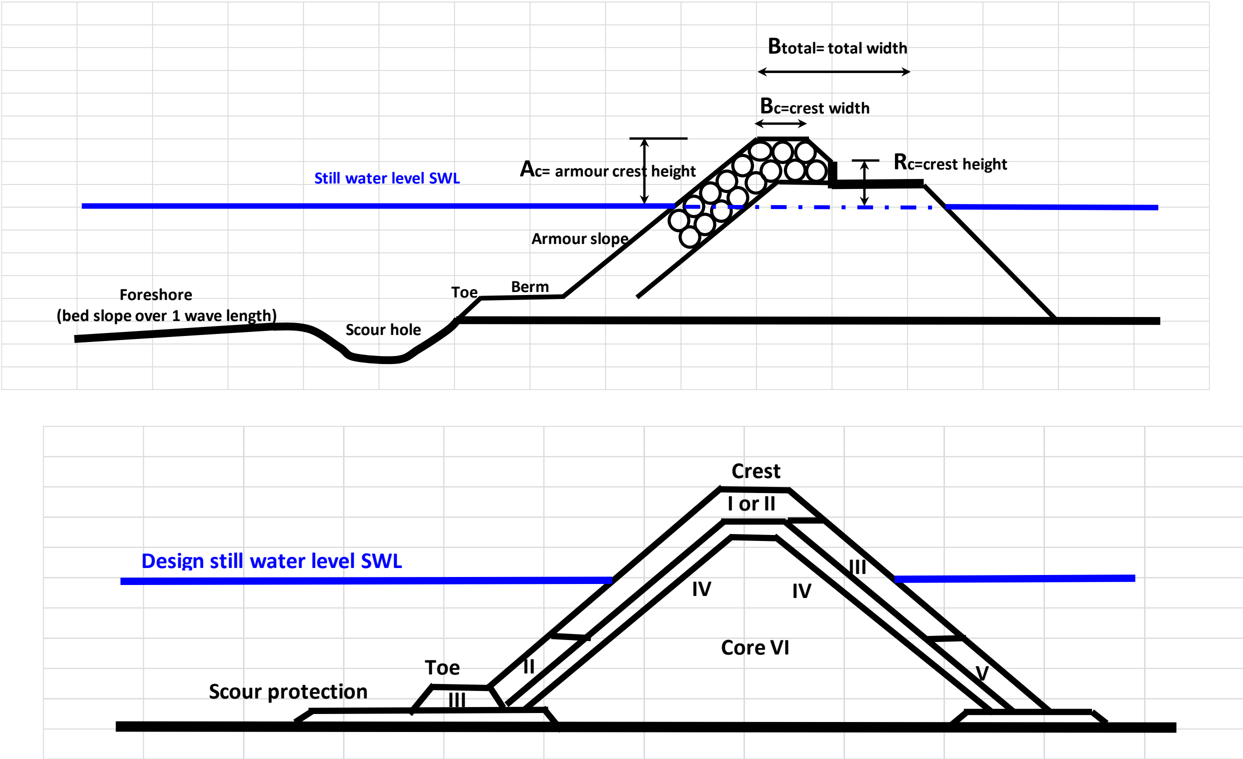
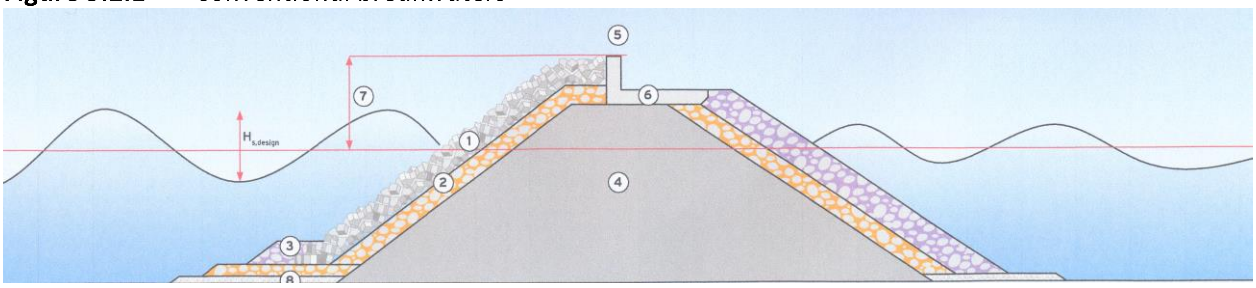


Figure 5.2.1 *Conventional breakwaters*



1= armour units (Xblocs), 2=underlayer, 3=toe, 4=core, 5=crest, 6=crown wall, 7=freeboard, 8=filter layer

Figure 5.2.2 *Conventional breakwater with concrete armour units (Xblocs)*

5.2.2 Crest height

The crest may have a superstucture or crown wall (= cap block plus vertical wave wall):

- often an access road is required for construction and maintenance;
- a vertical wall may reduce the wave overtopping and protect the lee side;
- a superstructure is a rigid element in a flexible structure; uneven settlement of the breakwater may lead to problems;
- a vertical wall will lead to larger wave impact forces.

Significant cost savings can be made if the crest height is reduced as much as possible.
The crest level should allow for post-construction settlements and mean sea level rise.

The water depth at the toe of a breakwater generally is in the range of 5 to 10 m during storm events. Hence, the maximum wave height at the toe is of the order of 3 to 6 m with wave periods in the range of 10 to 15 s. At open ocean coasts large swell waves (3 to 6 m) may also occur with wave periods in the range of 15 to 20 s.

The surf similarity parameter ξ is relatively large for steep, rough slopes.
Assuming a surface slope of 1 to 2 ($\tan\alpha = 0.5$) and a wave steepness in the range of s = 0.02 to 0.05, the surf simularity partameter is in the range of $\xi = (\tan\alpha)/s^{0.5} = 2$ to 5.

The crest height of rough, sloping, permeable breakwaters is primarily based on the allowable wave overtopping and wave transmission affecting the rear side of the structure and the harbour basin with moored ships. The allowable overtopping rates are in the range of 1 to 10 l/m/s. A value of 10 l/m/s is a rather large value (about 5% overtopping waves).

5.2.3 Armour layer, underlayers and core

The armour layer is supported by the core, the underlayers and the toe protection.
The core and toe generally are founded on granular filter layers to prevent the erosion of finer grains from the sand bed. Filter layers are not required, if a geotextile filter fabric is used. Special easy sinkable geotextiles for underwater are available (www.geotextile.com).
One or more filter/underlayers are used to obtain a smooth transition form the core to the largest rock sizes or concrete units of the armour layer; otherwise finer stones from lower layers may be washed through the upper layer during storm events.

The outer armour slope consists of a single or double layer of armour units, depending on type of armour units (rocks or concrete units) and the placement method (randomly or interlocking), see **Table 5.2.1**. Generally, randomly placed rock units are placed in a double layer, whereas a single layer of highly interlocking concrete units can be used.
Stability values are given in Section 3.3.

Types of units	Placement and interlocking	Number of layers	Slope of armour layer	Size D_{n50}	Relative volume of concrete
Rocks	Randomly placed (no interlocking) Ordely placed (some interlocking)	2 2	1 to 1.5		
Rocks (quarry run)	Randomly placed (no interlocking)		1 to 3 (1 to 2 asphalt bonding)		
Cubes (strong units)	Randomly placed (no interlocking) Orderly placed (no interlocking)	2 1 (only for high crest)	1 to 1.5	D	220%
Tetrapods (weak legs)	Randomly placed (fair interlocking)	2	1 to 1.5	0.65D	210%
Accropodes (strong units)	Strict pattern (high interlocking)	1	1 to 1.33	0.7D	100%
Core Locs (strong units)	Strict patterm (high interlocking)	1	1 to 1.33	0.8D	80%
Xblocs (strong units)	Strict pattern (high interlocking)	1	1 to 1.33	0.8D	80%

D= height of unit

Table 5.2.1 *Placement and interlocking characteristics of armour units*

Rock/Stone size classes	Mass M_{50} (tonnes and kg)	Size D_{n50} (m)	Size grading D_{n85}/D_{n15}
I	10 - 30 tonnes	> 1.55	
II	6 - 10 tonnes	1.30 - 1.55	1.2 (narrow grading)
III	3 - 6 tonnes	1.05 - 1.55	1.5 (narrow grading)
IV	1 - 3 tonnes	0.70 - 1.05	1.5 (medium grading)
V	0.3 - 1 tonnes	0.45 - 0.70	1.5 (medium grading)
Quarry run			
VIa	60 - 300 kg (boulderstones)	0.30 - 0.45	1.5 (medium grading)
VIb	10 - 60 kg (large cobblestones	0.15 - 0.30	2.0 (wide grading)
VIc	0.1 - 1 kg (fines; crushed stones)	0.03 - 0.07	2.5 (wide grading)

$D_{n50} = (M_{50}/\rho_s)^{1/3}$ with ρ_s= 2700 kg/m^3 (rock/stone density)

Table 5.2.2A *Rock and stone size classes based on Rock manual*

Rock/Stone size classes	Mass M_{50} (tonnes and kg)	Size D_{n50} (m)	Size grading D_{n85}/D_{n15}
I	20 - 30 tonnes	1.95 - 2.25	1.2 (narrow grading)
II	10 - 20 tonnes	1.55 - 1.95	1.3 (narrow grading)
III	4 - 10 tonnes	1.15 - 1.55	1.4 (medium grading)
IV	1 - 4 tonnes	0.70 - 1.15	1.6 (medium grading)
V	0.4 - 1 tonnes	0.50 - 0.70	1.4 (medium grading)
Quarry run			
VIa	70 - 400 kg (boulderstones)	0.30 - 0.50	1.7 (wide grading)
VIb	3 - 70 kg (large cobblestones	0.10 - 0.30	3.0 (wide grading)
VIc	0.3 - 3 kg (small cobblestones)	0.05 - 0.10	2.0 (wide grading)

$D_{n50} = (M_{50}/\rho_s)^{1/3}$ with ρ_s= 2700 kg/m^3 (rock/stone density)

Table 5.2.2B *Rock and stone size classes based on Sigurdarson et al. 2001*

Type and grading	Size D_{n50} (m)	Layer thickness (m)
30-60 mm	0.037	0.2 minimum
40-100 mm	0.063	0.2 minimum
50-150 mm	0.089	0.2 minimum
80-200 mm	0.126	0.2 minimum
5-40 kg	0.19	0.29
10-60 kg	0.24	0.36
40-200 kg	0.35	0.53
60-300 kg	0.41	0.62
0.3-1 ton	0.63	0.95
1-3 ton	0.9	1.35
3-6 ton	1.18	1.77
6-10 ton	1.44	2.16

$D_{n50} = (M_{50}/\rho_s)^{1/3}$ with ρ_s= 2700 kg/m^3 (rock/stone density)

Table 5.2.2C *Rock and stone size classes based on Rijkswaterstaat 2000 (CUR 197)*

Rocks

Randomly placed rocks in a double layer are mostly used in conditions with incoming waves ($H_{s,toe}$) less than 3 m. Larger waves require rock sizes (>10 tonnes) which cannot be handled and/or may not be available in most countries. Interlocking concrete units are then much more attractive.
The highest stability is obtained, if the rocks/stones are as much as possible paced in an interlocking arrangement (Icelandic berm breakwaters).

Rock size classes are given in **Tables 5.2.2A,B,C**. Generally, rocks are described/identified by their weight range (for example: 3-6 tonnes). In developing countries the maximum weight of rocks that can be handeld (transportation) is of the order of 5 to 10 tonnes (Class III rocks).

The grading of the stones can be given by D_{max}/D_{min} (see **Table 5.2.2**) or by D_{85}/D_{15}, with D_{15}, D_{85} = size for which 15% and 85% is smaller. The grading also depends on the specifications of the quarry. The rocks/stones of the outer layer should have a narrow grading to obtain a stable structure with minimum movement. The core may have a relatively wide grading (quarry run).

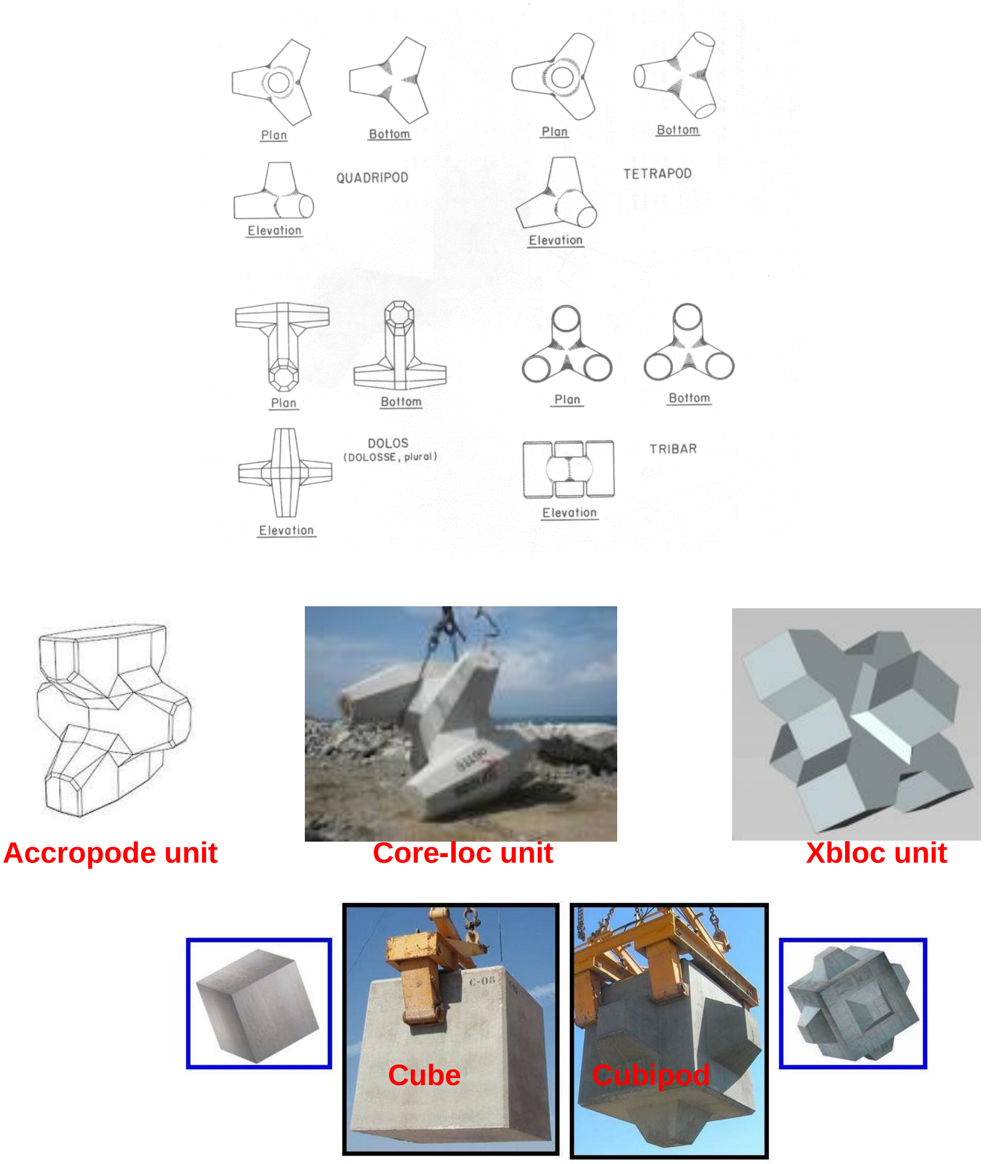


Figure 5.2.3 *Example of concrete armour units*

Figure 5.2.4 *Desintegrated concrete cubes (left) and new cubes (right); IJmuiden, The Netherlands*

Concrete interlocking units

A special class of modern concrete units is the class of interlocking units in a single layer (less construction cost). Various types of interlocking concrete armour units are shown in **Figure 5.2.3**.
A single layer of concrete units with high interlocking offers a reliable solution in conditions with large waves (> 3 m), because the units act as an integral layer whereas randomly placed units act as a layer of individual units. Highly interlocking concrete units in a single layer can be placed at a slope of 1 to 1.5. Examples are: Accropodes, X-blocs, Core-locs. Tetrapods (with 4 legs) only have a fair degree of interlocking and are generally placed in a double layer.

Laboratory tests have shown that high-interlocking concrete units in a single layer are stable to very high wave heights (N_{cr}-values up to ≅ 3.5). But as soon as damage starts to develop, a rather sudden failure of the whole structure does occur. This can be overcome by using a large safety factor (1.5) to compensate for errors in the design parameters.
A weak spot at breakwaters consisting of interlocking concrete units in a single layer generally is the transition from the slope to the horizontal crest. Special measures/units are required to prevent the presence of gaps, where relatively large impact forces can occur. Placement (in a strict pattern) under water may be problematic if visibility is low.

Concrete mass units (Cubes and Cubipods)

Various types of concrete mass armour units (cubes/cubipods) are shown in **Figures 5.2.3** and **5.2.4**.
The stability of these types of armour units is mainly based on heavy mass and friction between the side planes. Generally, non-interlocking mass units are randomly placed in a double layer at a slope of 1 to 2. Minor damage and rearrangement of the units in a double layer is acceptable.
Stability studies (Van Gent and Luis, 2013) have shown that simple straight cubes in a single layer at a slope of 1 to 1.5 are feasible and economically competitive with interlocking concrete units. Cubes are easy to produce, to place (also in low visibility underwater conditions) and to maintain. Therefore, concrete cubes (low or high-density concrete) in a single layer are rather attractive for developing countries Concrete cubes in a single layer should be placed in a regular pattern (open space of 20% to 30%). A danger is the settlement of cubes which may lead to relatively large open spaces locally and exposure of the underlayer, see **Figure 3.3.5**.
Settlement of the cubes along the slope due to wave action may also lead to relatively large gaps near the crest (transition from slope to horizontal), which are weak spots as regards wave impacts on the cubes of the crest.
Therefore, the crest of a breakwater with cubes should be relatively high (overtopping < 10%). The mass of the (filter) underlayer of rocks should be about 10% of the mass of the cubes to obtain a relatively smooth surface for placement of the cubes.
The safety factor of cubes in a single layer layer should be relatively large (1.5), as damage is not acceptable.

In the case of a low crest, the horizontal cubes on the crest will experience relatively large impact forces due to a change in slope (gap) from the outer layer to the horizontal crest, which may cause damage. As the surface of orderly placed cubes in a single layer is rather smooth, the wave overtopping rate is relatively large for a low-crested-structure. This can be reduced somewhat by constructing using a roughness element on the surface of each cube.

A structure of concrete units in a single layer should always be compared as regards construction costs to a double layer of rock units, if rocks are locally available.
Disadvantages of cubes in a single layer are:

- more overtopping due to relatively smooth surface;
- presence of gaps at the transition between armour slope section and horizontal crest.

Cubes in a double layer can be used for relatively low-crested structures. The cubes in a double layer should be placed randomly (which often is difficult as the cubes tend to lie face to face) to avoid the presence of gaps between the slope section and the horizontal crest. Cubes in a double layer can also be used for submerged breakwaters.
Cubes or cubipods in a double layer can also be used for massive breakwaters in high-wave conditons.

Figure 5.2.4 shows desintegrated concrete cubes (units of 20, 30 and 45 tonnes) and new cubes of the 2 km long breakwater at IJmuiden (North Sea), The Netherlands. The concrete cubes tend desintegrate on the long term after 30 to 50 years due to cracking under severe wave action in relatively deep water (10 m). As a result the breakwater has got an aesthetic rock-type appearance. Regular maintenance is required using new cubes.

Cubes with small pods on the side planes are known as cubipods, see **Figure 5.2.3**. They can only be placed randomly in a single or double layer at a slope of 1 to 1.5 or 1 to 2. Weights in the range of 15 to 35 tonnes (size of about 2.5 m) in a single layer have been used in the long breakwater of Punta Langosteira (Spain). Wave heights are as large as 7 m (T_p= 18 s).
Advantages of cubipods (compared to straight cubes) are:

- more friction between the units and underlayer;
- less overtopping due to presence of pods;
- both for low-crested and high-crested structures;
- less volume (5% to 15%) of concrete per unit area due to larger spaces between the units.

Burcharth et al. (2010) have compared the stability of cubes and cubipods:

- 2D steep waves: cubes and cubipods have about the same stability;
- 3D medium steep waves: cubipods are slightly more stable than cubes;
- 3D long waves: cubipods are significantly more stable than cubes.

Example breakwater of cubes

Most harbour breakwaters in The Netherlands are made of cubes randomly placed in a double layer (Hoek van Holland, Scheveningen, IJmuiden). The long (4 km) northern jetty at Hoek van Holland (approach to Rotterdam harbour) consists of cubes (weight of 20 to 30 tonnes) without a road at the crest. This reduces the crest width. The jetty was made by using a barge with crane. A barge has a large loading capacity, which is more economic for a long jetty than transportation of individual blocks by a crest road.
Figure 5.2.5 shows a plan view of the breakerwater of Scheveningen fishery harbour along the North Sea coast of The Netherlands. The breakwater is made of cubes, which are placed randomly in a double layer (slope in the range of 1 to 1.5 and 1 to 2). The size of the cubes ($D_{n,50}$) is about 2 m.
The length of the south breakwater is about 600 m beyond the low water line at the beach.

The crest width (including road) is about 10 m.
The crest level is about 7 m above mean sea level (MSL).

The water depth at the entrance of the harbour is about 7 m to MSL. The tidal range is about 2 m.
The most extreme event (return period of 100 years) is a storm with a maximum storm surge level of 4 m (including tide) above MSL yielding a maximum water depth of about 11 m at the head of the breakwater. The maximum wave height ($H_{s,toe}$) is estimated to be in the range of 5 to 7 m with wave periods (T_p) in the range of 10 to 14 s. The computed results based on **ARMOUR.xls** are given in **Table 5.2.3**.
The computed cube size (randomly in double layer) is in the range of 1.5 to 2.2 m. The cube size is 2 m.

Cube size	**Armour slope tan(α)**	**$H_{s,toe}$ = 5 m T_p = 10 s N_{od} = 1, N_w = 2500**	**$H_{s,toe}$ = 6 m T_p = 12 s N_{od} = 1, N_w = 2500**	**$H_{s,toe}$ = 7 m T_p = 14 s N_{od} = 1, N_w = 2500**
$D_{n,50}$ (m)	0.67 (1 to 1.5)	1.55 m	1.80 m	2.05 m
$D_{n,50}$ (m)	0.5 (1 to 2)	1.65 m	1.90 m	2.20 m

Table 5.2.3 *Computed cube armour size for Scheveningen harbour*

Figure 5.2.5 *Breakwater of randomly placed cubes, fischery harbour Scheveningen, The Netherlands*

5.2.4 Armour sizes

To determine the rock/stone sizes, the wave climate along the full length of the breakwater should be known, from deeper to shallower water. Most practical, the breakwater should be divided into a number of subsections. The spreadsheet-model **ARMOUR.xls** (based on the equations of Chapter 2 and 3) can be used to determine the wave overtopping rates and the armour sizes. Examples are given in **Section 3.3**.

In deep water with non-breaking incoming waves, the outer armour layer of the largest rocks/stones should be extended to a distance of about $1.5H_{s,toe}$ below the still water level.
In shallow water with breaking incoming waves (depth-limited conditions), the outer armour layer should be extended to the toe.

The thickness of the layers is given by: $\delta= n\ k_\delta\ D_{n50}$ (5.2.1)

The number of units per m² is given by: $N_a= n\ k_\delta\ (1-p_v)/(D_{n50})^2$ (5.2.2)

The packing density is: $\phi = N_a/(D_{n,50})^2$ (5.2.3)

with:

δ = layer thickness (m);
n = number of layers (1 to 3);
k_δ = layer thickness coefficient (see **Table 5.2.4**);
D_{n50} = stone size (m);
N_a = number of stones per m²;
P_v = volumetric porosity;
ϕ = packing density.

Based on general practice, the stone mass/size of the underlayers and core is related to the mass/size of the stones of the outer armour layer, as follows:

First underlayer Front (Class IV): $M_{50,underlayer\ 1} \cong 0.1$ to $0.2\ M_{50,outerlayer}$
$D_{n50,underlayer\ 1} \cong 0.4$ to $0.6\ D_{n50,outerlayer}$

Second underlayer Front (Class V): $M_{50,underlayer\ 2} \cong 0.02$ to $0.04\ M_{50,outerlayer}$
$D_{n50,underlayer\ 2} \cong 0.25$ to $0.35\ D_{n50,outerlayer}$

Core (Class VI Quarry run): $M_{50,core} \cong 0.001$ to $0.01\ M_{50,outerlayer}$
$D_{n50,core} \cong 0.1$ to $0.2\ D_{n50,outerlayer}$

First underlayer Rear Side (Class V): $M_{50,underlayer\ 1} \cong 0.1$ to $0.2\ M_{50,rear}$
$D_{n50,underlayer\ 1} \cong 0.4$ to $0.6\ D_{n50,rear}$

with:
$D_{n,50}$ = nominal diameter of rock or concrete unit= $(M_{50}/\rho_r)^{1/3}$ and ρ_r = density of rock/concrete (kg/m³).

Types of armour units	**k_δ - coefficient**	**P_v -volumetric porosity**
Smooth, uniform rock, n=2	1.02	0.38
Rough, uniform rock, n=2	1.0	0.37
Rough, uniform rock, n>3	1.0	0.40
Graded rock	1 to 1.5 depending on grading	0.37
Cubes	1.1	0.47
Tetrapods	1.04	0.5
Dolosse	0.94	0.56

Table 5.2.4 *Coefficients (SPM, 1984)*

5.2.5 Seaward end of breakwater

A round or circular seaward end of a shore-connected breakwater is termed the roundhead. This end section experiences severe wave forces during storm events. The most severe attack is at the leeward far end quarter section of the roundhead. The primary armour layer should be extended to this section. The slope of the roundhead can have a milder slope to reduce the wave forces. Furthermore, the crest level can be somewhat reduced. Laboratory model tests are generally required to finalize the geometry and dimensions.
Sometimes, a caisson-type structure is constructed at the end of a breakwater to install ship navigation aids (lights, radar).
Scour protection due to both currents and waves should be studied in detail.

5.3 Cross-section of berm breakwaters

Berm breakwaters (Sigurdarson et al., 2010) are mainly used in severe wave climates with relatively large incoming waves (Iceland, Norway). The berm is mainly designed to create a breakwater with a high wave energy absorption, to minimise wave reflection and wave overtopping (and thus wave transmission). Good interlocking of carefully placed large rocks at the front and at the edge of the berm is of major importance, see zone with class I rocks of **Figure 5.3.2**.

The stability of the outer armour layer strongly depends on the size and grading of the rocks used in the armour layer. Armour layers consisting of a relatively wide range of rocks (wide graded) may deform or reshape into an S-type profile (see **Figure 5.3.1**) during design conditions, as has been observed at many built breakwaters. Wave overtopping and wave transmission will increase after reshaping of the armour layer resulting in reduced functioning of the breakwater. Berm breakwaters can only be made in countries where large rock sizes (> 10 ton) can be produced and transported using special equipment (Iceland, Norway, etc.). Icelandic berm breakwaters are designed to keep their form without any reshaping, see **Figure 5.3.1**. These latter breakwaters are built up of several narrowly graded armour classes with the largest armour classes placed at the most exposed locations within the breakwater cross-section. The rock units have an interlocking arrangement at the front of the berm. The largest rock size is up to 2.5 m with a weight of about 40 tons. An armour layer of these types of rocks has a large porosity/permeability which increases the stability of the structure and decreases the wave overtopping rate and wave reflection. The wave transmission due to overtopping decreases significantly, but the wave transmission due to wave penetration through the structure may increase somewhat.

The construction of Icelandic breakwaters requires careful quarry selection of rock units in narrow graded size classes. Examples of Icelandic berm breakwaters with non reshaping armour layers including the required rock size classes (between I = largest size and V = smallest size) are shown in **Figures 5.3.2** and **5.3.3**.

The outer slope mostly is 1 to 1.5. Steeper slopes of 1.3 have also been used, but these slopes are significantly less stable.

The berm level mostly is high-crested: at about 0.5 to $1H_{s,toe}$ above the design SWL. A low-crested berm just above the design level is less effective (more damage at the edge of the berm, Andersen et al., 2012).

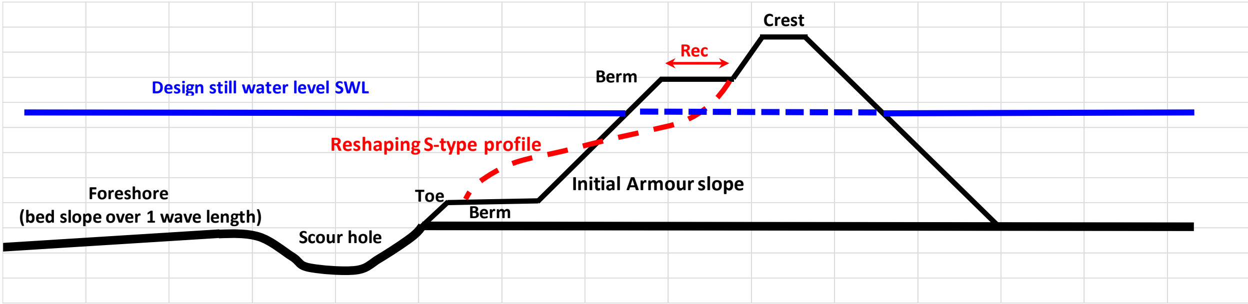


Figure 5.3.1 *Berm breakwater with fully reshaping profile (FR-MA)*

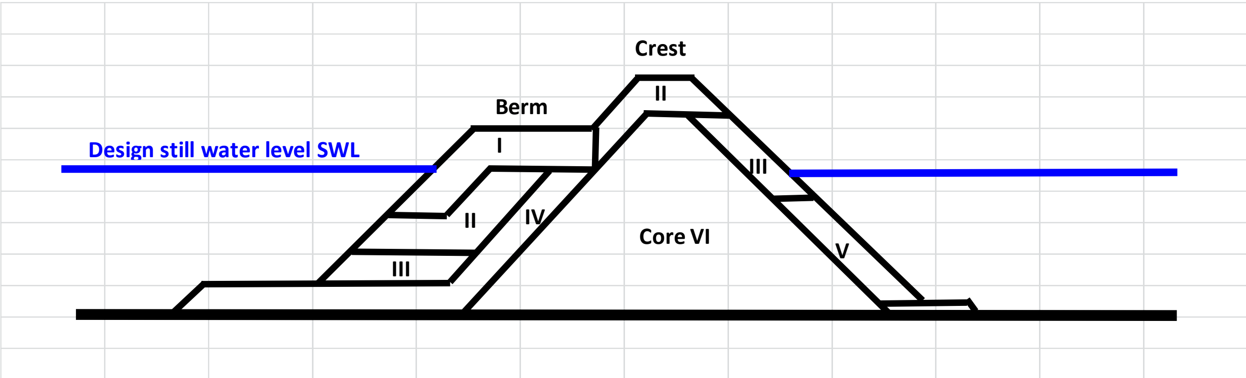


Figure 5.3.2 *Berm breakwater with none reshaping armour layer (Rock size classes, see Table 5.2.2)*

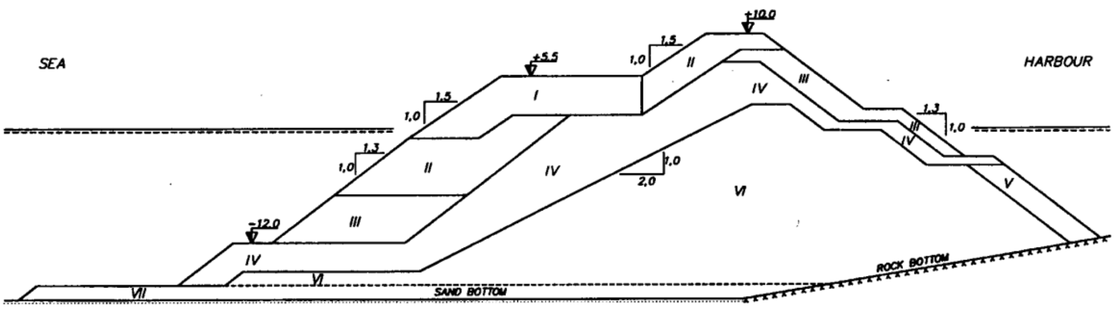
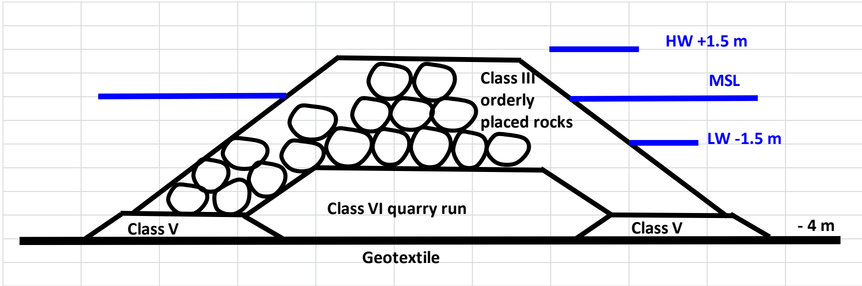


Figure 5.3.3 *Cross-section of Sirevag berm breakwater, Norway (Sigurdarson et al., 2001)*
($H_{s,toe}$ = 5 to 7 m, T_p = 14 s for 100 years return period; length = 500 m)



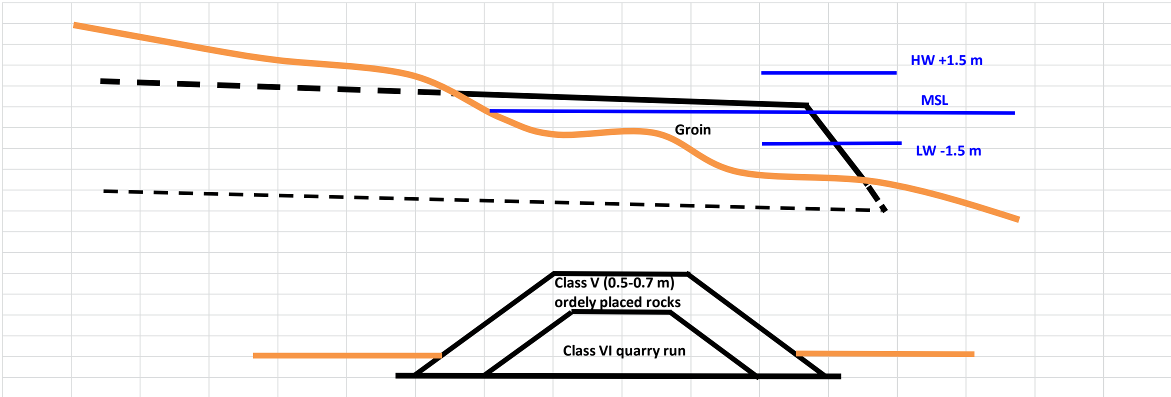


Figure 5.4.1 *Examples of cross-sections of low-crested breakwater and groin*

5.4 Cross-section of low-crested breakwaters

The armour layer of submerged or emerged, low-crested structures in shallow water mostly is extended to the toe on both sides of the structure for practical reasons, see **Figure 5.4.1**.
A double layer of relatively large rocks in nearshore shallow water may require that part of the structure needs to be placed below the bed level in a dredged trench. The $D_{n,50}$ can be reduced by using a milder slope than 1 to 2 in the surf zone. To reduce the armour size, the rocks of the armour layer of groins should be placed orderly above LW and impregnated with bituminous materials. The armour size can be computed by using the spreadsheet-model **ARMOUR.xls**. Examples are given in **Section 3.3**.

5.5 Settlement of armour units and subsoil

Consolidation of the subsoil and settlement of the rubble mound during and after construction may cause lowering of the crest. This must be compensated by a certain overheight during the design phase. The required overheight is relatively small, if the structure is built out from the beach/shore by land-based equipment moving over the structure during construction. Crest lowerings up to 0.5 m in 5 to 10 years have been reported for detached breakwaters along the Italian coast (Burcharth et al. 2006).
Another longterm problem is the reduction of the porosity of the structure due to sedimentation, flora and fauna, which will lead to a larger reflectivity of waves and less wave absorption and thus to more damage in later years (Burcharth et al. 2006).

Failure of a breakwater can be caused by geotechnical factors such as slope failure due to toe scour, foundation failure due to erosion (washing out) of sediment particles from the bed through the core materials and due to seismic action. Breakwaters in shallow water (surf zone with breaking waves) may show significant

settlements (0.5 m) during and after construction (within 1 to 5 years) due to consolidation of the rubble structure and scour of the bed (Burcharth et al. 2006) requiring overdimensioning of the design.
Geotechnical investigations based on test borings are required to know the soil properties (non-permeable fine cohesive sediments or permeable sands and gravels), bearing capacity, risks of liquefaction, expected settlements and long term subsidence.
Geotechnical design is based on stability and deformation analysis of the subsoil under various loads: permanent loads, variable wave impact loads, accidental loads (ship collision) and seismic loads.
A berm breakwater has been built in Iceland on very weak soil consisting of more than 20 m of thick soft organic silty soil (Sigurdarson et al. 1999, 2001). The potential for liquefaction and settlement under various loads was studied in detail. The breakwater was constructed in stages to allow the construction to settle over significant vertical distance. The observed settlement was 1.3 m six months after construction.
Geotextile filter material is used to prevent the erosion of relatively fine sediment particles from the subsoil through the relatively coarse core materials (0.1 to 0.3 m). Various types of geotextiles are available (www.geofabrics.com; www.geotextile.com; www.usfabricsinc.com).

6 DESIGN GUIDELINES FOR GRANULAR AND GEOTEXTILE FILTERS

6.1 Granular filters

The classical criteria of Terzaghi for granual filters can be expressed as (Giroud 2010):

Permeability criterion $D_{15,filter} \geq D_{15,soil}$ (6.1.1a)
Retention criterion $D_{15,filter} \leq D_{85,soil}$ (6.1.1b)

Equation (6.1.1a) means that the D_{15} of the filter must not be too small.
Equation (6.1.1b) means that the D_{15} of the filter must not be too large. The filter should only retain large soil particles; the large soil particles accumulating in the filter lead to a decrease of the pores of the filter and so on creating the filter process, see also **Figure 6.1.1**.

The $D_{15,filter}$ is used, because the opening (pore) size of granular filter material is approximately equal to $0.2D_{15}$. Thus, particles with size < $0.2D_{15,filter}$ can pass through the filter.

Giroud (2010) shows that the permeability criterion is equivalent with:
$k_{filter} \geq 25\ k_{soil}$ (6.1.2)

with: k= permeability coefficient.
The thickness of a granular filter should be at least 0.25 m.

Giroud (2010) also shows that the retention criterion can be refined to:
$D_{15,filter} \leq 10\ D_{85,soil}$ for uniform soils (narrow grading) (6.1.3a)
$D_{15,filter} \leq 5\ D_{85,soil}$ for non-uniform soils (wide grading) (6.1.3b)

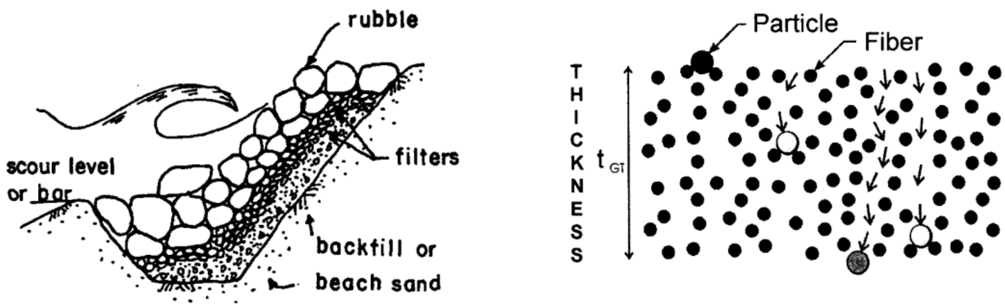
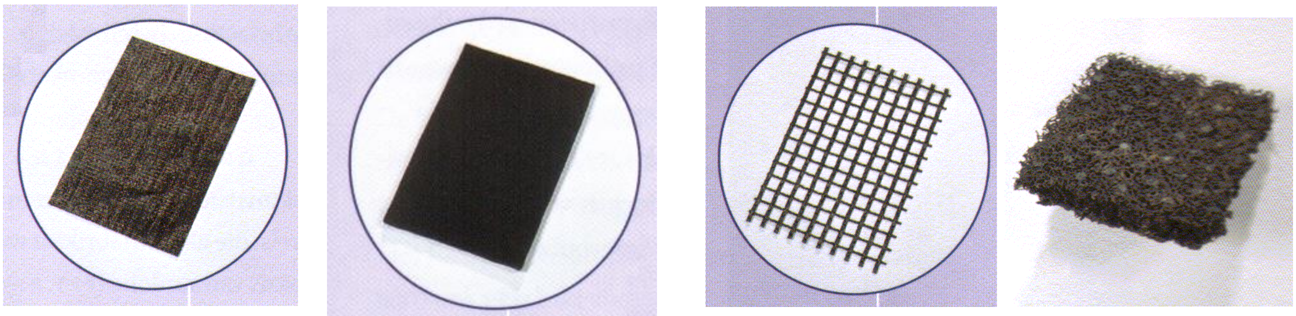


Figure 6.1.1 *Granular filter layers (left) and filter process through geotextile filter (right)*

6.2 Geotextile filters

Geotextiles of polypropylene or polyester materials can be divided into (see **Figure 6.2.1**):

- woven geotextiles (thin two-dimensional fabrics; thickness of 0.5 to 3 mm);
- non-woven geotextiles (thick three-dimensional fabrics; thickness of 10 to 20 mm);
- geo grids (thin two dimensional structures);
- geo-matrasses (three-dimensional structures filled with granular materials, 10 to 30 mm thick).



woven geotextile | non-woven geotextile | geogrid | geomattrass

Figure 6.2.1 *Examples of geotextile filter materials*

The filter criteria can be formulated, as follows (Giroud 2010):

$O_{filter} \leq 2\ D_{85,soil}$	for uniform soils	(6.1.4a)
$O_{filter} \leq\ D_{85,soil}$	for non-uniform soils	(6.1.4b)
$k_{filter} \geq\ k_{soil}$		(6.1.4c)
$A_R \geq\ 10\%$	for woven geotextiles (2D)	(6.1.4d)
$p_{filter} \geq\ 0.55$	for non-woven geotextiles (3D)	(6.1.4e)

with: O = opening size of filter, k= permeability, A_R = openingspercentage (ratio of area of all openings and total area) and p = porosity factor of filter.

Using $D_{15,filter} = 1\ D_{85,soil}$ for uniform soils may easily lead to clogging (blocking) of the filter.
Using $D_{15,filter} = 2\ D_{85,soil}$ for non-uniform soils may easily lead to piping (creation of small channels).

Thin 2D woven filters should be relatively open (openingspercentage > 10%).

The 3D non-woven filters should be sufficiently thick. A soil particle moving through a filter moves from constriction to another, following a filtration path, see **Figure 6.1.1**. The particle will be stopped or will pass depending on the constriction size. Based on analysis of Giroud (2010), a 3D filter should have at least 25 constrictions over its thickness. In practice, this means a thickness of about 20 to 30 mm. Granular filters are relatively thick (> 0.25 m) and have sufficient constrictions.

Geotextiles for underwater applications require ballast material (granular materials or reinforcing steel bars) for succesful placement underwater. Ballast material (steel bars) can be attached to the geotextile by using cable ties for small-scale structures. Furthermore, two lines (ropes) are laid on the upper side of the geotextile along its length (see **Figure 6.2.2**). The geotextile and lines are rolled onto a steel pole. The rolled geotextile panel can be lowered to the bottom from a barge. The leading edge of the geotextile is anchored at the bottom. The two lines are hauled from on board a barge. A layer of rock should be placed on the geotextile immediately to ballast it. Successive geotextiles should have an overlap of about 1 m.
Double layer geotextile matrasses with granular fill materials are also available for medium-scale structures. Various types of geotextiles are available (www.geofabrics.com; www.geotextiles.com; www.usfabricsinc.com).

Figure 6.2.3 shows a geotextile with traditional twigs, which are roped to the geotextile. The mats, which have been used for a massive breakwater at Ostend (Belgium), are about 20 m wide and 50 m long. The mats are towed to the sink location at high tide and ballasted with stones (10 to 300 kg) at slack tide.

Major marine contractors have ships and special equipment to install geotextiles under water and to precisely dump stones.

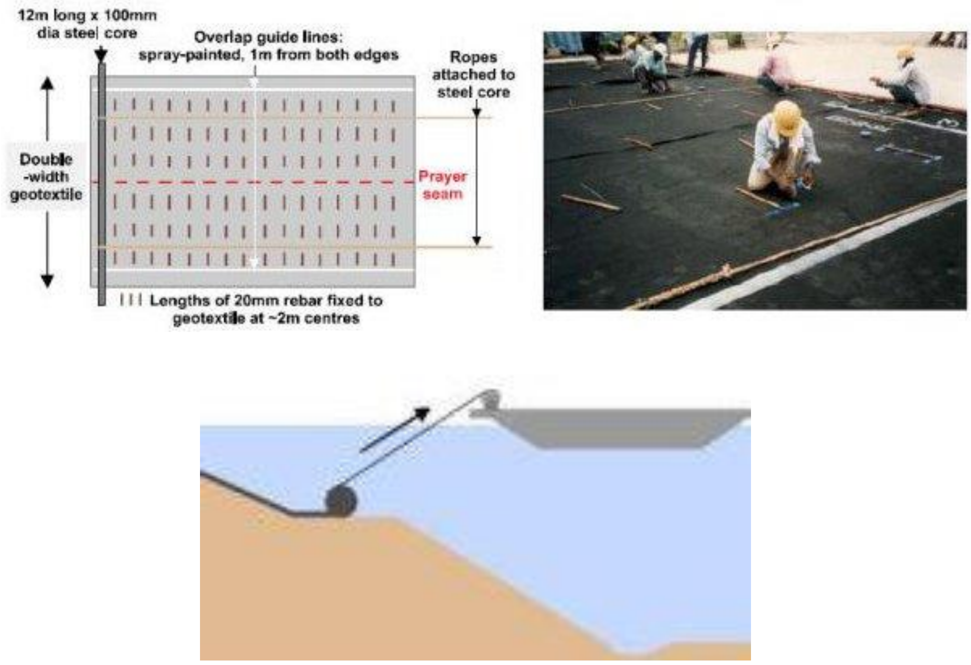


Figure 6.2.2 *Preparation of geotextile for underwater applications (www.usfabricsinc.com)*

Figure 6.2.3 *Preparation of geotextile for massive breakwater*

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ANNEX I SPREADSHEET-MODEL ARMOUR.xls

The spreadsheet-model ARMOUR.xls can be used to compute the wave runup, wave overtopping rate, wave transmission and the armour size.
Two wave models are available to compute the nearshore wave height and wave angle based on the offshore data.

The excel 2010 file consists of 7 sheets, as follows:

- sheet 0: Overview;
- sheet 1: Seadikes and revetments;
- sheet 2: High-crested, low-crested (emerged and submerged) breakwaters and groins;
- sheet 3: Toe protection;
- sheet 4: Formulae of Van der Meer and Van Gent;
- sheet 5: Wave model refraction-shoaling;
- sheet 6: Wave model Battjes- Janssen.

Hereafter, an example of the input and output data of sheet 3 is shown.
(input data in **red**; output data in **blue** and **black**)

Armour size (concrete and rock units) of high-crested, low-crested and submerged breakwaters

General input data

Fluid density	1025	(kg/m3)
Density of rock armour units	2650	(kg/m3)
Density of concrete armour units	2300	(kg/m3)
Kinematic viscosity	0.000001	(m2/s)
Width of armour crest (Bc)	2	(m)
Total width of crest (Btotal)	10	(m)
Width of berm (Bberm=0, if no berm)	0	(m)
Seaward slope of armour layer Tan (alpha1)	0.67	(-)
Slope of rear armour layer Tan (alpha2)	0.5	(-)
Roughness factor armour (see table 2.6.3 on right)	0.45	(-)
Permeability factor of armour slope Van Gent	0.3	(-)
Permeability factor of armour slope Van der Meer	0.4	(-)
Design stability number CONCRETE UNITS (single layer) excl. safety factor (Table 3.3.3)	3	(-)
Safety factor runup	1.2	(-)
Safety factor wave overtopping rate	1.5	(-)
Safety factor wave transmission	1.2	(-)
Safety factor Stone sizes in double layer	1.1	(-)
Safety factor Stone sizes in single layer	1.5	(-)

Input data (10 columns): water depth, maximum water level, wave parameters, damage parameters
(each row is a new case)

Water Depth in front of toe to MSL ho (m) (MSL= mean sea level)	Crest height Rc to MSL (m)	Maximum water level above MSL (m)	Flow velocity in front of toe parallel to breakwater u-mean (m/s)	Effective Significant wave height at toe (m)	Wave period Tmean or Tpeak or Tm-1,o (s) (Single topped spectrum (Tmean = 0.9Tp) (Tm-1 = 0.9Tp) Used in method van der Meer)	Wave incidence angle at toe with normal to axis of structure Beta (degrees) (Beta=0 for normal waves) (Beta=90 for parallel waves)	Damage Sd rock units 0=start of damage 2=minor damage 4 severe 10=failure (Max value=5)	Damage Nod concrete units 0=start of damage 0.5=minor 2 severe 5=failure (Max value=1)	Number of waves Nw (-)
7	10	3	0.01	4	10	0	2	0.5	1800
7	10	4	0.01	6	14	0	2	0.5	1800
7	10	4	0.01	6	14	0	4	1	1800
8	0	-2	0.01	3	8	0	2	0.5	1800
8	0	2	0.01	4	10	0	2	0.5	1800
2.5	1	-1.5	0.01	1.5	5	0	2	0.5	2500
3.5	1	-0.5	0.01	2.1	6	0	2	0.5	2500
4.5	1	0.5	0.01	2.7	7	0	2	0.5	2500
5.5	1	1.5	0.01	3.3	8	0	2	0.5	2500
6.5	1	2.5	0.01	3.9	9	0	2	0.5	2500
7.5	1	3.5	0.01	4.5	10	0	2	0.5	2500
8.5	1	4.5	0.01	5.1	11	0	2	0.5	2500
6	6	0	0.01	3.6	10	0	2	0.5	2500
6	3	0	0.01	3.6	10	0	2	0.5	2500
6	2	0	0.01	3.6	10	0	2	0.5	2500
6	1	0	0.01	3.6	10	0	2	0.5	2500
6	0	0	0.01	3.6	10	0	2	0.5	2500
6	-1	0	0.01	3.6	10	0	2	0.5	2500
6	-2	0	0.01	3.6	10	0	2	0.5	2500
6	-3	0	0.01	3.6	10	0	2	0.5	2500
6	-5	0	0.01	3.6	10	0	2	0.5	2500
6	0	0	0.01	5.1	11	0	2	0.5	2500
6	0	0	0.01	5.1	11	0	2	0.5	2500
6	0	0	0.01	5.1	11	0	2	0.5	2500

Example of ouput data
(each row is a new case)

FRONT SLOPE AND CREST
ROCK/STONE SIZES Van Gent 2003
Randomly placed rocks in double layer

Wave angle factor (-)	Stability Number Ncr (-)	Correction factor at start	Correction factor after iteration (-)	Correction factor after iteration (-)	UNCorrected Size Dn,50 (if Rc/Dn>4) (m)	Rc/Dn50 (-)	Rc/Dn50 after iteration (-)	Corrected Size Dn,50 (m)	Corrected Size Dn,50 after iteration (m)	Corrected Size Dn,50 (if Rc/Dn<4) after last iteration (m)
1	1.58775125	1	1	1	1.589088296	4.40504157	4.40504157	1.5890883	1.5890883	1.589088296
1	1.58775125	1.00974784	1.00933399	1.00935133	2.383632445	2.54170353	2.54066181	2.3606215	2.3615894	2.361548816
1	1.82384724	1.00457527	1.00443466	1.00443894	2.075072567	2.90469437	2.9042878	2.0656218	2.06591096	2.065902159
1	1.58775125	1.03127921	1.02947381	1.02957622	1.191816222	1.73060106	1.7275714	1.15566784	1.15769456	1.157579398
1	1.58775125	1.2620154	1.30690623	1.31503904	1.589088296	-1.58835151	-1.64485036	1.25916712	1.21591608	1.208396291
1	1.53644038	1	1	1	0.615809022	4.05970019	4.05970019	0.61580902	0.61580902	0.615809022
1	1.53644038	1.02916179	1.02749012	1.02758431	0.862132631	1.79060928	1.78770078	0.83770369	0.83906659	0.838989681
1	1.53644038	1.09425815	1.09135015	1.09143903	1.108456239	0.49359555	0.49228382	1.01297509	1.01567424	1.015591536
1	1.53644038	1.161837	1.16765238	1.16786371	1.354779848	-0.42879181	-0.43093805	1.16606706	1.16025957	1.160049619
1	1.53644038	1.22235083	1.24757458	1.25054184	1.601103457	-1.14516413	-1.1687951	1.30985591	1.28337294	1.280327777
1	1.53644038	1.27445434	1.32674742	1.33735311	1.847427066	-1.72463418	-1.79539891	1.44958278	1.39244821	1.381405595
1	1.53644038	1.31894133	1.40286515	1.4269783	2.093750674	-2.20479674	-2.34508725	1.58744792	1.49248178	1.467261751
1	1.53644038	1	1	1	1.477941652	4.05970019	4.05970019	1.47794165	1.47794165	1.477941652
1	1.53644038	1.02040639	1.01930957	1.01936758	1.477941652	2.071272	2.06904563	1.44838534	1.44994386	1.449861352
1	1.53644038	1.04396719	1.04144346	1.04158585	1.477941652	1.41273127	1.40931607	1.41569741	1.41912807	1.418934075
1	1.53644038	1.07946523	1.07616572	1.07630101	1.477941652	0.7303842	0.72815169	1.36914243	1.37334021	1.373167585
1	1.53644038	1.12865422	1.12865422	1.12865422	1.477941652	0	0	1.3094725	1.3094725	1.309472495
1	1.53644038	1.1931482	1.20749717	1.20858848	1.477941652	-0.80730399	-0.81701274	1.23869076	1.22397112	1.222865914
1	1.53644038	1.27445434	1.32674742	1.33735311	1.477941652	-1.72463418	-1.79539891	1.15966622	1.11395857	1.105124476
1	1.53644038	1.3739943	1.50905365	1.56424068	1.477941652	-2.78900246	-3.06315269	1.07565341	0.97938311	0.944830086
1	1.53644038	1.49311915	1.79046942	2.01295563	1.477941652	-4.04107739	-4.84584602	0.98983504	0.82544926	0.734214721
1	1.53644038	1.63312035	2.22776409	3	1.477941652	-5.52498248	-7.5367119	0.90498025	0.66341928	0.492647217
1	1.53644038	1.12865422	1.12865422	1.12865422	2.093750674	0	0	1.85508604	1.85508604	1.855086035
1	1.53644038	1.12865422	1.12865422	1.12865422	2.093750674	0	0	1.85508604	1.85508604	1.855086035
1	1.53644038	1.12865422	1.12865422	1.12865422	2.093750674	0	0	1.85508604	1.85508604	1.855086035