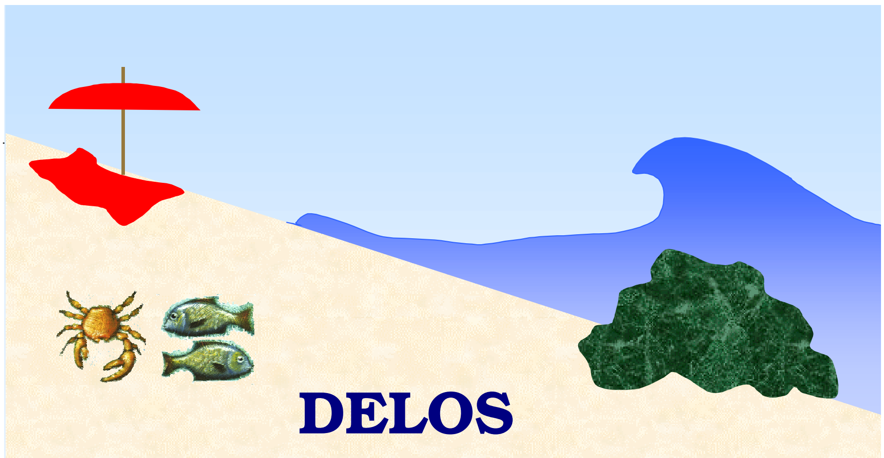
EU Fifth Framework Programme 1998-2002
Energy, Environment and Sustainable Development

Environmental Design of Low Crested Coastal Defence Structures



Deliverable 43

Structural Design Report for LCS
Final Report

Contract n°EVK3-CT-2000-00041

Structural design final report

DELOS Delivery no 43

The present report presents additional information and new formulae not included in DELOS D22 (Structural design preliminary report). Existing formulae for design of LCS's for coast protection purposes can be found in D22 and are therefore not repeated here.

Contents

1 Wave transformation by structures

Waves coming from deep water may reach a structure after refraction and breaking. As soon as waves reach a structure, such as an LCS, a lot of processes start. The waves may break on the structure, overtop it, generate waves behind the structure and reflect from the structure. Another effect may be wave penetration through openings between structures and diffraction around the head of structures. Both wave penetration and diffraction do not depend on the fact whether the structure is low-crested or not and, therefore, one is referred to handbooks for these items.

The main effect of an LCS is that energy can pass over the crest and generate waves behind the structure. The description of this wave transmission is the main objective in this chapter. As wave reflection decreases for lower structures, also this item will be treated.

The main parameters describing wave transmission have been given in Figure 1, here for a rubble mound structure. These are:

H_i = incident significant wave height, preferably H_{m0i}, at the toe of the structure
H_t = transmitted significant wave height, preferably H_{m0t}
T_p = peak period
s_{op} = wave steepness, $s_{op} = 2\pi H_i/(gT_p^2)$
R_c = crest freeboard
h_c = structure height
K_t = transmission coefficient H_t/H_i
ξ_{op} = breaker parameter $\xi_{op} = \tan\alpha/(s_{op})^{0.5}$

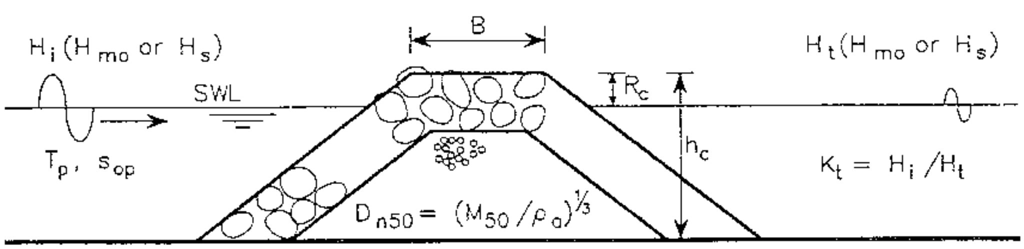


Figure 1. Governing parameters for wave transmission

1.1 Rubble mound low-crested structures

An extensive database on wave transmission was gathered in the DELOS project. This database was analysed to come up with the best formulae describing wave transmission. The full analysis is given in Briganti et al. (2003). The gathered database, made up of 2337 tests, include the data by Van der Meer and Daemen (1994) and by d'Angremond et al. (1996) on rock and tetrapod structures (old database); Calabrese et al. (2002) with large scale tests on shallow foreshores (GWK); Seabrook and Hall (1998) on submerged structures with very wide crests; Hirose et al. (2000) on Aquareef blocks with very wide crests; and Melito and Melby (2000) on structures with corelocs. Within the DELOS project tests were performed at the University of Cantabria (UCA) and the Polytechnic of Catalonia (UPC), both in Spain. Table 1 gives the datasets with the number of tests and ranges tested.

Table 1. Overall view of extensive database on wave transmission at rubble mound structures

Database	Armour type	Rc/Hi	B/Hi	B/Lop	ξop	Hi/Dn50	Hi/h	sop	Tests #
Old database	various	-8.7	0.37	0.009	0.7	0.3	0.03	2*10-4	398
		4.0	43.48	0.51	8.26	6.62	0.62	0.06	
UCA	rubble mound	-1.5	2.67	0.04	3.97	0.84	0.1	0.002	53
		1.53	30.66	0.4	12.98	2.42	0.37	0.02	
UPC	rubble mound	-0.37	2.66	0.07	2.69	2.65	0.17	0.02	24
		0.88	8.38	0.24	3.56	4.36	0.33	0.034	
GWK	rubble mound	-0.76	1.05	0.02	3	1.82	0.31	0.01	45
		0.66	8.13	0.21	5.21	3.84	0.61	0.03	
M & M	core locks	-8.2	1.02	0.02	2.87	0.68	0.05	0.01	122
		8.9	7.21	0.13	6.29	4.84	0.5	0.054	
Seabrook	rubble mound	-3.9	1.38	0.04	0.8	0.78	0.11	0.01	632
		0	74.47	1.66	8.32	3.2	0.58	0.06	
Aquareef	aquareef	-4.77	1.24	0.02	1.78	0.59	0.1	0.01	1063
		-0.09	102.12	2.1	5.8	4.09	0.87	0.08	

The main conclusion by Briganti et al. (2003) is that if submerged rubble mound structures with very wide crests are considered, two formulae should be considered, one for relatively narrow crested structures and one for very wide and submerged structures. The formulae are give by:

For $B/H_i < 10$:

$$K_t = -0.4\frac{R_c}{H_{si}} + 0.64\left(\frac{B}{H_{si}}\right)^{-0.31}\left(1 - e^{-0.5\xi}\right) \tag{1}$$

with minimum and maximum values of K_t = 0.075 and 0.80. Equation 1 is the original formula of d´Angremond et al. (1996), which proved to be applicable for the full dataset with the restriction given on crest width. For much wider crest widths a new formula was derived with similar structure.

For $B/H_i > 10$:

$$K_t = -0.35\frac{R_c}{H_{si}} + 0.51\left(\frac{B}{H_{si}}\right)^{-0.65}\left(1 - e^{-0.41\xi}\right) \tag{2}$$

Equation 2 has a minimum value of $K_t = 0.05$. The maximum value K_{tu} depends on the crest width of the structure. Very large crest widths give lower maximum values:

$$K_{tu} = -0.006 \frac{B}{H_i} + 0.93 \tag{3}$$

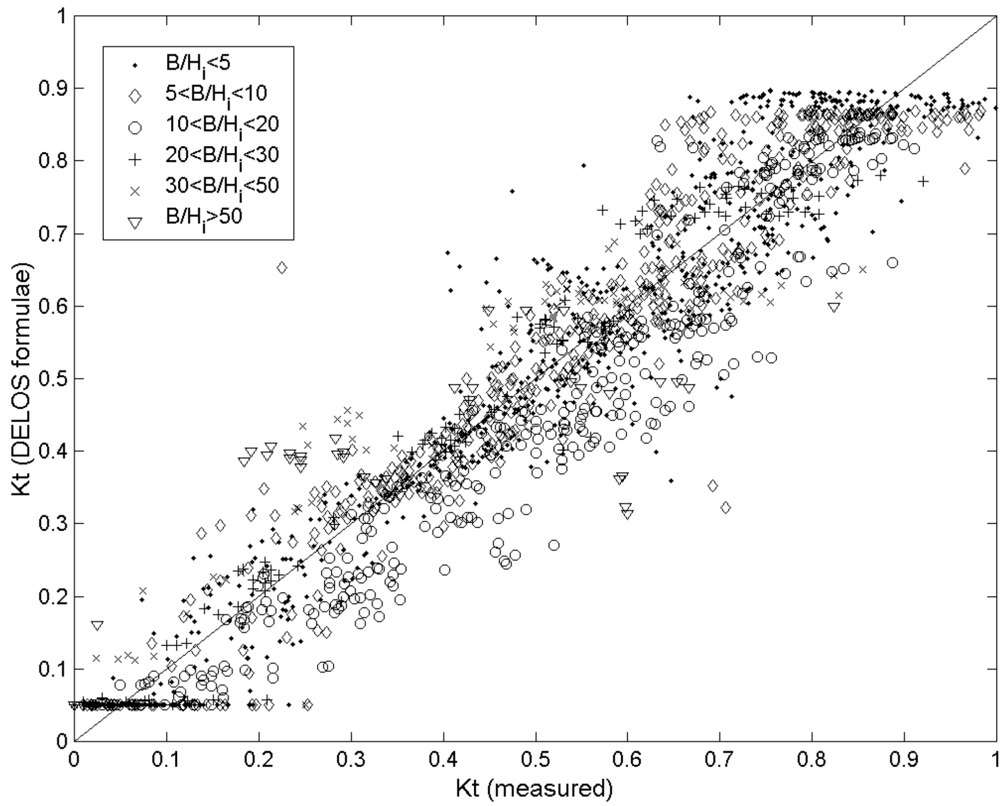


Figure 2. Calculated and measured transmission coefficients on rubble mound structures (Briganti et al. 2003)

A comparison of calculated and measured transmission coefficients is given in Figure 2. The results show still quite some scatter. The performance of formulae 1 and 2 + 3 may be evaluated in terms of round mean square error (RMSE) and R^2. They show an RMSE of 0.072 0.082 and R^2 equal to 0.91 and 0.90, respectively.

The DELOS project gave also results with regard to oblique wave attack and transmission, see Van der Meer et al. (2003). The main conclusion on the effect of angle of wave attack was that there was non to marginal influence on wave transmission up to a wave angle of 70° (0° is perpendicular wave attack). This conclusion means that equations 1 – 3, developed for perpendicular wave attack, can also be used for oblique wave attack, up to 70°.

Another question with regard to oblique wave attack is whether the transmitted wave angle is similar to the incident wave angle. The same research showed that this was not the case, the transmitted wave angle is consequently smaller than the incident one:

$$\beta_t = 0.80\ \beta_i \quad \text{for rubble mound structures} \tag{4}$$

where β_t = the angle of transmitted waves and β_I = the incident wave angle

1.2 Smooth low-crested structures

Not all low-crested structures are of the rubble mound type. Sometimes smooth and impermeable structures exist, for example low-crested structures covered with asphalt or armoured with a block revetment. Often the slope angles of the structure are gentler (1:3 or 1:4) than for rubble mound structures, mainly for construction reasons.
Wave transmission over smooth low-crested structures is completely different from rubble mound structures. First of all, the wave transmission is larger for the same crest height, simply because there is no energy dissipation by friction and porosity of the structure. Furthermore, the crest width has less or even no influence on transmission, as also on the crest there is no energy dissipation, which is completely different from rubble mound structures. Only for very wide (submerged) structures there could be some influence on the crest width, but this is not a case that will often be present in reality as asphalt and block revetments are mainly constructed in the dry and not under water. The presence of tide or storm surges make it possible to construct these kind of structures above water.

As smooth structures are different from rubble mound structures, also different formulae will be given for the transmission coefficient and the influence of oblique wave attack. The wave transmission can be calculated by, see Van der Meer et al. (2003):

$$K_t = [-0.3R_c/H_i + 0.75[1 - \exp(-0.5\xi_{op})]] \cos^{2/3}\beta \tag{5}$$

with as minimum $K_t = 0.075$ and maximum $K_t = 0.8$.

and limitations: $1 < \xi_{op} < 3 \quad 0° \leq \beta \leq 70° \quad 1 < B/H_i < 4$

Equation 5 already includes the effect of oblique wave transmission by the term $\cos^{2/3}\beta$. It was very clear from the experiments that wave transmission decreases with increasing obliquity. Figure 3 show this dependency, where on the vertical axis the measured transmission coefficient is given as a ratio to formula 5, without the cosine part.

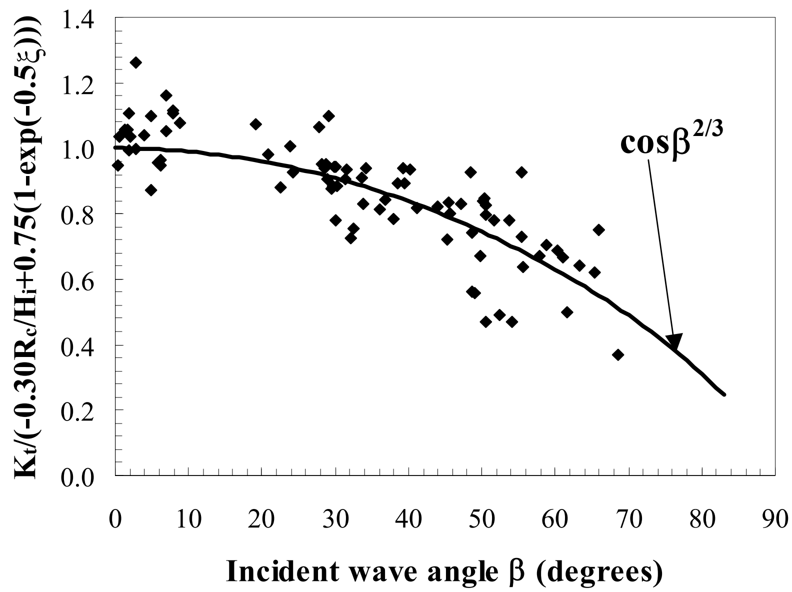


Figure 3. Influence of angle of wave attack on wave transmission for smooth structures

Oblique wave attack has also influence on the transmitted wave angle and in a different way than for rubble mound structures. Up to 45° the transmitted wave angle is similar to the incident one. Beyond 45° the waves jump along the structure and generate consequently a transmitted wave angle of 45°, regardless of the incident angle. Thus:

$$\beta_t = \beta_i \qquad \text{for } \beta_i \leq 45° \qquad\qquad (6)$$
$$\beta_t = 45° \qquad \text{for } \beta_i > 45° \qquad \text{for smooth structures}$$

1.3 Application of a neural network

It is clear in Figure 2 that still quite some scatter exists if formulae are based on various investigations and a large dataset. One of the main drawbacks of empirical formulae is that, in order to keep the application fairly simple, a reduced number of parameters is taken into account.

A neural network is a tool which has proven its usefulness if a process is difficult to describe and if a large dataset is available. In fact this is the case for wave transmission at rubble mound low-crested structures. In Panizzo et al. (2003) a neural network was made with the DELOS dataset as described in Table 1. Figure 4 gives the structure of the neural network and also the input parameters. The number of input parameters is larger than in formulae 1 – 3. The parameters in the formulae are R_c/H_i; B/H_i; and ξ_{op} (in Figure 4 given as Ir). For the neural network also H_i/D_{n50}; B/L_{op}, and H_i/h were added. This gives the added effect of the rock size, another effect of the wave length than only the breaker parameter, and the effect of wave height to water depth.

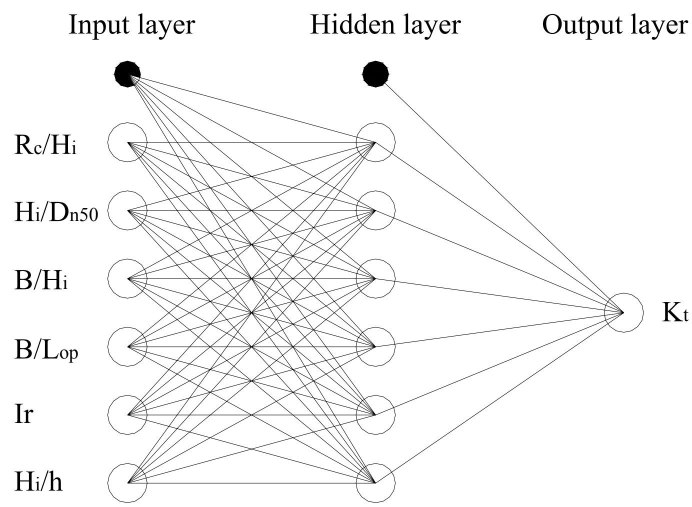


Figure 4. Structure of the neural network with the input parameters used

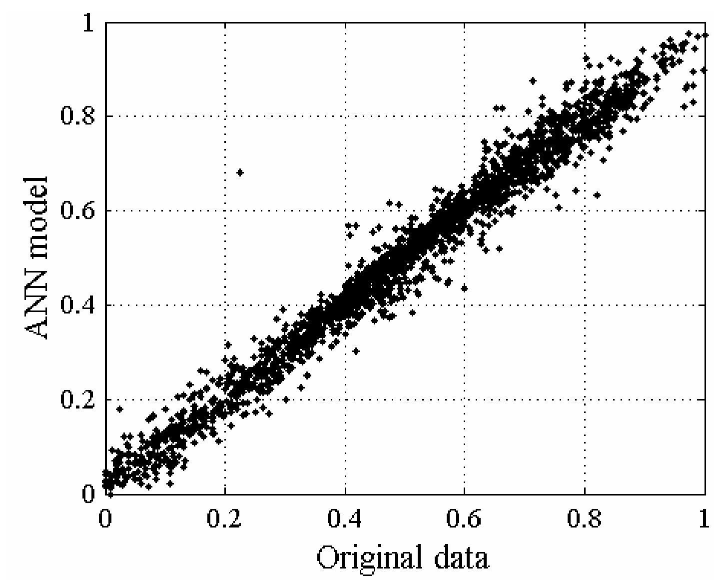


Figure 5. Comparison of wave transmission predicted by the neural network and measured

The results of the neural network are given in Figure 5 as predicted versus measured wave transmission coefficients. This graph should be compared with Figure 2 and it is clear that, due to the presence of an extensive dataset, the neural network performs much better than the empirical equations 1 – 3.

The drawback of a neural network, of course, is that an equation is directly applicable by the reader, a neural network is only applicable if the reader has access to this neural network.

1.4 Spectral change due to wave transmission.

Transmitted spectra are often different from incident spectra. Waves breaking over a low-crested structure may generate two or more transmitted waves on the lee side. The effect is that more energy is present at higher frequencies than for the incident spectrum. In general the peak period is quite close to the incident peak period, but the mean period may decrease considerably. A first analysis on this topic can be found in Van der Meer et al. (2000).

The wave transmission coefficient only contains information about the wave heights behind the structure. It is the spectrum which contains wave period information. Very often information is required on both wave heights and periods, for example for wave run-up or overtopping at structures behind a low-crested structure, or for calculation of morphological changes.

Figure 6 shows an example of a transmitted spectrum for a smooth structure and gives clearly the picture that energy is present more or less at a similar level up to high frequencies. Based on this, a simple and rude model was developed by Van der Meer et al. (2000), which is shown in Figure 7. In average 60% of the transmitted energy is present in the area of < $1.5f_p$ and the other 40% of the energy is evenly distributed between $1.5f_p$ and $3.5f_p$.

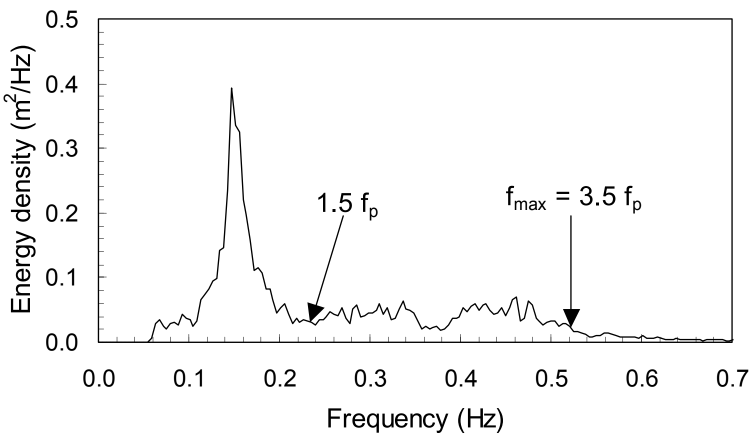


Figure 6. Example of transmitted spectrum with energy at high frequencies

It are these assumptions of division of energy in 60%/40% parts and the frequency of $f_{max} = 3.5f_p$, only based on a limited number of tests, which were more elaborated with new data of the DELOS project, see Briganti et al (2003) and Van der Meer et al. (2003).

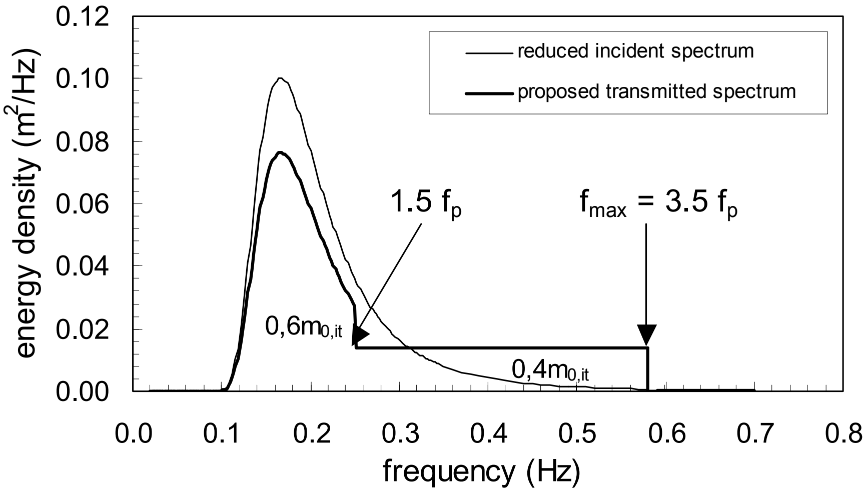


Figure 7. Proposed method by Van der Meer et al. (2000) for transmitted spectrum

The conclusion was that overall results are similar to the proposed method in Figure 7, although rubble mound structures give a little smaller values than smooth structures. Briganti et al. (2003) have taken this a little further and come to the conclusion that rubble mound and smooth structures do not give a similar behaviour. The method is also applicable for submerged rubble mound structures, *but not for emerged ones*. In the latter case much less energy goes to the higher frequencies and f_{max} may become close to 2.0. More research is needed to improve the method as described above.

1.5 Reflection from rubble mound low-crested structures

As far as wave transformation over low-crested structures is concerned, the DELOS project was focused on wave transmission only. Wave reflection was not considered to be an important aspect and was only treated at the end of the project. Preliminary results are given here.

Wave reflection at non-overtopped structures is described in the Rock Manual (CUR/CIRIA, 1991). For rock structures there are the data of Van der Meer (1988) and of Allsop and Channel (1989).The most simple prediction formula given in the Rock Manual is:

$$K_r = 0.14\ \xi_{op}^{0.73} \qquad \text{for } \xi_{op} < 10 \tag{7}$$

This formula, together with the original data, is shown in Figure 8. A more elaborated formula for rock slopes in the Rock Manual is:

$$K_r = 0.071\ P^{-0.82} \cot\alpha^{-0.62}\ s_{op}^{-0.46} \tag{8}$$

In this formula the slope angle has a little larger influence than the steepness, compared to the relationship in the breaker parameter ξ_{op}. Also the permeability (see Van der Meer

(1988) has a small influence. In the case of overtopped structures, the P-value will often be close to P = 0.4 - 0.6 and the influence of the slope angle will reduce if the structure becomes more submerged. Therefore the simple formula 7 was taken for comparison.

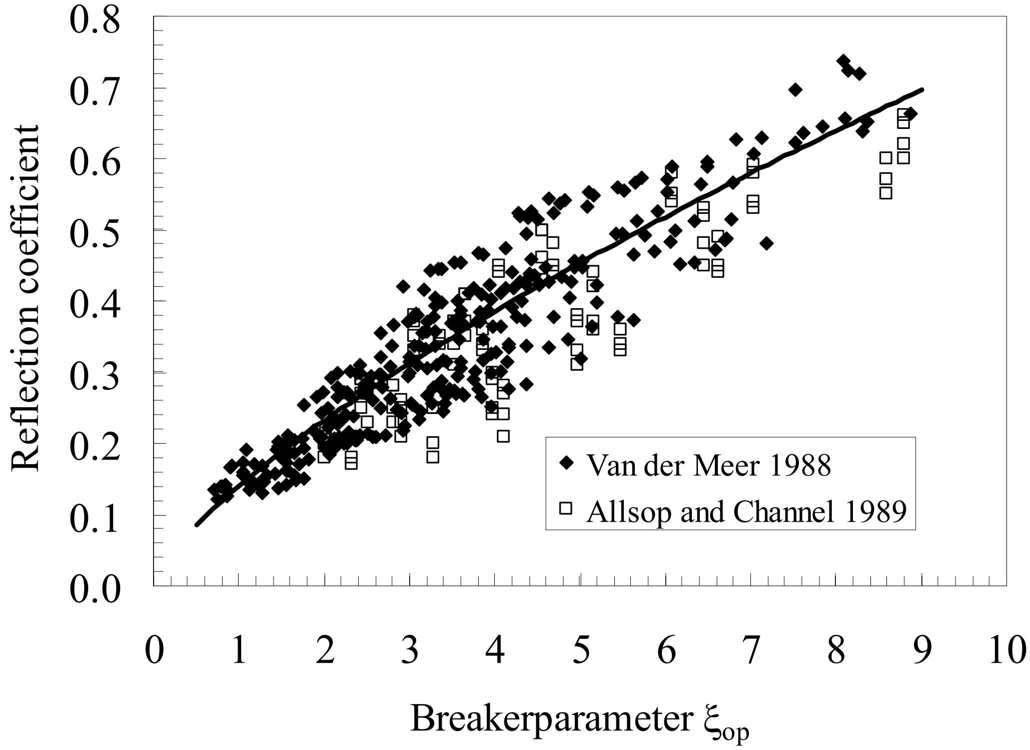


Figure 8. Reflection on non-overtopped rock slopes, CUR/CIRIA (1991)

It is expected that (very) submerged structures will have smaller reflection than non-overtopped, due to the fact that more energy will go over the structure. It is also expected that the relative crest height R_c/H_s has the main influence on a possible reduction of the reflection coefficient. The crest width will have no influence as waves reflect from the seaward side only.

Within the DELOS project there are 4 data sets with low-crested structures:

- UPC – large scale 2D tests; in total 63 tests
- UCA – small scale 2D tests; in total 53 tests
- UB – 3D tests in Aalborg by University of Bologna; random waves; lay-out 1; in total 28 tests
- INF – 3D tests in Aalborg by Infram; rubble mound structure; perpendicular attack; in total 19 tests.

Comparison of reflection coefficients with Figure 8 showed, for various reasons, quite some scatter. But is was clear that lower structures gave indeed lower reflection. In order to reduce the scatter and to come to a conclusion about the reduction in reflection by low-crested structures, the averages of groups of similar data points were taken. Furthermore, it was assumed that for the highest structures tested (R_c/H_i > 0.5), the influence on the reflection would be very small or not existing.

Based on these assumptions a reduction in average reflection coefficients was determined for data groups of the four mentioned projects. Figure 9 gives the final graph, which still must be considered as a preliminary result.

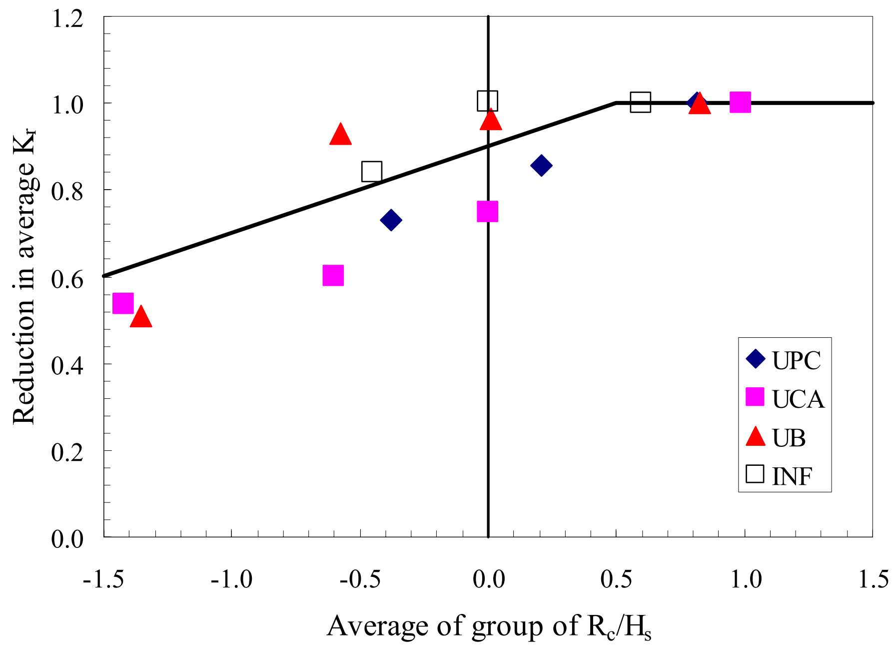


Figure 9. Reduction in reflection coefficient for low-crested rubble mound structures

The most simple relationship for low-crested structures becomes:

Reduction factor f_r on K_r: $f_r = 0.2\ R_c/H_s + 0.9$ for $R_c/H_s < 0.5$ (9)

$f_r = 1$ for $R_c/H_s \geq 0.5$

The reduction factor f_r in equation 9 can be applied to reflection coefficients determined by equations 1 or 2, or by other existing equations for wave reflection. Above results are valid for rubble mound structures. There is no method for smooth structures other than using also equation 9, but now applied to a prediction of reflection from smooth non-overtopped structures. Such prediction formulae can be found in the mentioned Rock Manual.

2 Formulae for structural stability

Filter layers, core materials and stone sizes for toe protection can be designed with existing tools for conventional structures, see e.g. the Coastal Engineering Manual.

2.1 Required stone sizes in armour layers

Stones used in the armour layer of a LCS must be sufficiently large to avoid undesirable damage. As LCS's are built in shallow water the highest waves will often be depth limited. The structures will typically be exposed to design waves numerous times during the lifetime. As damage is cumulative it is important to design such structures for a low damage criterion. Design recommendations are therefore here given for initiation of damage.

For conventional breakwaters only a small amount of energy is allowed to pass over or through the structure. Damage will therefore mainly happen to the front slope. For the Low-crested structure wave energy can pass over the structure making them more stable than the conventional type. Consequently smaller rubble stones can be used in the armour layer.

Numerical models are still too inaccurate to describe the stability phenomenon especially in case of 3D-waves. Therefore numerical models cannot be used in establishment of design formulae. To few existing laboratory tests were available to establish design formulae and therefore additional tests were performed at Aalborg University within the DELOS to establish the following design formulae. The formulae are described in Kramer and Burcharth (2003a) and a detailed report about the tests is available in the deliverables of DELOS, see Kramer and Burcharth (2003b). The new experiments and two existing datasets were included in establishing the design formulae. Structure geometries, wave basin/flume layouts, stone characteristics and types of waves generated were different in all three datasets, see Table 2.

Table 2. Model characteristics for NRC, Delft and AAU tests.

Parameter	**Test facility and year**		
	NRC 1992	**Delft 1995 (trunk)**	**AAU 2002**
Armour unit size D_{n50} [m]	0.025	0.035	0.033
Structure height H/D_{n50}	16.0	19.1	9.1
Crest width B/D_{n50}	6.0	Not known	3.0 and 7.6
Freeboard Rc/D_{n50}	-2.0 to 2.4	2.0	-3.0 to 1.5
Structure slope	1:1.5	1:2, leeward 1:1.5	1:2
Foreshore slope	Horizontal	Horizontal	1:20
Type of waves	2D irregular	2D irregular	3D irregular
Reference	Vidal *et al* 1992	Burger 1995	Kramer *et al* 2003

In the tests the trunk and the roundhead were divided in different sections and damage was measured within each section. Low narrow crested breakwaters built in shallow water are only a few stone-sizes high and wide. One stone removed from the edge of the crest will cause a large hole in the cross-section. When one section reached the initiation of damage stage it was therefore chosen to define the whole structure to be in this stage. In Figure 10 (left) a line representing the lower limit of the test results is given. This line represents the least stable part of the structure. The function for the line is given below by (10). If the highest waves are depth limited then the significant wave height can be replaced by the approximation Hs=0.6·h (h is water depth). By inserting in (10) ρ_r = 2.65t/m^3 corresponding to Δ=1.6, and Hs=0.6·h the curves in Figure 10 (right) are obtained. It is seen that the worst conditions are under slightly submerged conditions, i.e. Rc=-0.36·Hc, where Hc is the breakwater height. This relation is used in (10) to calculate the required D_{n50} and the following rule of thumb is found: D_{n50} = 0.29·Hc.

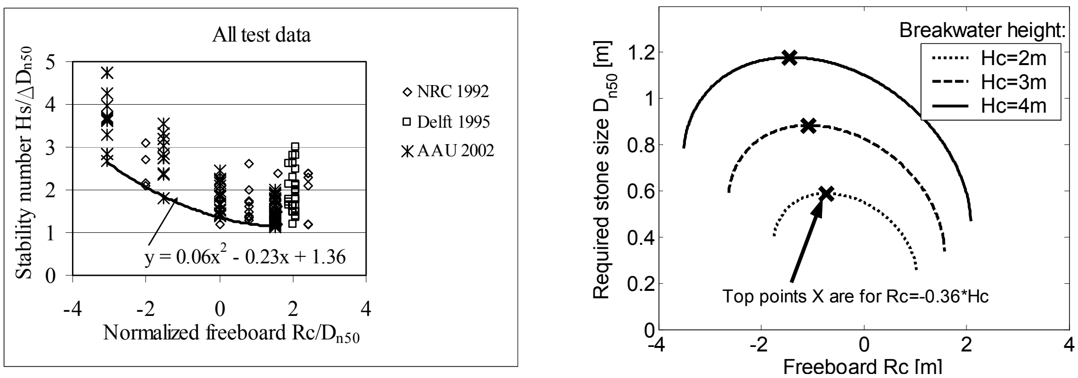


Figure 10. Design graphs for stability of low crested breakwaters corresponding to initiation of damage. Test results (left) and formula in case of depth limited waves (right).

Recommendations

It is recommended to choose a crest width at least equal to the largest significant wave height. The crest width should correspond to at least three stones. The stones in the trunk and the roundhead should be of the same size. If it is chosen to use only one stone size (no core, i.e. homogeneous cross-section) design by (10) and (11) given below will be conservative.

Required stone size in shallow water waves. When designing a low crested breakwater the highest significant wave heights must be calculated for different water depths caused by tide and storm surge. The corresponding necessary stone sizes for each of these water depths can then be found from Figure 10. In this way the "worst condition" will be the water depth giving the smallest stone size. It is recommended to choose the stone size according to the lower line shown in Figure 10 (left) given by (10). The formula is only valid for relatively low freeboards given by the ranges in (10).

$$\frac{Hs}{\Delta D_{n50}} = 0.06\left(\frac{Rc}{D_{n50}}\right)^2 - 0.23\frac{Rc}{D_{n50}} + 1.36 \qquad \text{, for } -3 \le Rc/D_{n50} < 2 \qquad (10)$$

In (10) Hs is the significant wave height, Rc is the freeboard (negative if submerged), D_{n50} is the mean nominal diameter of the armour, and $\Delta = (\rho_r - \rho_w)/\rho_w$, where ρ_r and ρ_w are the densities of rock and water, respectively.

Required stone size in depth limited waves. If the highest waves are depth limited and regular rock are used then slightly submerged conditions are the most critical. The required D_{n50} can be estimated by the following rule of thumb:

$$D_{n50} = 0.29 \cdot Hc \qquad \text{, Hc is the structure height} \tag{11}$$

According to (11) the structure height will be no more than 3 to 4 D_{n50}.

2.2 Scour protection

It is imperative to construct a protection layer for toe protection. This protection layer may be constructed in the form of a protection apron. The apron must be designed so that it will remain intact under wave and current forces, and it should be "flexible" enough to conform to an initially uneven seabed. With this countermeasure, scour can be minimized, but not entirely avoided. Some scour will occur at the edge of the protection layer, and consequently, armour stones will slump down into the scour hole. This latter process will, however, lead to the formation of a protective slope, a desirable effect for "fixing" the scour. The determination of the width of the protection layer is an important design concern. The width should be sufficiently large to ensure that some portion of the protection apron remain intact, providing adequate protection for the stability of the breakwater.

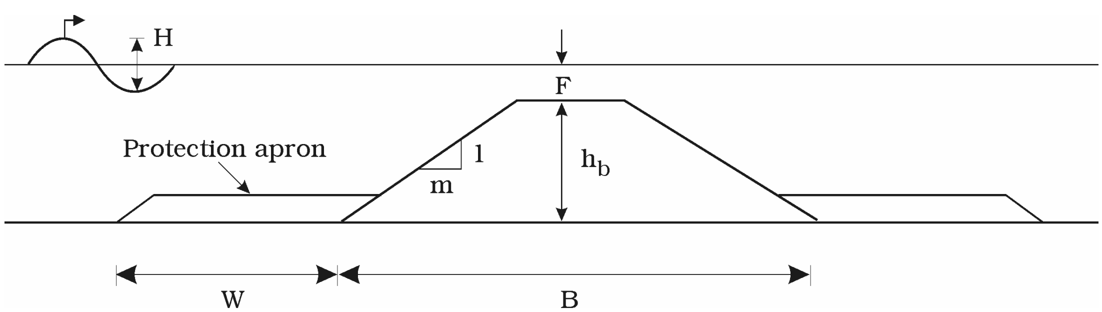


Figure 11 Definition sketch.

Formulae for the width of the toe protection at the trunk section

On the basis of the experiments on scour at LCS undertaken in DELOS, it is recommended that the width of the protection apron (Figure 11) be calculated by the following empirical equation

$$W = \alpha \frac{L}{4}$$

in which

$$\alpha = 1 - \frac{mh_b}{L/4} \quad ,$$

m is the slope of the surface of the breakwater (Figure 11),
h_b is the height of the breakwater (Figure 11), and
L is the wave length of the incident wave.

This is for the scour protection at the offshore side of the breakwater. The scour experiments undertaken in DELOS suggest that the same width may be selected for the toe protection apron at the onshore side. Extra precautions must be exercised towards reinforcing the protection layer on this side to protect the protection material against damage caused by wave overtopping.

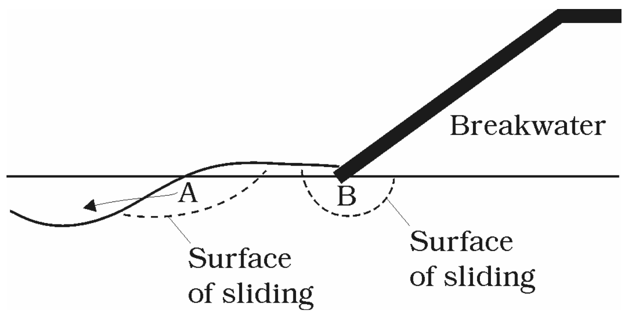


Figure 12 Possibility of sand slide in front of breakwater.

The above equation is based on the scour experiments where the mode of sediment transport was in the no-suspension regime. In the case of the suspension-regime sediment transport, from the knowledge of scour at emerged breakwaters, no scour is expected at the toe (at the offshore side of the breakwater), and therefore scour is not an immediate threat to the breakwater. However, soil failure illustrated in Figure 12 may be a risk for stability, and hence may need to be considered (Sumer and Fredsøe, 2002).

Furthermore, the preceding equation is for scour protection against the *local* scour caused by the combined effect of steady streaming and phase-resolved stirring of sediment by waves (Sumer and Fredsøe, 2002). Due considerations must be given to *global* scour caused by the far-field flow circulations around the breakwater.

Formulae for the width of the toe protection at the head section

It is recommended that the width of the protection apron be calculated by the following empirical equation

$$W = W_e \quad if \quad \frac{F}{H} < -0.9$$

$$W = (-0.29\frac{F}{H} + 0.74)W_e \quad if \quad \frac{F}{H} > -0.9$$

in which

F is the free board (Figure 11; negative values correspond to slightly or fully emerged breakwaters),
H is the wave height, and
W_e is the width recommended for “fully” emerged breakwaters, given by
$W_e / B = A\ KC$
B is the diameter of the round head at the bed,
A is 1.5 for complete scour protection and 1.1 for a scour protection which allows a scour depth of 1% of B, and
KC is the Keulegan-Carpenter number, $KC = (2\pi a)/B$ in which a is the amplitude of the orbital motion of water particles at the bed, and may be calculated using the small-amplitude, linear wave theory.

The above equation is based on the experiments where the breakwater slope was 1:1.5 (i.e., m = 1.5, Figure 11). Therefore, for slopes steeper than 1:1.5, the width necessary for protection may be increased, and for slopes milder than 1:1.5, it may be reduced.

Furthermore, the above equation is for scour protection against the *local* scour caused by the combined effect of steady streaming and phase-resolved stirring of sediment by waves (Sumer and Fredsøe, 2002). Due considerations must be given to *global* scour caused by the far-field flow circulations around the breakwater.

Finally, the recommended width is for protection at the offshore side of the head. Experiments show that the implemented widths of the protection layer are able to protect the sand bed against the breaker-induced scour at the onshore side of the head. However, scour (damage) may occur in the protection layer itself due to wave breaking and wave overtopping. Therefore, additional reinforcement is recommended at the onshore side regarding the protection material.

3 References

Allsop, N.W.H. and A.R. Channel, 1989. “Wave reflections in harbours: reflection performance of rock armoured slopes in random waves. Hydraulic Research, Wallingford, Report OD 102

Briganti, R., J.W. van der Meer, M. Buccino and M. Calabrese, 2003. “Wave transmission behind low crested structures”. *ASCE, Proc. Coastal Structures, Portland, Oregon*.

Burger, G. (1995) "Stability of low-crested breakwaters", MSc. Thesis Delft University, report H1878/H2415 and Final Proceedings, EU research project Rubble mound breakwater failure modes, MAST 2 contract MAS2-CT92-0042.

d'Angremond, K., J.W. van der Meer and R.J. de Jong, 1996. "Wave transmission at low-crested structures". *ASCE, Proc. ICCE, Orlando, Florida,* 3305-3318.

Calabrese M., V. Vicinanza, M. Buccino, 2002, "Large scale experiments on the behaviour of low crested and submerged breakwaters in presence of broken waves", *Proc. 28th Int. Conf. on Coastal Engineering, ASCE, 1900-1912*.

CUR/CIRIA, 1991. Rock Manual. "Manual on the use of rock in coastal and shoreline engineering". CUR Report 154, the Netherlands; CIRIA Special Publication 83, UK.

Hirose, N., A. Watanuki and M. Saito, 2002. "New Type Units for Artificial Reef Development of Ecofriendly Artificial Reefs and the Effectiveness Thereof", *Proc. 30th International navigation congress, PIANC, 2002.*

Kramer, M. and Burcharth, H.F. (2003a), Stability of Low-Crested Breakwaters in Shallow Water Short Crested Waves, Coastal Structures 2003 Conference, Portland, USA.

Kramer, M. and Burcharth, H.F. (2003b) "3D Stability Tests at AAU", part of internal report for DELOS deliverable D31, available from the Internet www.delos.unibo.it.

Melito, I and J.A. Melby, 2002 "Wave runup, trasmission, and reflection for structures armoured with CORE-LOC*". J. of Coastal Engineering 45, 33 – 52. Elsevier*.

Panizzo, A., R. Briganti, J.W. van der Meer and L. Franco, 2003. "Analysis of wave transmission behind low-crested structures using neural networks". *ASCE, Proc. Coastal Structures, Portland, Oregon*.

Seabrook S.R. and K.R. Hall, 1998. "Wave transmission at submerged rubble mound breakwaters", *Proc. 26th Int. Conf. on Coastal Engineering, ASCE ,2000-2013.*

Sumer, B.M. and Fredsøe, J (2002): The Mechanics of Scour in the Marine Environment. World Scientific, 552 p.

Van der Meer, J.W., 1988. "Rock slopes and gravel beaches under wave attack". PhD-thesis Delft University of Technology, the Netherlands

Van der Meer J.W. and I.F.R. Daemen, 1994. "Stability and wave transmission at low crested rubble-mound structures", *J. of Waterway, Port Coastal and Ocean Engineering, 1, 1-19.*

Van der Meer, J.W., H.J. Regeling and J.P. de Waal, 2000. "Wave transmission: spectral changes and its effect on run-up and overtopping". *ASCE, Proc. ICCE, Sydney, Australia.* 2156-2168.

Van der Meer, J.W., B. Wang, A. Wolters, B. Zanuttigh and M. Kramer, 2003. "Oblique wave transmission over low-crested structures". *ASCE, Proc. Coastal Structures, Portland, Oregon*

Vidal, C., Losada, M.A., Medina, R., Mansard, E.P.D. and Gomez-Pina, G. (1992) "A Universal Analysis for the Stability of both Low-Crested and Submerged Breakwaters", Proc. 23rd International Conference on Coastal Engineering, pp. 1679-1692, Italy.