Stability of Low-Crested Breakwaters in Shallow Water Short Crested Waves

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Abstract

The paper presents results of 3D laboratory experiments on low-crested breakwaters. Two typical structural layouts were tested at model scale in a wave basin at Aalborg University, Denmark, to identify and quantify the influence of various hydrodynamic conditions (obliquity of short crested waves, wave height and wave steepness) and structural geometries (crest width and freeboard) on the stability of low-crested breakwaters. Results are given in terms of recommendations for design guidelines for structure stability. Damage parameters for the trunk and the roundhead are proposed based on analysis of observed damage. Results for initiation of damage are compared to existing data and a good agreement is found.

Introduction

Low-Crested structures (LCS's) are typically built in shallow water as detached breakwaters for coastal protection purposes. The structures are usually parallel to the shore line with wave attack almost perpendicular to the structure. However under special environmental conditions more oblique waves can occur. Groin systems or breakwaters for harbours where structures are not parallel to shore line are other examples in which oblique wave attack occur. As the structures 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. Several 2D laboratory experiments on trunk armour layer stability of LCS's have been performed in wave channels; see e.g. Ahrens 1987, Van der Meer *et al* 1990/1996, Loveless and Debski 1997. To our knowledge only one 3D test series with long crested waves has been carried out on complete LCS's, see Vidal *et al* 1992. These tests were carried out in the wave basin at NRC, Canada, 1991-1992 on a 4.7m long structure and was tested in irregular head-on waves. Only one structure geometry was tested with cross-section slopes 1V:1.5H. The results could therefore only quantify the influence of freeboard on the stability for that specific geometry.
The task for the new stability tests on LCS's (mainly roundhead but also trunk) was to supplement existing tests in order to identify the influence on rubble stone stability of:

1) Obliquity of short crested waves
2) Wave height and steepness
3) Crest width
4) Freeboard

The tests were performed at Aalborg University (AAU) within the EU-project DELOS (see acknowledgements for further information) and a detailed report about the tests is available in the deliverables for the project, see Kramer and Burcharth 2003. 69 tests were performed with irregular 3D waves generated using the cosine power spreading function with spreading parameter S=50, see Mitsuyasu *et al* (1975). The wave height was increased in steps until severe damage occurred. Two wave steepnesses of 0.02 and 0.04 and angle of incidences in the range -30° to +20° were generated (0°=normal incidence), see Figure 1. A five wave gauge array was used to measure the directional wave spectra. Digital video and digital photos were taken to visualize and quantify the damage progression.

Experimental set-up

Wave basin layout. The wave basin used in the tests has dimensions as shown in Figure 1. The absorbing sidewalls were made of crates (121x121cm, 70cm deep) filled with sea stones with nominal diameter D_{n50} of approximately 5cm. The absorbing beach was made of quarry rock with D_{n50}=1.5cm.

Structural layout and cross-sections. The trunk and the roundhead were constructed by carefully selected quarry stones with density 2.65t/m^3. The stones were painted in different colours to identify and quantify damage using digital photos. Two different cross sections were tested at different water levels; see Figure 1 and Figure 2. The length of the structure was 5m. Negative freeboards represent submerged structures. A circular roundhead with crest radius equal to half the trunk crest width was chosen. The water depth in front of the wave paddle was varied from 33cm to 48cm, which gives water depths at the structure between 0.25m and 0.40m.

Materials. Three types of armour stones were used. Carefully selected stones (Type A) were used in the test sections where damage was measured, see Figure 3. For the dummy section between the trunk and roundhead test section a net with large masks (2x2cm) was covering the surface to avoid damage in that area. This made rebuilding easier and gave less strict specifications for the armour material (Type B). For the dummy section between the side-wall (to the right on Figure 3) and the trunk test section, larger stones (Type C) were used to avoid damage. Type A stones were used in 15cm ($5 \cdot D_{n50}$) strips on each side of the test sections to ensure correct boundary conditions. For the core was used more wide graded stones (Type D), cf. Table 1. The porosity (n) for armour Type A and core Type D was $n_{(Type\ A)} = 0.44$ and $n_{(Type\ D)} = 0.43$.

Wave conditions. In all tests a Jonswap spectra with peak enhancement factor 3.3 and a spreading parameter S = 50 was used. The target length of each series was 1000 waves. A test block was defined by fixed water level, wave direction, wave steepness, and spreading, see Table 2. In each test block the significant wave height Hs was increased in steps until severe damage was observed. It was attempted to get four tests in each block. However, this was not possible in all blocks due to the progress of the damage. Target conditions were therefore continuously adjusted according to target damage during a tests block. After each block the breakwater was rebuilt.

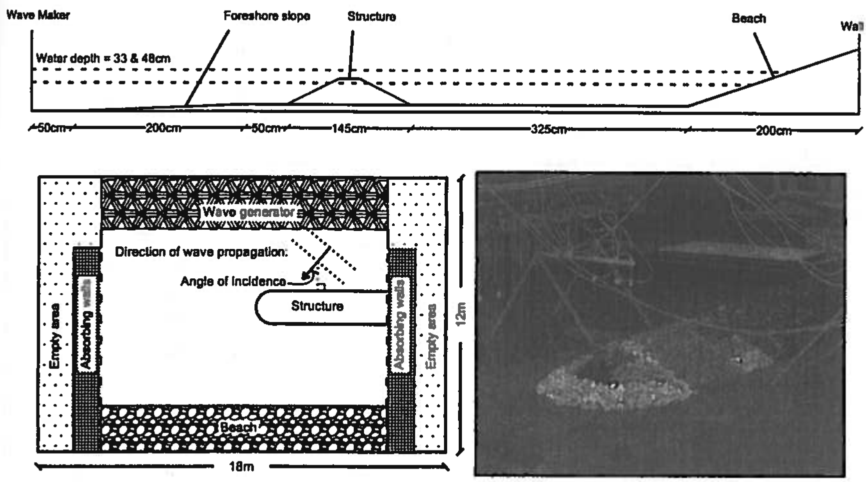
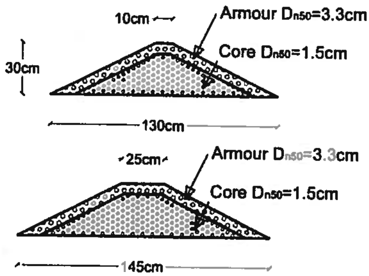


Figure 1. Wave basin layout and bottom topography.



Parameter	Value
Crest width	$3D_{n50}$ and $8D_{n50}$
Crest height Hc	0.30m
Structure slope	1V : 2H
Freeboards Rc	-0.10, -0.05, 0.0 and 0.05m
Armour	D_{n50}=0.033m
Core	D_{n50}=0.015m
Armour layer thickness: 0.66m ($2D_{n50}$)	

Figure 2. Cross-section geometry.

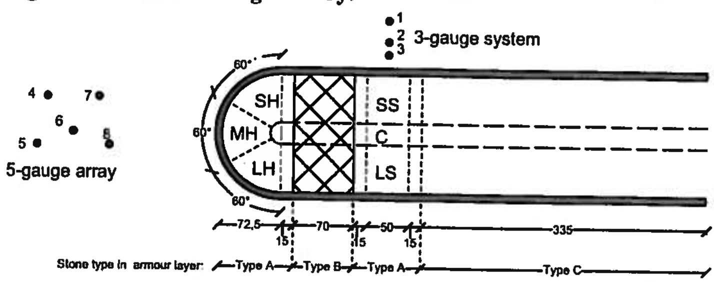


Figure 3. Stone types in structural sections. Measures in cm.

Table 1. Grading of materials.

Stone type	Size			
	D_{n50} [cm]	D_{n85} [cm]	D_{n15} [cm]	D_{n85}/D_{n15}
Type A (armour, test section)	3.25	3.60	3.01	1.20
Type B	3.07	3.43	2.68	1.28
Type C	4.74	5.24	4.32	1.21
Type D (core)	1.44	1.83	1.11	1.64

Table 2. Test conditions.

Narrow crest (width = 0.1m)			
Test block	Dir. [°]	Free-board [m]	Wave steepness
1	0	0.05	0.02
2	0	0.05	0.04
3	0	0.00	0.02
4	0	0.00	0.04
5	0	-0.05	0.02
6	0	-0.05	0.04
7	0	-0.10	0.02
8	0	-0.10	0.04

Wide crest (width = 0.25m)			
Test block	Dir. [°]	Free-board [m]	Wave steepness
9	0	0.05	0.02
10	-20	0.05	0.02
11	-10	0.05	0.02
12	10	0.05	0.02
13	20	0.05	0.02
14	-30	0.05	0.02
15	0	0.00	0.02
16	0	-0.05	0.02
17	0	-0.10	0.02

Measurements. Three kinds of measurements/observations were performed:

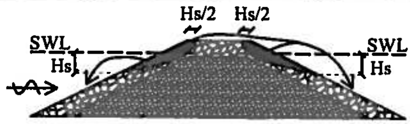
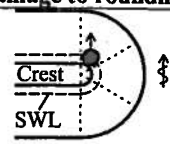
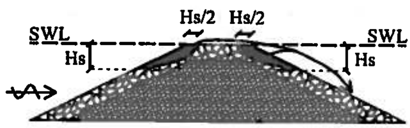
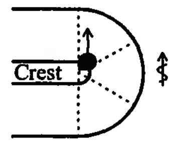
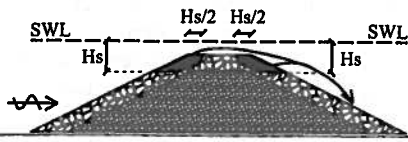
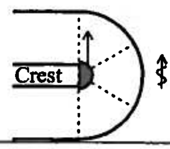
- Continuous wave recordings during the tests.
- Visual observations of wave breaking.
- Measurements of damage in terms of displacement of stones after each test by use of digital photos.

The roundhead was split in three sections of 60° each, see Figure 3. The three sections were called: Seaward Head (SH), Middle Head (MH) and Leeward Head (LH). The trunk was split in three parts called: Seaward Slope (SS), Crest (C), and Leeward Slope (LS). The damage was measured within each section.
Waves were recorded by an array of five wave gauges to be used in estimating incoming and reflected wave spectra, see Figure 3. At the position of the array almost 1.5 metres from the roundhead the influence of the roundhead (reflection and diffraction) on the incoming waves is believed to be negligible. However, the trunk reflects some wave energy which is re-reflected by the paddles. Therefore the waves in front of the trunk might in reality be slightly higher with more wave breaking than at the array. Measurements from the 3-gauge system and visual observations were performed to quantify this effect.

Description of damage
It is important to choose a damage parameter which effectively describes a certain level of acceptable damage for the structure. Often the structures are designed for a state called "initiation of damage", referred to as the state where a few stones starts to move. Mathematically this level is usually either expressed as a certain percentage of displaced stones or as a relative eroded area.
Depending on the freeboard three different damage behaviours of the breakwaters were identified, see Table 3. For the tested range the exposed areas of the trunk was found not to be depending on the crest width. Damage only takes place at the edges of the trunk crest. In other words a few displaced stones cause the same damage to the trunk crest for the wide structure as to the more narrow structure. The exposed crest section covered an area from the edge of the crest and Hs/2 towards the centre. Very narrow crest structures will be vulnerable to damage as the exposed area will cover the whole crest width. It is therefore recommended to choose a crest width at least equal to Hs when designing a low crested breakwater. No influence of wave steepness was found. In case of oblique waves the area of damage for the trunk remained the same as for normal incidence waves, but the stones were displaced in the direction of wave propagation.
For the roundhead the same effects were identified. However, due to the distribution of the damage the crest width becomes important as a few stones displaced from a wide roundhead are less damaging as for a narrow roundhead. Under slightly emergent conditions the damage was found to be the same as for conventional non-overtopped breakwaters, i.e. the most exposed area was the part around SWL (Still Water Level) covering 60° of the leeward head. For zero freeboard or submerged conditions the

exposed area shifted towards the crest. In this way the whole top part of the roundhead crest was vulnerable under submerged conditions. For conventional non overtopped breakwaters larger armour units and/or higher density units are often used in the most exposed part to ensure a stable roundhead. If it is chosen to reinforce the roundhead of a low crested structure it is important that the whole top part of the roundhead is reinforced. As for the trunk the exposed section covered an area vertically down to approximately SWL-Hs.

Table 3. Sections prone to damage. Filled black areas indicate exposed stones.

Freeboard	Damage to trunk	Damage to roundhead
Rc > 0 Slightly emergent crest		
Rc = 0		
Rc < 0 submerged crest		

For the roundhead the wave direction had an influence on the location of the exposed area. In Figure 4 it is seen, that the damage is shifting towards the middle part of the head when a smaller part of the head is exposed to direct wave attack (going right in Figure 4, wave directions >0°). It is also seen, that the exposed area is getting smaller and that it moves towards the toe. During the experiments it was experienced that wave breaking tend to focus at the roundhead forming a jet of water and air slamming down on the top part of leeward head. This effect shifted towards the middle head in case of oblique waves causing the middle head more prone to damage. The effects experienced in oblique waves again recommend, that if reinforcement of the roundhead is needed, then the whole top part of the roundhead should be reinforced.

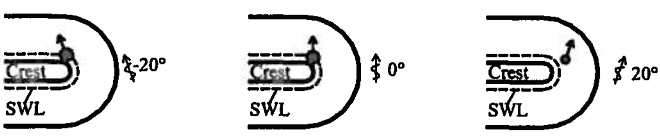


Figure 4. Influence of wave direction on exposed roundhead areas.

Based on the observed damage the following damage parameters seems appropriate in describing the damage of low crested breakwaters with small or moderate crest widths.

Damage parameter for trunk. Broderick (1983) defined the damage parameter $S = A_e/D^2_{n50}$. A_e is the average eroded area in a section of width X. S is often used to quantify damage of rubble structures. Broderick's equation can be written in terms of number of displaced units (N). The number of displaced stones N in the test section is assumed to equal the eroded volume $V_e = N \cdot D^3_{n50}/(1-n)$. The average eroded area in the test section can then be calculated as $A_e = V_e/X$, and Broderick's equation becomes:

$$S = \frac{N \cdot D_{n50}}{(1-n) \cdot X} \tag{1}$$

In (1) n is porosity of armour, X is width of test section and D_{n50} is the nominal diameter of armour units. In the AAU experiments the numbers of displaced units N were measured. In the AAU experiments for the trunk (1) becomes $S = 0.11 \cdot N$.

Damage parameter for roundhead. The methodology proposed by Vidal *et al* (1995) seems appropriate. Vidal observed that the region most prone to damage was between Hs/2+SWL and SWL-Hs. This is the same as observed in the AAU experiments. Vidal suggested using the mean head radius R calculated from (2). In this way the vertical extend and the horizontal extend (along the crest width) of the damage is taken care of in a proper way.

$$\begin{aligned} R &= \frac{B}{2} + \frac{\cot\alpha\,(Hs + Rc)}{2}, \text{ for } Rc \le \frac{Hs}{2} \\ R &= \frac{B}{2} + \cot\alpha\left(\frac{Hs}{4} + Rc\right), \text{ for } Rc > \frac{Hs}{2} \end{aligned} \tag{2}$$

In (2), B is the crest width, α is the structure slope, and Rc is the freeboard. Further the arc length $A_{l\theta}$ is calculated as $A_{l\theta} = R\theta$, where θ is the angle covered by the actual section of the roundhead, e.g. $\theta = \pi/3$ equal to 60° for the leeward head. It is then possible to calculate a damage parameter according to (3).

$$S_{head} = \frac{N \cdot D_{n50}}{(1-n) \cdot A_{l\theta}} \tag{3}$$

Definition of initiation of damage. For the trunk a uniform distribution of S was chosen corresponding to initiation of damage such that S=0.5 for the seaward slope, S=0.5 for the crest, and S=0.5 for the leeward slope. S=0.5 corresponds to approximately 4 displaced stones along the 50cm wide test section in the experiments. From the experiences during the experiments and the exposed areas shown in Table 3 this level seems reasonable as "initiation of damage". Some authors (see e.g. Van der Meer *et al* 1996, Vidal *et al* 1995) have performed similar experiments and suggest a higher S-value for the seaward slope than for the remaining parts of the trunk. The AAU experiments did not show any behaviour of the eroded area to support a larger S-value for the front slope.

For the roundhead a uniform distribution was chosen, such that S=1 for the seaward head, S=1 for the middle head, and S=1 for the leeward head. Again there were no reasons to allow larger S-values in some regions.

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.

Please note that in the present experiments S is calculated by counting displaced stones. Calculations therefore disregard settlements. This implies that S-values obtained by profilers generally are larger if settlements occur.

Comparison of results to existing data

The freeboard and the wave height are the most important parameters in describing the stability of the structural sections. The normalized freeboard Rc/D_{n50} has therefore been used as one primary parameter, and the wave height ratio or stability number Ns = $Hs/\Delta D_{n50}$ as another primary parameter. $\Delta = (\rho_r - \rho_w)/\rho_w$, where ρ_r and ρ_w is the density of rock and water respectively. In Kramer and Burcharth (2003) it was concluded that measurements from wave gauges 1-3 (shown in Figure 3) should be used in describing Hs for the stability number. Reflection coefficients for the structure leading to initiation of damage were in all tests less than 12%. Reflections were in general small giving approximately the same incident wave height as total wave height. Visual observations of wave breaking did not show any increase in wave breaking in front of the structure. The following results are therefore based on total wave heights measured from gauge 3.

Table 4. 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 experiments described in Table 4 the damage of the trunk was investigated for the seaward slope, the crest and the leeward slope. In the NRC tests the roundhead was only divided in two sections. The comparison has therefore been performed for the same two sections. All the test data were re-analysed to ensure consistency according to the chosen damage levels. Structure geometries, wave basin/flume layouts, stone characteristics and types of waves generated were different in all three datasets. Because of this some deviations between the results is expected and also observed, see Figure 5. However, when the differences are kept in mind all datasets are considered to be in reasonable agreement. The seaward head and trunk seaward slope is slightly more stable in the AAU experiments especially under submerged conditions. The structure slope was gentler in the AAU experiments and a larger stability is therefore expected for the AAU structure. There were other differences in the two test setups, but it seems reasonable to relate the differences in stability to the structure slopes.

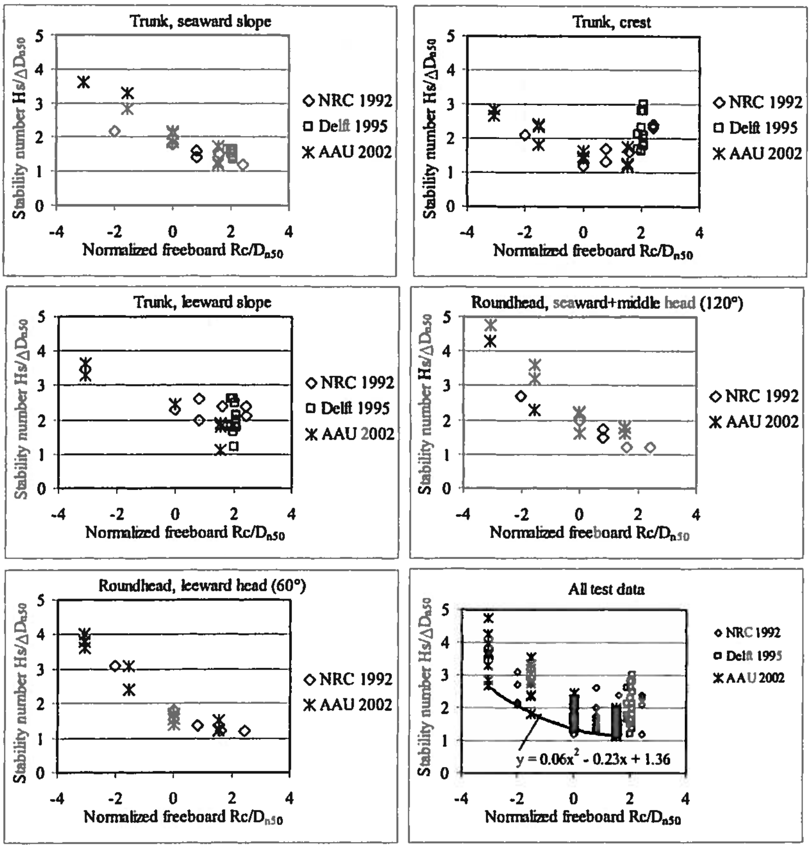


Figure 5. Comparison with existing stability results for initiation of damage.

Results of new experiments
For the trunk the crest is the least stable section and the leeward slope the most stable part, see Figure 6. For the roundhead the leeward head is the least stable part, and the stability of the middle head and seaward head is approximately the same. The trunk crest is the least stable part under submerged conditions, and for emergent conditions the leeward head is the least stable.

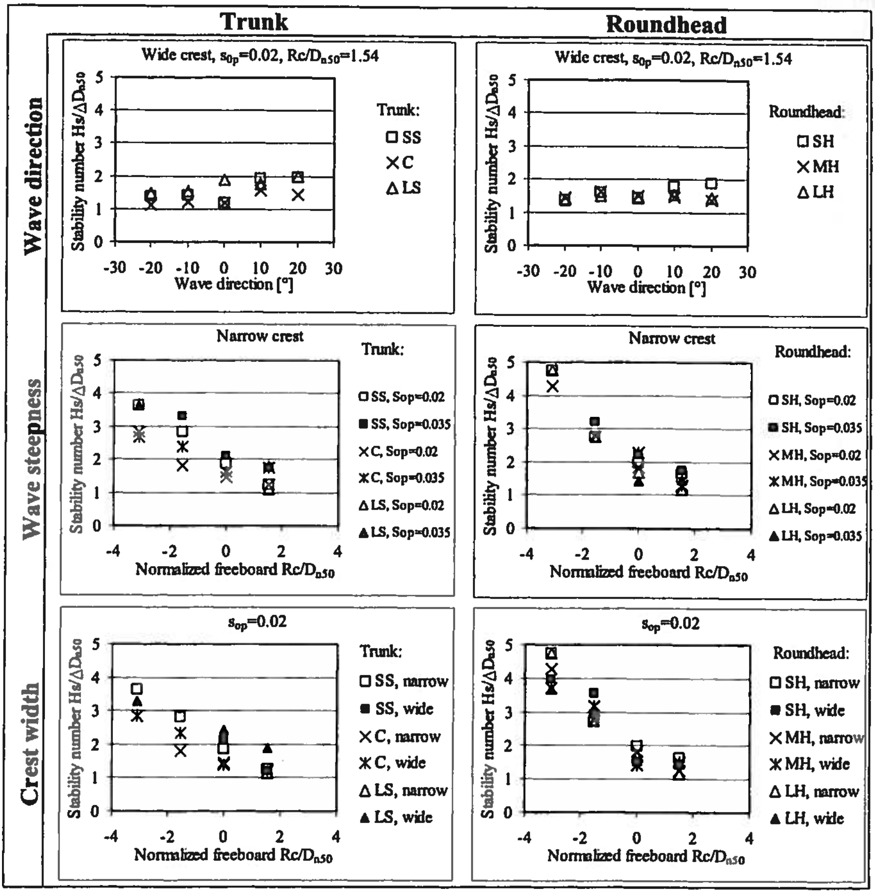


Figure 6. Results of new experiments for initiation of damage.

Wave direction. All parts of the trunk are slightly more stable under oblique wave attack than under normal incidence wave attack. The stability of the roundhead sections in case of oblique waves <0° (a large part of the head exposed to direct wave attack) is the same as for normal incidence waves. The stability of the leeward and middle part of the roundhead in case of oblique waves >0° (when a large part of the head is in lee of direct wave attack) is the same as for normal incidence waves, but the area of damage shifts towards the middle part of the head. During the experiments it was experienced that wave breaking tends to focus at the roundhead forming a jet of water slamming down on the top part of leeward head. This effect shifted towards the middle head in case of oblique waves causing the middle head more prone to damage.

Wave steepness. From Figure 6 it is seen that the data for s_{0p}=0.02 and s_{0p}=0.035 are fairly close. However, the series with s_{0p}=0.02 (long waves) tend to give slightly more damage than series with s_{0p}=0.035 (short waves) meaning the structure is more stable for s_{0p}=0.035.

Crest width. No significant difference in response can be identified for the tested crest widths indicating that for the tested range the influence of crest width is small.

Depth limited waves

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). In Figure 5 (bottom right) 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 (4). By inserting in (4) $\rho_r = 2.65 t/m^3$ corresponding to Δ=1.6, and Hs=0.6·h the curves in Figure 7 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 (4) to calculate the required D_{n50} and the following rule of thumb is found: D_{n50} = 0.29·Hc.

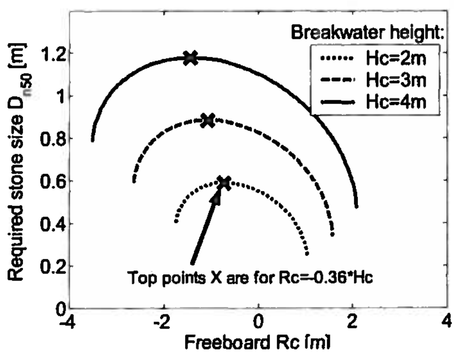


Figure 7. Design graphs for stability of low crested breakwaters corresponding to initiation of damage in case of depth limited waves.

Conclusions and guidelines

The results showed that the influence of crest width, wave steepness and obliquity of the waves is small within the tested ranges. The influence of the freeboard is large as a submerged structure is significantly more stable than an emerged low crested structure. For larger emergence than tested the overtopping will be reduced and consequently the trunk leeward slope and crest will become more stable.

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 (4) and (5) 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 5 or Figure 6. 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 5 (bottom right) given by (4). The formula is only valid for relatively low freeboards given by the ranges in (4).

$$\frac{Hs}{\Delta D_{n50}} = 0.06\left(\frac{Rc}{D_{n50}}\right)^2 - 0.23\frac{Rc}{D_{n50}} + 1.36 \qquad \text{, for } -3 \le Rc/D_{n50} < 2 \qquad (4)$$

In (4) 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:

$$D_{n50} = 0.29 \cdot Hc \qquad \text{, Hc is the structure height} \qquad (5)$$

According to (5) the structure height will be no more than 3 to 4 D_{n50}.

Acknowledgements
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