

CHAPTER 126

INTERACTIONS IN THE STABILITY OF TOE-BERM AND MAIN-ARMOUR FOR RUBBLE-MOUND BREAKWATERS : AN EXPERIMENTAL STUDY

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Abstract

This experimental study is concerned with one particular aspect of possible failure modes for a rubble-mound breakwater : the interactions between the toe-berm and the main-armour of the breakwater. Through a series of laboratory tests in a wave basin under long-crested and short-crested waves, we investigate the mutual influence of both these component parts of the breakwater on its general stability. The effects of several wave parameters are examined for four sizes of toe-berm stones. For the trunk section, experimental results are found to compare quite satisfactorily with existing design formulas both for the main-armour and the toe-berm. As a general trend from the tests results, the interaction processes appear to have only moderate effect. Their major feature is an increase of damage to the armour when the toe-berm is unstable. On the opposite, minor effects of main-armour on toe-berm stability were observed. In particular, the "toe-berm armouring process" (by units falling from the armour) appears to occur only marginally and under precise conditions.

1. INTRODUCTION - SCOPE OF WORK

The common practice for designing a breakwater is usually to use existing design formulas and rules for each individual part of the breakwater (main armour, rear armour, crest,...). This design approach is quite well documented in the scientific literature, in particular for the main armour (e.g. Van der Meer, 1992). In a following step, the whole breakwater profile has to be further tested and optimized through model tests in wave flume or basin, to obtain a reliable and safe structure.

The overall stability of the breakwater is however not only a function of the stability of each individual component part of the breakwater, but also of the interaction mechanisms between these parts. Present knowledge on the latter point is quite limited and it is not straightforward to find in the literature precise and quantitative descriptions of these possible interaction effects. Improving this knowledge was the main objective of the European Research Project "Rubble-Mound Breakwater Failure Modes (RMBFM)" of MAST-2. The study presented in this paper is concerned with one particular aspect of these possible failure modes : the interactions between the toe-berm and the main-armour of a breakwater.

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This experimental study continues previous tests performed in a wave flume at the University of Bologna (UB) in Italy (Lamberti and Aminti, 1994) within the same RMBFM project. In particular, Lamberti (1994) emitted the idea that, for sufficiently wide toe-berms, the main feature of this interaction process could be a phenomenon of "toe-armouring" by stones falling down from the damaged armour. If these stones remain on the toe-berm, they may increase its stability (lower damage to the toe-berm). Still addressing these possible interactions, we focus here more closely on three-dimensional effects, by considering the following aspects :

- wave obliquity (effect of angle of wave incidence)
- wave directionality (effect of angular spreading of wave energy)
- behaviour on the trunk section and at round-heads. Within the frame of this paper, attention will be mainly paid to the trunk section. Additional results for the round-head section may be found in Benoit and Donnars (1996).

In the present study, these effects are investigated through a series of laboratory experiments in a multidirectional wave basin. The experimental conditions and procedure are described in Section 2. Results for the main-armour and the toe-berm are presented and compared with other experimental results and existing design formulas in Section 3 and 4 respectively. The effects of the interaction processes are discussed in Section 5. The conclusion of Section 6 summarizes the main findings of this study and gives some recommendations for practical design.

2. EXPERIMENTAL SET-UP AND TEST CONDITIONS

2.1 Brief description of the wave basin used for laboratory tests.

The tests have been conducted in the multidirectional wave basin of Laboratoire National d'Hydraulique (LNH) in Chatou (France) (Figure 1). This experimental facility is dedicated to physical modelling applied to maritime and coastal studies. The overall dimensions of the basin are 54 m x 31 m x 1.3 m (maximum water depth for operational use : 0.80 m). This basin is fitted out with a multidirectional piston-type wave-maker composed of 56 paddles (segment width : 0.40 m).

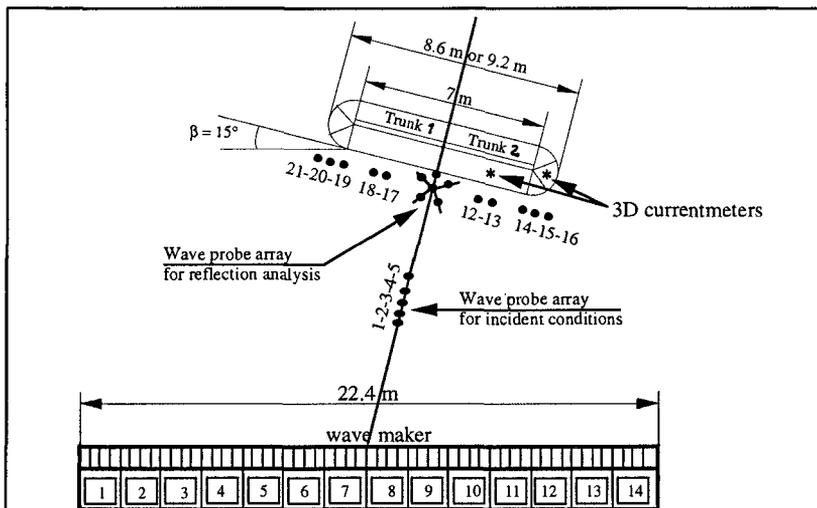


Figure 1 : General lay-out of experimental set-up in LNH wave basin.

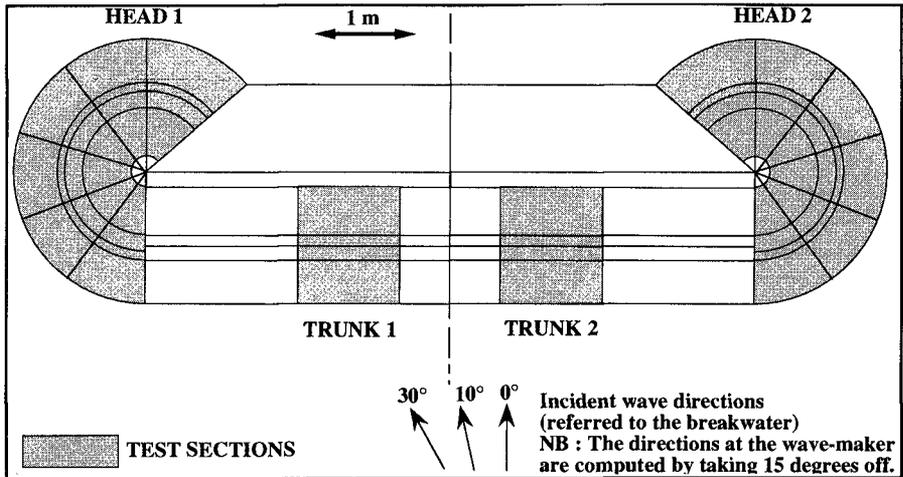


Figure 3 : Test sections on the breakwater and wave directions.

2.3 Choice of governing parameters and test programme.

The choice of varying governing parameters for the experimental tests is based on the analysis performed by Gerding (1993) and Lamberti (1994) :

- the **nominal diameter of the toe-berm stones** [4 values]. The first value corresponds to the "design" value as determined by a conventional design approach, whereas the other ones lead to "unstable" berms (see § 2.5).
- the **slope of the main-armour** [2 values : 1:1.5 and 1:2.5] (see §2.2).
- the **wave steepness** [2 values : $s_{om} = 2\%$ and $s_{om} = 5\%$]
The wave steepness is defined as : $s_{om} = H_s/L_{0m} = H_s/1.56T_m^2$. As "longer" waves ($s_{om} = 2\%$) are thought to be more severe for the stability of the breakwater than "shorter" waves ($s_{om} = 5\%$), most of tests are performed with the 2 % steepness for incident waves.
- the **angle of wave incidence** [2 values : $\beta = 0^\circ$ and $\beta = 30^\circ$]. One test has also been conducted under a $\beta = 10^\circ$ angle of wave incidence to check whether such a value could lead to higher damage, as shown by Galland (1994) for toe-berm stability at concrete armoured structures or by Juhl and Sloth (1994) for wave overtopping.
- the **angular spreading of energy** [2 values : $s = \infty$ (unidirectional waves) and $s = 15$]. The spreading index s corresponds to the exponent in the model of directional spreading function used for generating drive signals for the wave-maker $D(\theta) = 1/\Delta \cos^{2 \cdot s}(\theta)$. The short-crested case ($s = 15$) produces a directional sea-state with moderate angular spreading (directional width of about 10 degrees).

The test programme is based on the above choice of governing parameters. Due to this rather large number of parameters, it was not possible to test all the combinations between all parameters, which would have resulted in $4 \times 2 \times 2 \times 2 \times 2 = 64$ tests. This number has been reduced to 30 by considering only some of the above combinations (Benoit and Donnars, 1996). Because two half-breakwaters are tested simultaneously, the overall number of tests is finally equal to 15. Due to the fact that only three sections are tested under oblique wave, the total number of sections (trunk + round-head) examined during the experiments is equal to 52.

2.4 Review of incident wave conditions.

A JONSWAP-type spectrum is used for the distribution of wave energy over frequencies with a value of the peak enhancement factor $\gamma = 5$ for the tests at steepness $s_{om} = 0.05$ and a value of $\gamma = 1$ for the tests at steepness $s_{om} = 0.02$.

For a given test, the wave steepness is constant and the target wave height is increased by successive steps : each step corresponds to a run. A test is thus composed of 7 runs. The target wave characteristics for the experiments (in terms of significant wave height H_s and mean period T_m) are summarized in Table 1.

Each run has a duration of about 2000 waves of target mean period T_m .

run n°	$s_{om} = 0.05$ ($\gamma = 5$)		$s_{om} = 0.02$ ($\gamma = 1$)	
	H_s (m)	T_m (s)	H_s (m)	T_m (s)
1	0.060	0.88	0.045	1.20
2	0.071	0.95	0.053	1.30
3	0.082	1.02	0.061	1.40
4	0.093	1.09	0.069	1.49
5	0.104	1.15	0.077	1.57
6	0.115	1.21	0.085	1.65
7	0.126	1.27	0.093	1.73

Table 1: Target wave characteristics for model experiments (H_s , T_m).

2.5 Review of incident wave conditions.

• **Armour stones** : The following characteristics were obtained for the stones of the main armour (two layers) : Density=2.55 ; Nominal diameter $D_{n50a} = 2.91$ cm ; Nominal weight $W_{n50a} = 63$ g ; $\Delta.D_{n50a} = 4.51$). Same stones are used for the armour-layer both at the trunk sections and at the round-heads.

• **Toe-berm stones** : The "design" value of toe-berm stones for trunk section (labelled T1) was determined according to the design formula of Gerding (1993), leading to the following characteristics : Density=2.72 ; Nominal diameter $D_{n50t} = 2.58$ cm ; Nominal weight $W_{n50t} = 47$ g ; $\Delta.D_{n50t} = 4.44$)

From the design value of toe-berm stones, the three other values (termed T2 to T4) are computed from the relationship proposed by Lamberti (1994) :

$$W_{n50t}(T_j) = \frac{W_{n50t}(T_1)}{2^{K_j}} \quad \alpha \quad K_j = \ln_2 \left(\frac{W_{n50t}(T_1)}{W_{n50t}(T_j)} \right)$$

The values of K_j are chosen to be :

- toe-berm stones T2 : $K_2 = 1.2$ $W_{n50t}(T2) = W_{n50t}(T1) / 2.3 \approx 20$ gr
- toe-berm stones T3 : $K_3 = 2.5$ $W_{n50t}(T3) = W_{n50t}(T1) / 5.66 \approx 7.5$ gr
- toe-berm stones T4 : $K_4 = 4.5$ $W_{n50t}(T4) = W_{n50t}(T1) / 22.6 \approx 2.2$ gr

The weight of toe-berm stones at the round-head are increased by about 25 % from the above values determined for the trunk section.

The sorting index D_{85} / D_{15} for the various toe-berm stones lies in the range [2 ; 2.5]. It must be emphasised that this sorting index is much larger (but also more representative of natural conditions) than the value of 1.1 used during the flume experiments at UB (Lamberti, 1994).

2.6 Measurement and analysis of damage.

There is no rebuilding of the breakwater between consecutive steps of a test : cumulative damage is observed and reported during the experiments.

Damage is first evaluated by counting the number of units displaced from the armour-layer and the toe-berm. This is the standard method for measuring damage during the tests. For the trunk section, the damage index is given by the number N_{od} of units displaced within a strip of width equal to one D_{n50} :

$$N_{od} = N_d \frac{D_{n50}}{l} \quad \text{where } l \text{ is the width of the trunk section.}$$

An optical sensor has also been used for several tests in order to get information on the changes in the profile of the breakwater and to estimate the eroded area A_e in the breakwater profile. The damage level is then defined as : $S = A_e / D_{n50}^2$.

By combining present experimental results and previous observations (Van der Meer, 1992 ; Burcharth, 1993), the relationship : $S \approx 2 \cdot N_{od}$ is obtained and will be used further in the analysis of test results.

3. STABILITY OF THE MAIN-ARMOUR FOR THE TRUNK SECTION

3.1 Descriptive analysis of the stability of the main-armour of the trunk

We only report here the main observations from the tests. A more complete description and analysis of results may be found in Benoit and Donnars (1996).

3.1.1 Influence of wave direction and directionality (angular spreading)

Examples of test results for the stability of the main-armour are plotted on Figure 4 for four wave conditions and the four toe-berm stone sizes (armour-slope 1:2.5 only). In the range of tested values, the wave direction does not appear to have a significant effect on the stability of the main-armour as long as the toe-berm is stable (toe-berms T1 and T2). When the toe-berm is unstable, damage levels are higher on the armour layer and the normal direction seems to be more severe than an oblique (30°) direction.

Damage on the main-armour is higher under short-crested waves (only one tested value of angular spreading) than under long-crested waves for the same incident wave height. This is a clear observation from present experiments, which appears to be in some contradiction with other experimental test results. For instance, Thunbo Christensen *et al.* (1984) found from model tests on a breakwater armoured with quarry stones that uni-directional waves result in 30-40 % more damage to the breakwater when compared to short-crested waves. Canel and De Graauw (1992) also concluded that short-crested waves result in an increase of the stability number (from 0 to 60 %) for rock for the 1 % damage level. Although no definite conclusion may be drawn from the rather low number of present experiments, this increase of damage under short-crested waves appears quite clearly. It is assumed to be due to a rather low angular spreading of wave energy.

This increase of damage under short-crested waves is clearly observed for the normal attack, but seems to be quite feeble for the 30° direction.

3.1.2 Influence of wave steepness (long-crested waves ; armour slope 1:2.5)

The effect was examined for toe-berms T1 and T2 only. For the same wave height, it is observed that the "longer" waves ($s_{om} = 2$ %) clearly result in more damage to the armour-layer than the "shorter" waves ($s_{om} = 5$ %).

3.1.3 Influence of toe-berm stones weight (long-crested waves only)

For the normal wave attack, a clear increase of damage to the main-armour with decreasing toe-berm stone weight is observable. For toe-berm stones T1 and T2, damage to the armour remains at an acceptable and comparable level. But for lighter stones (toe-berm stones T3 and T4), the main-armour is significantly more damaged. Severe damage occurs for the lightest toe-berm stones T4.

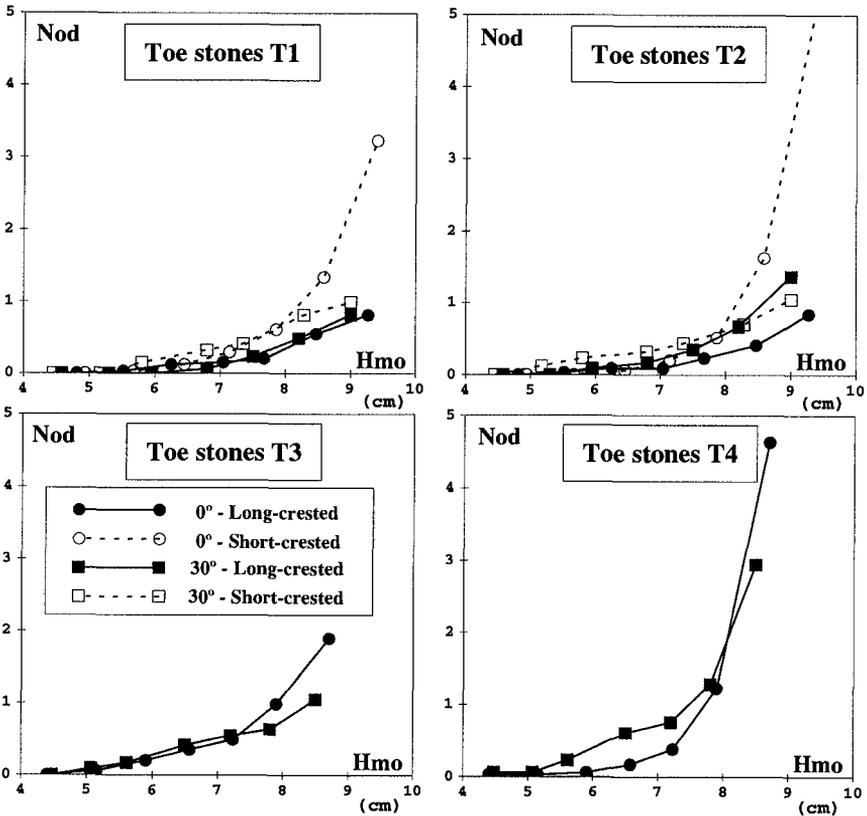


Figure 4 : Evolution of observed damage to the main-armour for four wave conditions (armour-slope 1:2.5 only).

3.2 Synthetic analysis of the stability of the main-armour of the trunk

In order to compare our test results to existing design formulas, we consider the Van der Meer design formulas (Van der Meer, 1988) for the armour layer :

$$\text{Plunging waves } (\xi_m < \xi_{mc}) : \frac{H_s}{\Delta \cdot D_{n50a}} = 6.2 \frac{P^{0.18}}{\sqrt{\xi_m}} \left(\frac{S}{\sqrt{N}} \right)^{0.2}$$

$$\text{Surging waves } (\xi_m > \xi_{mc}) : \frac{H_s}{\Delta \cdot D_{n50a}} = 1.0 P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cotg \alpha} \xi_m^P$$

where N is the number of waves in a storm or in the test (N=2000), S is the damage index defined from the eroded area A_c (here, S is computed from N_{od} by : $S = 2 \cdot N_{od}$; see § 2.6), ξ_m is the surf-similarity parameter defined from the mean wave period T_m , P is the notional permeability factor (taken to be 0.4 as suggested by Van der Meer (1988) for a permeable core with a filter and an armour composed of two layers of natural rocks). ξ_{mc} is the critical value of surf-similarity parameter ξ_m , determining the transition from plunging waves to surging waves :

$$\xi_{mc} = \left[6.2 P^{0.31} \sqrt{\tan \alpha} \right]_{P+0.5}^{-1}$$

The values surf-similarity parameter ξ_m depend on both the steepness (two possible values) and the slope of the armour (two possible values). From these

values, it appears that tests performed for the 1:1.5 slope are of "surging" type whereas they are of "plunging type" for the 1:2.5 slope (and 2 values of steepness).

The tests results are plotted and compared to Van der Meer stability formulas on Figure 5.a (plunging waves) and 5.b (surging waves). Experimental points from Van der Meer (1988), Galland (1994) and Lamberti (1994) are also superimposed.

— Analysis of the "plunging waves" tests (figure 5.a) :

An acceptable agreement is found between experimental points and the formula. However, one can observe that the lower values of damage are overpredicted by the formula, whereas the higher values seem to be underpredicted. This is in particular quite clear for the tests with 5 % steepness, which may indicate that the effect of steepness is not perfectly resolved in the Van der Meer formula for plunging waves. If we consider the experimental points from Van der Meer (1988) for permeable core, we again note that most of points lie above the design curve.

— Analysis of the "surging waves" tests (figure 5.b) :

Based on present experiments, the general trend of the formula is to overpredict the observed damage levels. The use of Van der Meer formula on these experiments thus seems to lead to a conservative design of the armour. Test results from Lamberti (1994) also confirm this trend in spite of a rather high scatter. However, one should also note that Van der Meer formula appears to be a good fit to the Van der Meer (1988) test results obtained with a permeable core. Additional analysis and comparisons should be performed in order to check this point. In particular, it was observed that the permeability factor P may play a significant role in the formula and the precise determination of P for an existing breakwater is not straightforward.

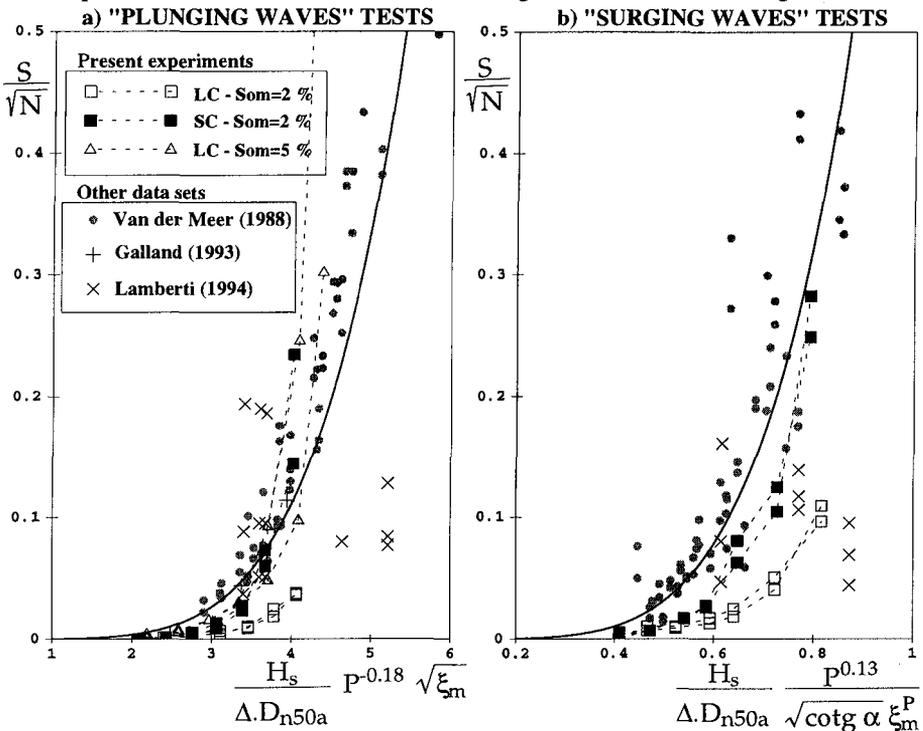


Figure 5 : Comparison of present test results with Van der Meer (1988) formula for the stability of the main armour (tests under normal wave attack).

4. STABILITY OF THE TOE-BERM FOR THE TRUNK SECTION

4.1 Descriptive analysis of the stability of the toe-berm of the trunk

4.1.1 Influence of wave direction and directionality (angular spreading)

Examples of test results are plotted on Figure 6 for four wave conditions and the four toe-berm stone sizes (armour-slope 1:2.5 only). From the tests, we observe that the normal direction seems to be usually more severe than the 30° direction.

Damage to the toe-berm is higher under short-crested waves than under long-crested waves only for the normal attack, whereas long-crested waves produce more damage for the 30° direction.

The toe-berms are less stable for the slope 1:2.5 slope of armour than the same ones associated with the 1:1.5 slope. This may be related to slight different hydrodynamical conditions of wave breaking and run-down, resulting in stronger action to the toe-berm.

4.1.2 Influence of wave steepness (long-crested waves ; armour slope 1:2.5)

The longer waves ($s_{om} = 2\%$) create more damage to the toe-berm than shorter waves ($s_{om} = 2\%$), but this trend is stronger than for the damage to the main-armour (see § 3.12). This indicates that the toe-berm is more sensitive to the steepness of incident waves than the main-armour.

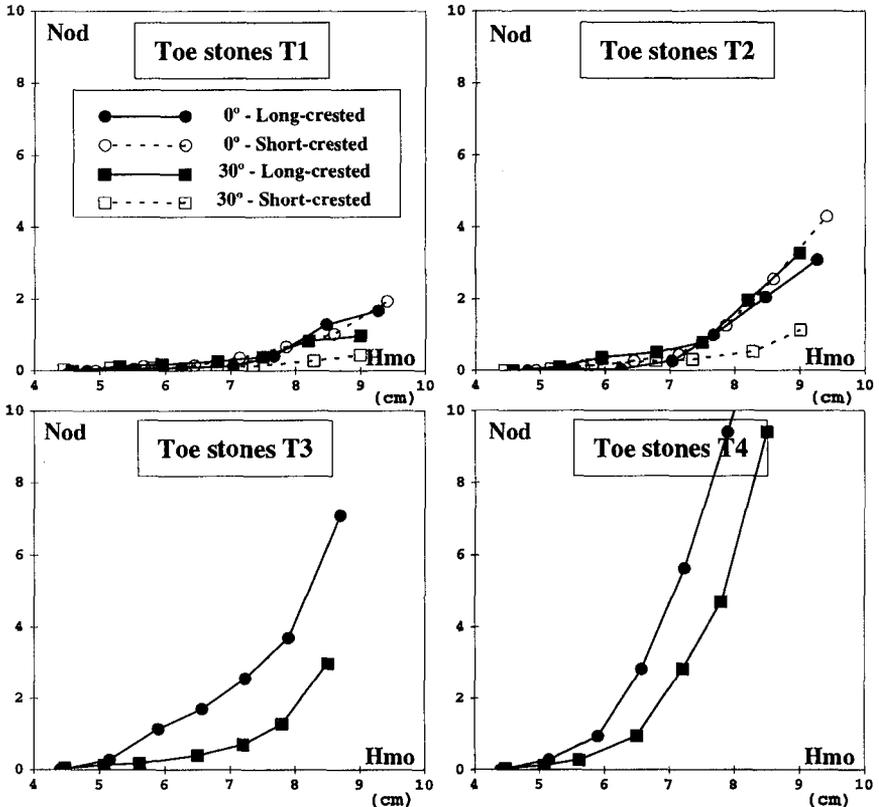


Figure 6 : Evolution of observed damage to the toe-berms for four wave conditions (armour-slope 1:2.5 only).

4.1.3 Influence of toe-berm stones weight (long-crested waves only)

For the normal wave attack, damage to the toe-berm increases with decreasing toe-berm stone weight. The effect of the toe-berm stone size is clearly more sensitive on the damage to the toe-berm itself than on the damage to the main-armour. However, one may distinguish two different behaviour : toe-berms T1 and T2 are quite stable with low damage levels, whereas toe-berms T3 and T4 are clearly unstable with higher damage levels.

4.2 Synthetic analysis of the stability of the toe-berm of the trunk

In this section, we consider the formula established by Gerding (1993) for the stability of the toe-berm of a rubble-mound breakwater :

$$\frac{H_s}{\Delta \cdot D_{n50t}} = \left(0.24 \frac{h_t}{D_{n50t}} + 1.6 \right) N_{od}^{0.15}$$

where Δ is the relative buoyant density of toe-berm stones ($\Delta = \rho_r / \rho_w - 1$), D_{n50t} is the nominal diameter of stones composing the toe-berm, h_t is the depth of toe-berm below the Mean Water Level and N_{od} is the damage index to the toe-berm.

The damage level is classified as : $N_{od} = 0.5$: hardly any damage ; $N_{od} = 2$: acceptable damage (design criteria) and $N_{od} = 4$: unacceptable damage.

On Figure 7, the experimental data points plotted on the graph are composed of all the present experiments conducted with normal wave attack and long-crested waves (including 2 slopes of main-armour, 4 toe-berm stone sizes and 2 values of wave steepness). On this figure, an acceptable agreement between present experiments and the formula from Gerding (1993) is obtained. However, one must note that the effects of mound-slope and wave-steepness are not included in Gerding's formula. Although these effects do not appear as dominant on experimental points, it is possible to distinguish on Figure 7 the data points for the 1:1.5 slope (triangles) and for the 1:2.5 slope (circles). The effect of steepness is more sensitive (compare crosses and circles for toe-berm stones T1 and T2), indicating that damage to the toe-berm appears to be lower for the "shorter" waves than for the "longer" waves. The inclusion of these effects in an extended formula appears as an interesting research item.

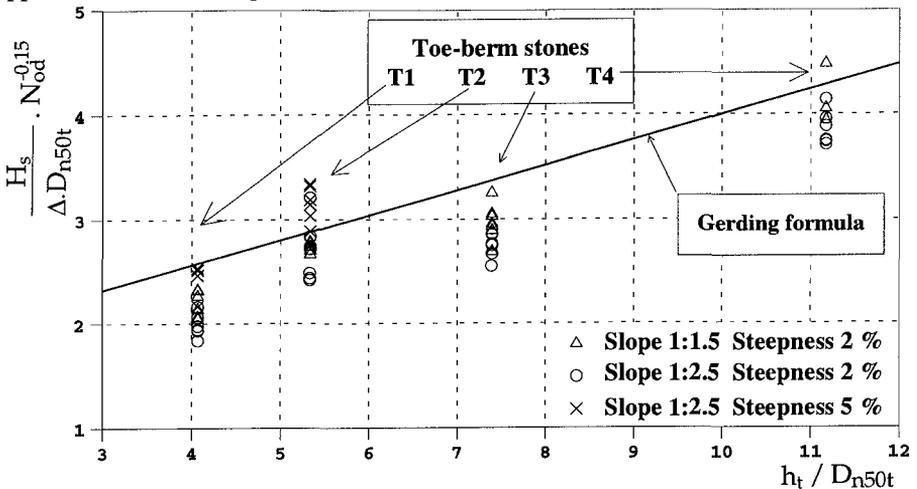


Figure 7 : Comparison of present test results with Gerding (1993) formula for the stability of the toe-berm (tests under normal wave attack only).

5. DISCUSSION OF THE INTERACTION PROCESSES

During the experiments, the following points have been observed :

- For most of tests, as the wave height increases between each run, damage first appears on the toe-berm and then on the main-armor. This is not really surprising, as the main-armor is made of stones whose weight is determined from "standard" design value whereas the toe-berm stones weight are equal (toe-berm stones T1) or lower than the design value (toe-berm stones T2 to T4). It is thus quite a "normal" behaviour that toe-berms are damaged first.
- The stability of the toe-berm continuously decreases as the weight of toe-berm stones decreases. The stability of the main-armor is also decreasing with the size of toe-berm stones (in particular for toe stones T3 and T4).
- Damage to the main-armor for the trunk section is mainly located between the toe (- 0.10 m referred to MWL) and about + 0.05 m referred to MWL.

In order to describe the interaction processes between the main-armor and the toe-berm, we start from the classification of evolution mechanisms proposed by Lamberti (1994). We further tentatively propose a correlation diagram between damage to toe-berm and damage to main-armor where the evolution processes (A, a, B, b, c) from Lamberti (1994) are schematically summarized (figure 8).

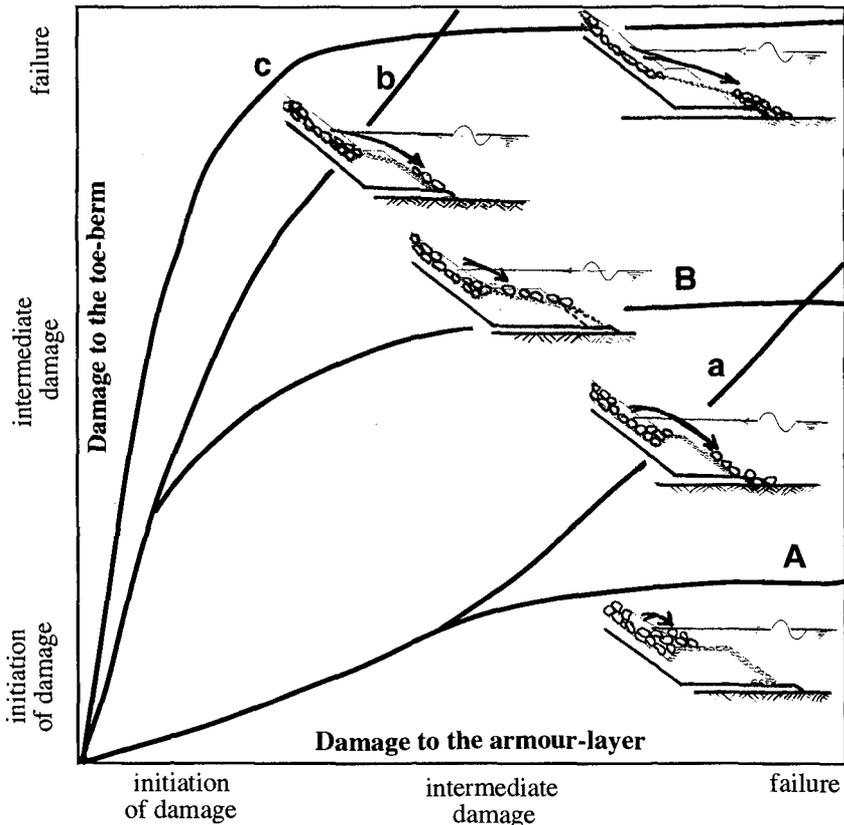


Figure 8 : Tentative analysis of correlation between damage to armour and damage to toe-berm after a classification of mechanisms from Lamberti (1994)

Based on the data from our experiments, it is possible to build such plots for the four toe stones used and for different wave conditions (Figure 9). On these plots, we can directly compare the synthetic evolution of the observed interaction process to the classification of evolution processes of figure 8. The main comments raising from this analysis are summarized below :

• **Toe-berm stones T1 and T2 : evolution process : a**

The toe-berm is rather narrow, but quite stable for the various wave conditions. Damage appears approximately at the same time on the main-armour and the toe-berm. There is no significant effect of toe-armouring process. As the toe-berm is quite narrow, the stones falling down from the armour do not stop on the toe-berm.

• **Toe-berm stones T3 : evolution process : b**

The toe-berm is less stable than the armour layer and is damaged first. It thus becomes narrower and is then not very effective in retaining armour units when damage further appears on the main armour.

• **Toe-berm stones T4 : evolution process : c**

The toe-berm is severely unstable and the static support of the armour layer fails when the toe-berm is fully damaged. The armour layer may then slide down abruptly, exposing the underlayer.

In particular, the evolution process B (stones falling down from the armour layer stop on the toe-berm and enhance its stability : toe-armouring process) only rarely occurred and then exhibited quite feeble effect on the stability of the toe-berm. Most

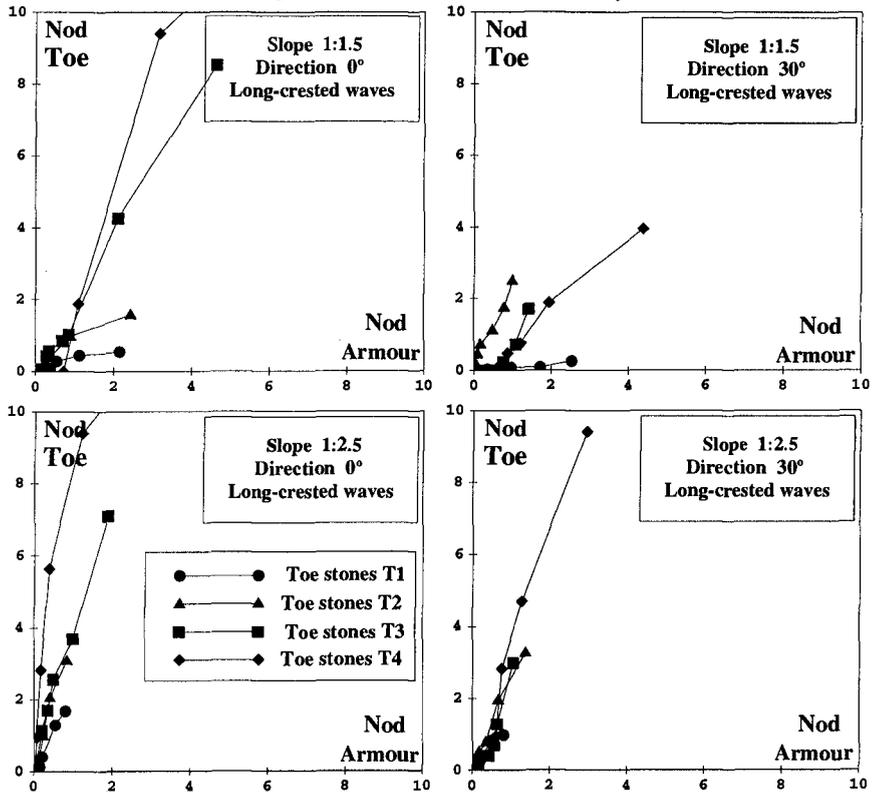


Figure 9 : Correlation diagrams based on present experiments for various conditions

of stones falling from the armour went directly to the floor of basin either because the berm was too narrow (toe-berm stones T1 and T2) or because it was already significantly damaged when damage started on the main-armour (toe-berm stones T3 and T4). For the breakwater profiles tested in the present study, there thus appears to be only a weak coupling in the stability of the main-armour and the toe-berm for the trunk section. It is suspected that the considered toe-berms were either too unstable or too narrow for the occurrence of the "toe-armouring" process. Such a process could maybe be observed with a wider and still quite stable toe-berm, but tests in this direction remain to be performed.

6. CONCLUSIONS

The major observations and conclusions raising from present experiments are **very briefly** summarized below for the trunk section of the breakwater :

- Effect of wave incoming direction : No definite effect emerges from the tests results for the main-armour. For the toe-berm however, the normal direction seems more severe than the 30° direction, at least for long-crested waves.
- Effect of wave directionality : More damage to the main-armour and to the toe-berm is observed under short-crested waves, in particular for normal wave attack. This point is rather in contradiction with previous experiments (Thunbo Christensen *et al.*, 1984 ; Canel and De Graauw, 1992), but is rather clear from present tests. It is supposed to be related to a quite low angular spreading of energy, which could be more severe for the stability. This point has however to be addressed by additional tests.
- Effect of wave steepness : Among the two tested values of steepness (2 % and 5%), the one corresponding to "longer" waves results in more damage both to the armour-layer and to the toe-berm. However, the toe-berm appears to be more sensitive to the steepness of incident waves than the main-armour.
- Effect of toe-berm stones size : when the toe-berm is "stable" (T1 or T2), the stability of the armour is not significantly affected by the stability of toe-berm. However, when the toe-berm is unstable (T3 or T4), higher damage is observed on the main-armour, leading sometimes to its failure. As expected, damage to the toe-berm increases as the toe-berm stone size decreases.

Comparing present results with existing design formulas, acceptable agreement was observed with the Van der Meer formulas for the stability of the main-armour. However some differences were also noted, in particular for the "plunging waves" formula : the lower values of damage to the armour-layer are overpredicted by the formula, whereas the higher values seem to be underpredicted. The Van der Meer formula for "surging waves" appears to lead to a somewhat conservative design. For the toe-berm, the formula from Gerding (1993) lies in acceptable agreement with present results, although the effects of wave steepness and mound-slope (not included in the formula) slightly increase the scatter of experimental points.

As a matter of conclusion, it appears from the present tests that the major feature of the interaction process between main-armour and toe-berm is an increase of damage to the armour when the toe-berm is unstable. This may lead to a total failure of the armour if the toe-berm is sufficiently eroded to fail in providing static support to the armour-layer. On the opposite, minor effect of main-armour on toe-berm stability was observed. In particular the "toe-armouring" process appeared to occur only marginally and under precise conditions. This process is thus regarded as a particular mechanism, which will occur only under specific conditions (precise relative stability of the toe-berm and the armour-layer, wide berm, ...).

As the effects of interaction processes between main-armour and toe-berm appear quite weak (unless toe-berm stones are significantly lighter than their design value), it appears both more natural and safe to use state-of-the-art stability formulas to establish a first design of both the toe-berm and the armour-layer independently. The full breakwater profile should then be tested in a wave flume, or preferably in a wave basin with a correct representation of the actual bathymetry in order to validate its design.

7. ACKNOWLEDGEMENTS

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