# CHAPTER 153

# TOE STABILITY OF RUBBLE MOUND BREAKWATERS

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## ABSTRACT

*At the Congress on Coastal Structures and Breakwaters, a stability relation for toe structures of rubble mound breakwaters was presented, see Van der Meer et al.,1995. In that relation the relative stone density, Δ, was used in the stability parameter $H_s/\Delta D_{n50}$. In the tests on which the relation was based, however, Δ was not varied, so some uncertainty remained on the influence of this parameter. Additional tests were therefore carried out with different stone densities, leading to the conclusion that there is no influence of the stone density, other than represented in the stability parameter $H_s/\Delta D_{n50}$. Computations were done, coupling the orbital velocities in the waves in front of the breakwater to the stability of the stones of the toe structure leading to encouraging results. Both computations and tests show an influence of the waterdepth in front of the breakwater which is not yet included in the stability relation.*

## 1. INTRODUCTION

Figure 1 shows a typical cross-section of a rubble mound breakwater, consisting cf several layers, covered with an armour layer bordered by a toe structure. The function of the toe is mainly to support the armour layer, and to provide a transition to low(er) weight units in the base of the structure. Data about the stability of the toe are less in number than about the stability of armour layers. That is the reason to attach special attention to this part of the structure.

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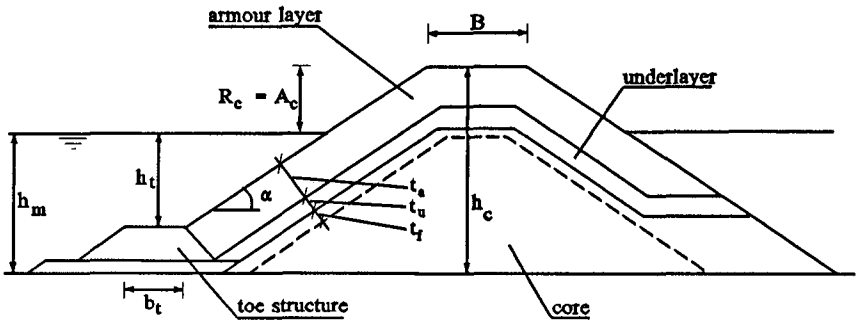


**Figure 1** Cross-section rubble mound breakwater

Stability of the toe was traditionally related to the stability of the armour layer. For values of $h_t/H_s = 1.5$, a minimum weight of the units in the toe was given as W/2. For a greater submergence, with values of $h_t/H_s > 2$, toe unit weight could be reduced to W/10 to W/15 (Shore Protection Manual, 1984). These design rules were attractive because of their simplicity. They were, however, not based on extensive research or comprehensive theoretical considerations.

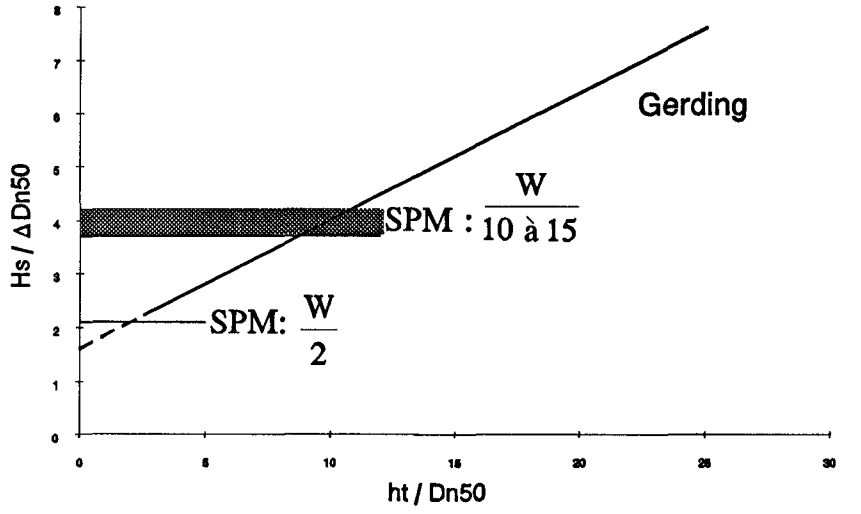


**Figure 2** Existing relations for toe stability

Several other investigations on the stability of breakwater toes have been done. Van der Meer et al., 1995 presented the following expression, based on the MSc work of Gerding at Delft University of Technology (DUT):

$$\frac{H_s}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6\right) N_{od}^{0.15} \quad (1)$$

in which $N_{od}$ is defined as the number of stones removed from the toe structure divided by the number of stones in a strip with a width of one $D_{n50}$ and a length equal to the width of the test section (Van der Meer,1993)

In this equation, the submergence of the toe is expressed as $h_t/D_{n50}$ instead of $h_t/H_s$, since that gave a better fit with the experimental data. Figure 2 shows equation (1) for $N_{od} = 0.5$ (threshold of motion), together with the values from the Shore Protection Manual.

The stability parameter, $H_S/\Delta D_{n50}$, in equation (1) contains the relative density of the stones of which the toe structure consists. In the tests on which the equation is based, however, $\Delta$ was constant. The question therefore remains whether the influence of $\Delta$ is repesented correctly in the equation. For another MSc-thesis, see Docters van Leeuwen,1996, tests were done with a range of stone densities in order to check this.

The second item in this paper is an attempt to link the experimental results to the wave motion in front of the breakwater. Hydraulic engineering knows (too) many empirical relations. This makes application without sufficient knowledge of the backgrounds dangerous. The hydraulic engineering department at DUT has a policy, for reasons of didactics, to try to relate as much as possible stability of stones to the water motion.

## 2. EXPERIMENTS

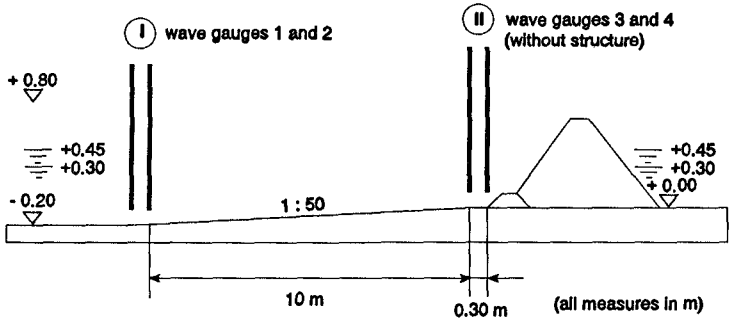


**Figure 3** Test set-up in wave flume

The tests were performed in a wave flume (length 40 m, width 0.8 m and depth 0.9 m) at the Laboratory of Fluid Mechanics at Delft University of Technology (DUT). An overview of the test set-up is given in Figure 3. In the flume a fixed bed foreshore with a slope of 1:50 was constructed. The length of the foreshore was 10 m. The rubble mound breakwater and toe structure were placed on a horizontal bed at a distance of 0.3 m from the end of the foreshore.

Irregular waves were generated according to a JONSWAP-spectrum. During a test the significant wave height was increased in 4 steps from about 0.1 m up till about 0.2 m. The peak period to match followed from the selected steepness $s_{0p}$. In the research of Gerding the influence of the wave steepness on the stability of the stones in the toe appeared to be small (due to the selected steep front slope giving little variation in reflection with the wave steepness, hence in load on the toe). Therefore the steepness was kept constant at a value $s_{0p} = 0.04$.

The wave heights used in the analysis are the measured wave heights at the beginning of the foreshore, reduced with the reflection and adapted for shoaling due to the sloped foreshore. The in this manner processed wave heights are indicated as incoming wave heights. The wave generator was provided with a reflection compensation. During the tests an average value of the reflection coefficient $C_r = 0.25$ was found with a minimum of 0.185 and a maximum of 0.315.

The breakwater was constructed with rubble according to Figure 4 to create a porous structure with a reflection coefficient similar to the one in prototype. The front slope was kept constant (2:3).

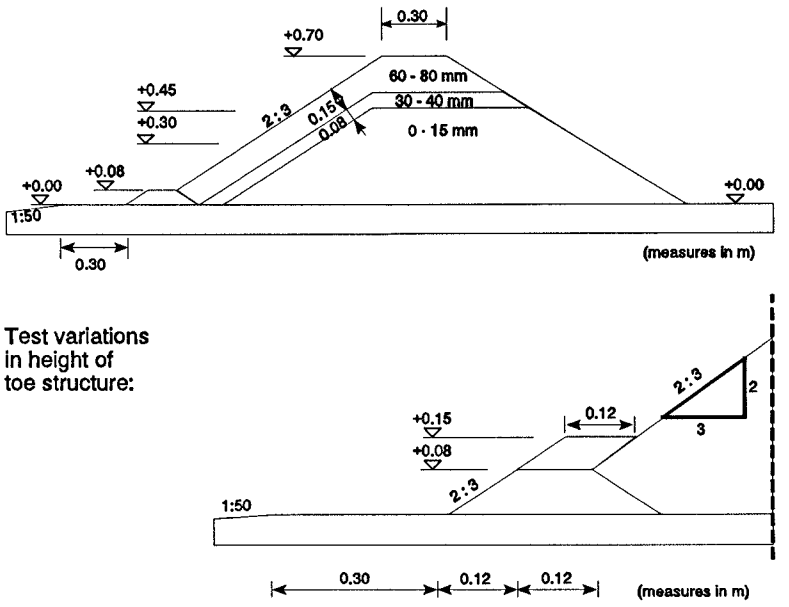


**Figure 4** Cross-sections tested breakwater with toe

The tests by Gerding showed little to no influence of the width of the toe, therefore the toe structure width, $b_t$ (perpendicular to the length axis of the breakwater) was kept constant (0.12 m). Two toe heights were used in the tests, namely $z_t$ = 0.08 m and 0.15 m. Each toe height was tested for two waterdepths $h_m$ = 0.30 m and 0.45 m in front of the toe structure (0.50 m and 0.65 m at the beginning of the foreshore respectively). For the rubble of the toe structure three different materials with varying size were used:

| Material | $\rho$ (kg/m$^3$) | $D_{n50}$ (mm) | $D_{n85}/D_{n15}$ |
|---|---|---|---|
| basalt | 2850 | 10.2 | 1.4 |
| basalt | 2850 | 15.1 | 1.2 |
| porphyry | 2550 | 9.8 | 1.34 |
| porphyry | 2550 | 14.4 | 1.31 |
| porphyry | 2550 | 21.0 | 1.26 |
| brick | 1900 | 23.1 | 1.45 |

The use of 6 stone types ($\rho$, $D_{n50}$), 2 waterdepth, 2 toe heights and 4 different wave heights led to a total number of 96 tests.

After each test the damage was determined by counting and weighing the total number of stones removed from the toe structure in the seaward direction. The damage is expressed with the damage number $N_{0d}$.

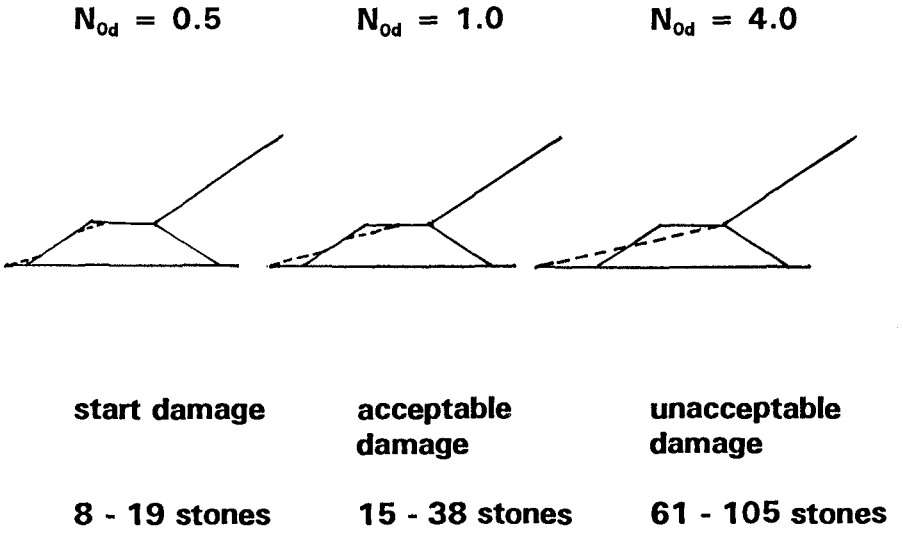


**Figure 5** Sketch of damage for various values of $N_{0d}$

Figure 5 shows various damage stages. $N_{0d} = 0.5$ was chosen as the threshold of motion in this study.
N.B. Gerding found in his tests no influence of the berm width on the damage level. Of course, the acceptability of damage does depend on the berm width.

## 3. RESULTS

For the 96 tests, mentioned in the previous section, the damage results were plotted against the (incoming) wave height. Figure 6 shows an example of these results.

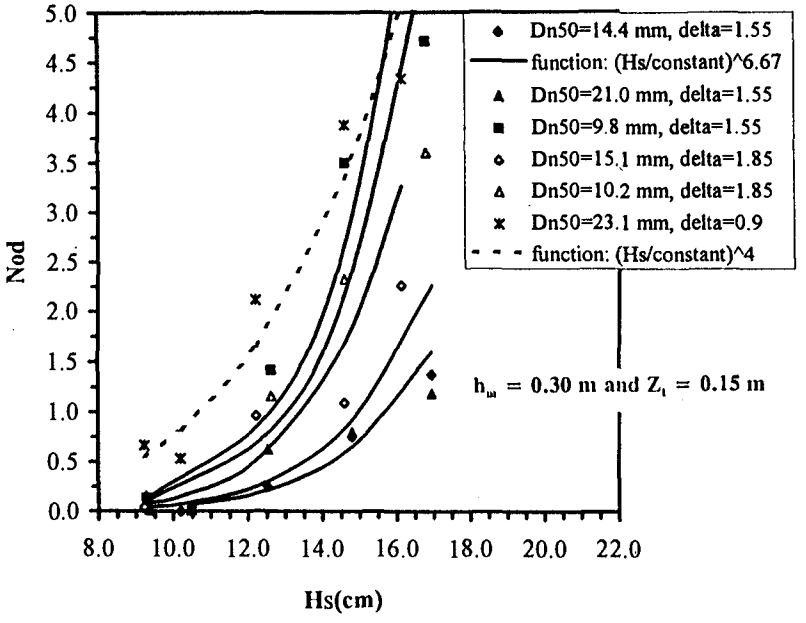


**Figure 6** Example of damage as function of wave height

First, to check the influence of the density in the Gerding formula, $(H_S/\Delta D_{n50})/N_{0d}^{0.15}$ was plotted against $h_t/D_{n50}$. The results showed considerable scatter, making it difficult to draw a firm conclusion on the influence of $\Delta$ in the equation. Figure 7 shows the results limited to the threshold of motion, $N_{0d}$ = 0.5. There is no clear distinction in this graph between the result points for the various densities. This justifies, for practical purposes, the use of $H_S/\Delta D_{n50}$ as a single stability parameter, indicating that there is no secundary effect of $\Delta$ on the stability (of course, there is a primary effect of $\Delta$ via the stability parameter).

In a more detailed investigation into the scatter of the results as presented in Figure 7, the data was also sorted out for toe height and waterdepth with the same horizontal and vertical axis. The toe height appeared to give no difference, but the waterdepth $h_m$ shows definitely two different lines, see Figure 8. This makes that $h_m$ should be involved, actually, in the stability relation, equation (1), although the differences are small.

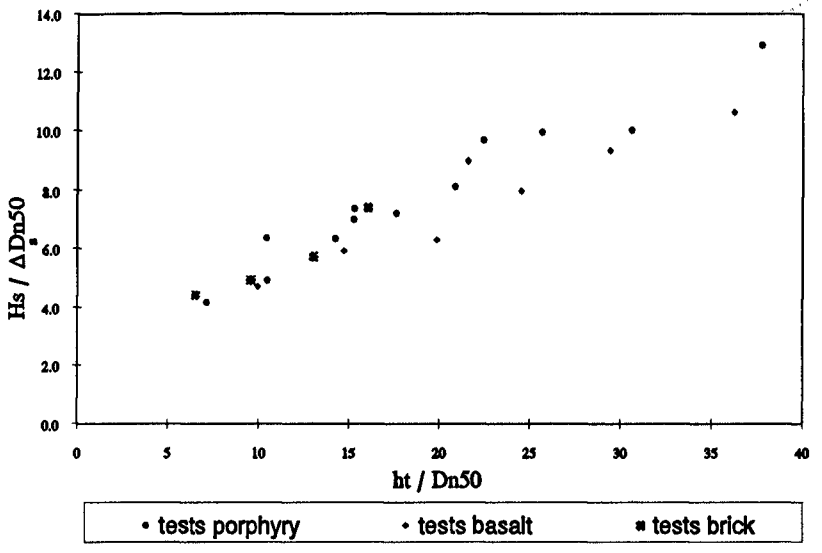


**Figure 7** Stability sorted out to stone density

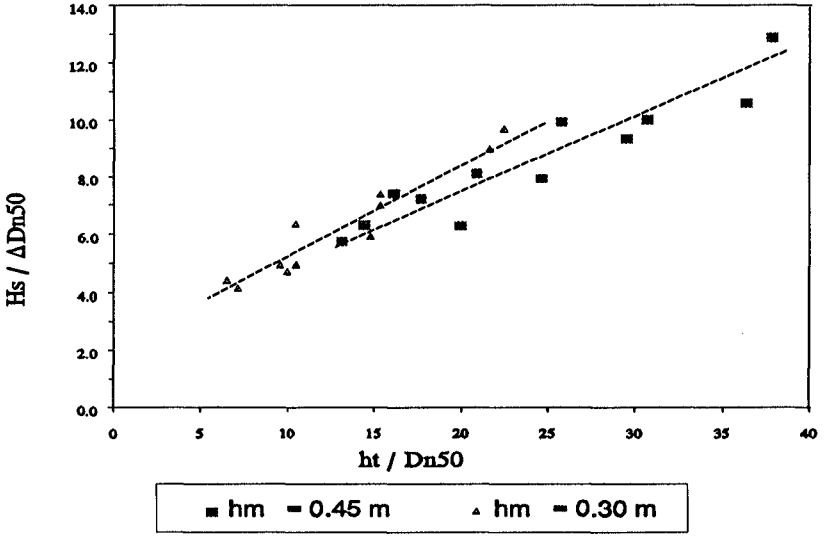


**Figure 8** Stability sorted out to water depth $h_m$

## 4. COMPUTATIONS

Empirical relations like equation (1) are only valid for the range of parameters that has been investigated. Application outside that range can be dangerous. Closely related to this drawback is the fact that empirical relations contribute very little to the understanding of the phenomena involved. In an attempt to understand more of the mechanisms behind the stability of toe structures and to extend the range of applicability of existing relations, computations were done, in which the orbital wave motion was linked to the stability of stones in the toe of a breakwater.

The orbital motion at the toe surface (at $h_m - h_t$) is derived from the linear wave theory:

$$\hat{u}_h = \frac{\omega H}{2} \frac{\cosh k(h_m - h_t)}{\sinh k h_m} \tag{2}$$

where the hat denotes the amplitude of the orbital velocity. The stone stability in orbital motion is approximated with the experimental results of Rance & Warren,1968, see also Schiereck et al.,1994:

$$\frac{a_h}{T^2 \Delta g} = 0.025 \left[\frac{a_h}{D_{50}}\right]^{-\frac{2}{3}} \tag{3}$$

in which $a_h$ is the orbital stroke at the breakwater toe, which is equivalent to $\hat{u}_h/\omega$.

Both equations need some adjustment. The orbital motion at the toe is not only due to incoming waves, but is also influenced by reflection from the breakwater. Furthermore, incipient motion will not be caused by $H_S$, but by a larger wave in an irregular wave field, e.g. $H_{1\%}$. Equation (2) is therefore rewritten into:

$$\hat{u}_h = \frac{\omega (1 + C_r) F * H_S}{2} \frac{\cosh k(h_m - h_t)}{\sinh k h_m} \tag{4}$$

in which F is a "tuning" factor.

In the tests by Rance & Warren, the sieve diameter D is used, while in the investigations by the authors, the nominal diameter $D_n$ is used, which is somewhat smaller. With $D_{n50} \approx 0.84 * D_{50}$ and $a_h = \hat{u}_h/\omega$, equation (3) becomes:

$$D_{n50} = \frac{2.15 * \hat{u}_h^{2.5}}{\sqrt{T_P} * (\Delta g)^{1.5}} \tag{5}$$

With these two equations the necessary stone size was computed for incipient motion in the experiments ($N_{0d}$ = 0.5). The accompanying, measured, incoming wave height $H_s$ was used, together with the measured reflection, $C_r$. The resulting, computed, diameters, together with the other relevant parameters from the tests, were adapted to the same dimensionless parameters as used with the test results, $H_s/\Delta D_{n50}$ and $h_t/D_{n50}$. Figure 9 shows the results.

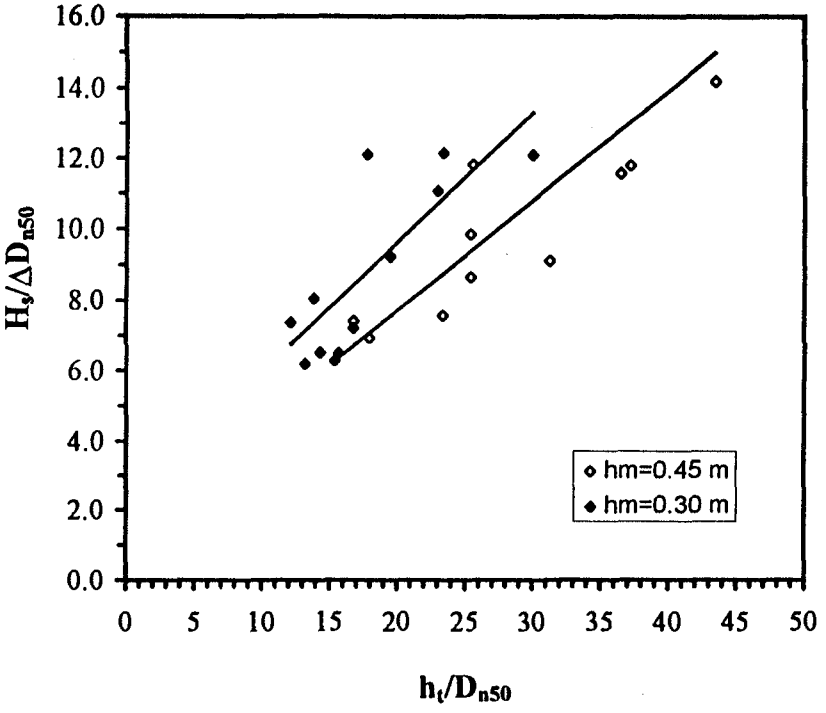


**Figure 9** Results of computations

The tuning factor, necessary to give more or less equal results as the tests, appeared to be 1.7. This means that, assuming a Rayleigh distribution, $H_{0.5\%}$ is responsible for incipient motion, which seems reasonable. This, together with the trend of the computational results and the fact that the same influence of $h_m$ is found in computations and experiments, is encouraging.

## 5. DISCUSSION

Stability of any structure is the relation between load and strength, in this case expressed as $H_s/\Delta D_{n50}$. The strength of a stone on the toe of a breakwater is expressed as $\Delta D_{n50}$. This is, of course, a simplification of the strength mechanism in which e.g. also friction between the stones plays a role. The idea, however, exists that this is reasonably satisfying. The load is expressed as $H_s$ which is a much more far-reaching simplification.

Stones in the toe of a breakwater move because of lift, drag and shear forces which again come from the velocity field in waves in front of the breakwater. This is a highly complex motion, even in a regular wave system, while, moreover, waves in nature are irregular. To this background it seems almost naive to try to catch the stability of stones in waves in simple formulas like equation (1).

Waves approaching a breakwater deform, mainly due to shoaling, bottom friction, reflection and breaking on the slope of the breakwater or the foreshore. All these processes can not be expressed by just including $H_s$ as the single load parameter. The reflection, for example, plays definitely a role in the load on the toe, but is not used in equation (1), since the incoming wave is regarded as the load. This seems logically, since the incoming wave is the boundary condition as offered by nature. The reflection is a response of the structure, but any difference in this response is not expressed in the equation, so it will appear as scatter in the experimental results.

The only way to include these aspects, is a model in which the complete water motion is represented correctly. This is, however, not very practical. The procedure followed in this paper can be seen as a compromise between simplicity and correctly representing the physical processes involved. The path is promising enough to walk on. Results will not only be easier to understand and to explain, but there will also be a wider application. An example may be the design of a foundation sill under a vertical, caisson-type, breakwater, which is loaded in a very similar way.

# 6. CONCLUSIONS AND RECOMMENDATION

## CONCLUSIONS

1. Tests in which the stone density is varied between $\rho = 1900$ and 2850 kg/m$^3$ show that there is no effect of the density on the stone stability in the relationship as proposed by Gerding:

$$\frac{H_s}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6\right) N_{od}^{0.15} \qquad (6)$$

other than expressed in the stability parameter $H_s/\Delta D_{n50}$.

2. Simple computations, relating the orbital wave motion to the stone stability on the toe structure, give promising results.

3. Both computations and experiments show an influence of the waterdepth in front of the breakwater on the stability, which is not expressed in the Gerding's relationship.

## RECOMMENDATION

It is worthwile to put more effort into computations. Equations like Gerding's stability relationship have both the advantage and disadvantage of simplicity. The advantage is obvious, the disadvantage is that it is impossible to capture all aspects of the physical reality in one equation. Possibly a combination of hydrodynamic equations, describing the velocity field in the waves and experimental results, describing the relationship between the velocity field and the stone stability, leads to a relatively simple, but more comprehensive, model. Such a model, including the wave reflection caused by any structure, could be applied not only for toes of rubble mound breakwaters, but also for foundations of caisson type, vertical breakwaters.

## SYMBOLS

| | | | |
|---|---|---|---|
| $b_t$ | width of the toe structure | | m |
| $D_{n50}$ | median nominal diameter of material | $(d_{n50}=(M_{50}/\rho_s)^{0.33})$ | m |
| $D_{50}$ | median sieve diameter of material | | m |
| g | acceleration due to gravity | | $m/s^2$ |
| $h_m$ | water depth near structure | | m |
| $h_t$ | water depth above the toe structure | | m |
| H | wave height | | m |
| $H_S$ | significant wave height | | m |
| k | wave number | $(k = 2\pi/L)$ | 1/m |
| L | wave length | | m |
| $L_0$ | deep-water wave length | $(L_0 = gT_P^2/2\pi)$ | m |
| M | mass | | kg |
| s | wave steepness | $(s = H/L_0)$ | - |
| $T_P$ | peak wave period of spectrum | | s |
| $\hat{u}$ | amplitude of orbital velocity | | m/s |
| $z_t$ | height of toe | | m |
| $\Delta$ | relative mass density of material | $(\Delta = (\rho_s-\rho_w)/\rho_w)$ | - |
| $\rho_s$ | mass density of material | | $kg/m^3$ |
| $\rho_w$ | mass density of water | | $kg/m^3$ |
| $\omega$ | angular frequency | $(\omega = 2\pi/T)$ | 1/s |

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