CHAPTER 327

PIPELINE PROTECTION IN THE SURF ZONE

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ABSTRACT

Stability relations for rock on a mild slope in breaking waves were investigated, both experimentally and theoretically. Assumptions were made for the influence of turbulence in breaking waves on the load exerted by the wave motion. It appears that with these asssumptions, the mechanism is reasonably described in a qualitative way. For design purposes the method is not accurate enough. This is possibly due to the neglection of the (vertical) velocity field near the bottom in a breaking wave, giving an underestimation of the difference in stability in spilling or plunging breakers. The experimental results seem consistent and can be used provisionally for design purposes. An interesting point is that they also are in line with existing relations for stability on steep slopes.

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1. INTRODUCTION

Pipelines on the sea bottom are usually protected in order to prevent damage by anchors or erosion. Where a pipeline crosses a beach, it often lays in a dredged trench, see Figure 1, and is covered with stones. For the design of such a protection, which can be seen as an armour layer on a mild slope, a provisional design rule was established, see Schiereck et al.,1994, based on theoretical considerations and experiments. For non-breaking waves on a mild slope, the

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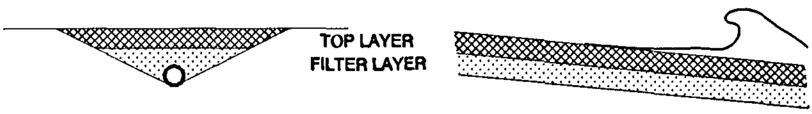

Figure 1 Outfall protection

result was reasonably satisfying. For the stability in breaking waves no theoretical concept was available and the number of experiments was not sufficient to give reliable results. In addition, new experiments were carried out, in which the slope, the stone density and the stone size were varied. Also, an attempt was made to derive a theoretical relationship for the stability of stones in breaking waves. The purpose of this attempt was twofold. First, experimental results that can be explained from theory are better understood, decreasing the danger of misusing empirical relations, while, vice versa, theories that can be verified by experiments get more practical value in hydraulic engineering. The second reason comes from didactics. Hydraulic engineering is still heavily based on empirical relations. Presenting all these relations without a link to the theory on fluid motion is considered a weak point in academical engineering education.

2. APPROACH

Experiments are indispensable to establish design rules in hydraulic engineering, so, laboratory tests are the basis of the research in this paper. But, as already stated in the introduction, the experimental results should be connected with the physical background of forces due to moving water. The creation of a link between the fluid motion and experimental results is tried with a simple, but complete description of the phenomena involved. The stability of stones on a slope is governed by the relation between load and strength. The strength is usually satisfactorily described with the effective weight and the friction of the stones. The load is much more complex. The moving water in a breaking wave will exert forces on a stone. Even in a breaking wave, the orbital velocities will play a role in the velocity field. Also, the breaking will cause turbulent eddies, with their own velocity field. The whole of orbital and turbulent velocity field is responsible for the loading on a stone. Another complicating factor is that waves in nature are irregular. Therefore, some statistal description of the waves is necessary.

In the computations the load will be composed of forces caused by the orbital motion, in combination with turbulent velocities due to breaking. For the stability, existing relations between the load and strength of a stone layer in an oscillating water motion will be used.

3. BASIC EQUATIONS

Orbital motion

The oscillatory flow near the bottom is approached with the linear wave theory:

$$\hat{u}_b = \omega \, a_b = \frac{\omega H}{2} \frac{1}{\sinh kh} \tag{1}$$

Turbulent velocities

For the turbulent velocities, an approach as given by Battjes,1975 and Battjes,1987 is used. Battjes coupled the rate of production of turbulence energy to the rate of dissipation of wave energy due to breaking:

$$q \approx (D_B / \rho_w)^{1/3} \tag{2}$$

in which q is the turbulent velocity scale ($q^2 = u_i u_j$). Figure 2 shows a comparison between measured and computed turbulent velocity scale. In this paper the expression for q, equation (2), will be used as a measure for the turbulent velocity.

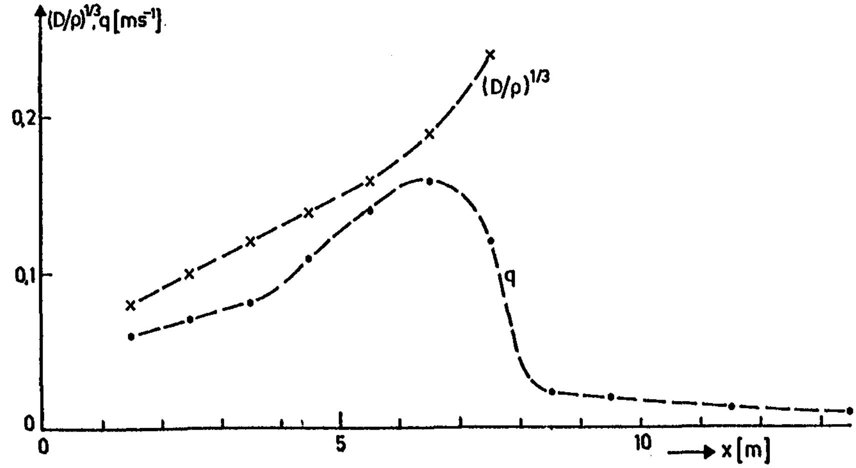

Figure 2 Comparison of computed and measured turbulent velocity scale

The dissipation of wave energy is derived from the analogy between a bore and a breaking wave, see Battjes and Jansen,1978:

$$D_B = \frac{1}{4} \frac{Q_B}{T_P} \rho_w g H_M^2 \tag{3}$$

in which Q_B is the fraction of the (irregular) waves that are broken, derived from:

$$\frac{1-Q_B}{\ln Q_B} = -\left(\frac{H_{RMS}}{H_M}\right)^2 \tag{4}$$

and where H_M is the maximum wave height, given by:

$$H_M = 0.88 * k^{-1} * \tanh(k\,h) \tag{5}$$

For more details, see Battjes & Janssen,1978. Several models are available in which this concept is implemented, e.g. the 2-dimensional DUT model HISWA, see Holthuijsen et al, 1989. In this study the 1-dimensional model ENDEC (Delft Hydraulics, see Battjes & Stive,1985) was used to compute the various wave parameters along the different slopes, since in this model the wave set-up is explicitely computed, which possibly could be important.

Wave height distribution

As a basis for the wave height distribution the Rayleigh distribution is taken. This distribution is also used in Battjes and Janssen,1978. In shallow water, the wave height distribution deviates from the Rayleigh distribution. $H_{1\%}$ in shallow water, which plays an important role in the stability calculations, is given by, according to Stive, see CUR/CIRIA, 1991:

$$H_{1\%-shallow} = \frac{H_{1\%-Rayleigh}}{\left(1+H_S/h\right)^{1/3}} \approx \frac{1.5 * H_S}{\left(1+H_S/h\right)^{1/3}} \tag{6}$$

Load - strength relations

A simple relation to express the stability of stones in oscillating flow is based on experiments in an oscillating water tunnel by Rance and Warren,1968 (see also Schiereck et al.,1994):

$$\frac{a_b}{T^2 \Delta g} = 0.025\,[\frac{a_b}{d_{50}}]^{-\frac{2}{3}} \tag{7}$$

Another approach is given by Sleath,1978. Analogous to the Shields approach in uniform flow, Sleath gives a relation between the shear stress (which is not necessarily the dominant load) and the stone weight, partly based on the experimental data by Rance & Warren (1968). The relation for stones reads:

$$\hat{\tau}_b = 0.056\,.\,(\rho_s - \rho_w).g.d \tag{8}$$

d is the equivalent spherical diameter, in this paper approximated by d_{50}, the median sieve diameter, which is easily available and differs only a few percent from the spherical diameter. The shear stress due to orbital velocities can be expressed by:

$$\hat{\tau}_b = \frac{1}{2} \cdot \rho_w \cdot f_w \cdot \hat{u}_b^2 \tag{9}$$

with f_w and $\hat{u}_b$ depending on the wave height, H, and the period, T. The circumflex over a parameter denotes "amplitude of". Given a certain wave height, the longer the period is, the larger the orbital velocity at the bottom, $\hat{u}_b$. For f_w, the friction coefficient, the opposite holds: the shorter the period, the larger the friction coefficient. In CUR/CIRIA (1991) an expression by Swart is given, based on Jonsson (1966), where f_w is given as a function of the orbital stroke at the bottom, related to the bottom roughness:

$$f_w = \exp[\,-6 + 5.2\,(\frac{a_b}{k_s})^{-0.194}\,] \qquad (f_{w\,max} = 0.3) \tag{10}$$

Computations
The combination of orbital velocities, turbulent velocities, wave height distribution and load-strength relations into a design procedure can be done in various ways. In section 5, this is further elaborated.

4. EXPERIMENTS

Experiments were done in a wave tank (length 40 m, width 0.8 m, depth 0.9 m) at the Laboratory of Fluid Mechanics, Delft University of Technology (DUT), see Ye,1996. The slope angles were 1:10 and 1:25. The mass densities of the stones were 2550 and 2850 kg/m^3 while the nominal diameters, dn_{50}, varied between 1 and 1.5 cm. The width of the sieve curves of the stones (d_{85}/d_{15}) used in the experiments was about 1.5. 3 to 4 layers of stone were used, in order to ascertain a proper roughness between stones and slope. The difference with the geometry of a real pipeline cover, which has a filter layer under the top layer, is assumed to be negligible with respect to the stability of the top layer.

The stones were laid in coloured strips of 0.25 m (measured in the wave direction) over the full width of the flume. The total number of stones displaced after every test, n, divided by the number of stones in the width of the flume, was used as the total damage S:

$$S = n\, d_{n50} / w \tag{11}$$

in which w is the width of the flume. An arbitrary damage level of 2 was chosen for incipient motion.

Irregular waves were generated according to a JONSWAP-spectrum; the number of waves tested per spectrum was 2000. The wave heights and spectra were determined at the toe of the slope. The water depth at that location was 0.6 m.

Figure 3 shows the results of the experiments. The stability is expressed as $H_S/\Delta d$, in which H_S is the significant wave height at the toe of the slope. The stability is plotted against the breaker parameter, ξ, the slope angle relative to the wave steepness.

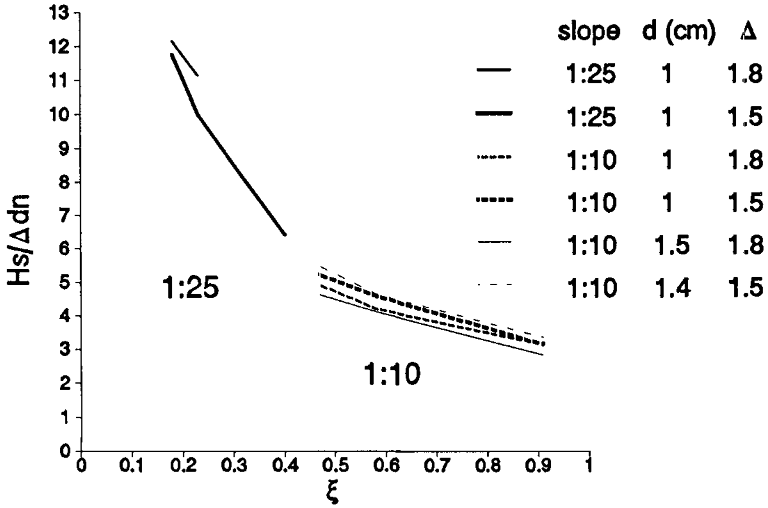

Figure 3 Experimental results

The general tendency is an increase in stability with decreasing ξ. This can partly be explained from the slope angle: the left-hand side of the figure represents the results for the slope 1:25 and the right-hand side those for 1:10. Within the results for each slope angle, there is also clearly an influence of the wave steepness: the larger the wave steepness, the larger the stability. This tendency has to do with the breaker characteristics and was previously found in other experiments, see e.g. Schiereck et al., 1994 and van der Meer,1988. Within this general tendency, there is some influence of stone density and stone dimension. Whether this is a matter of accuracy or comes from physical reasons, remains to be seen.

5. COMPARISON OF COMPUTATIONS AND EXPERIMENTS

In order to compare the computational results, obtained with the approach described in section 2, with the experimental results, at first the wave parameters along the slope were computed with ENDEC, using the measured wave characteristics at the toe of the slope at the threshold of motion. From previous investigations (see Schiereck et al.,1994) it appeared that in irregular waves, the higher waves are responsible for the incipient motion, in particular the wave height that is exceeded by 1% of the waves. This wave height is computed with equation (6) for various locations along the slope. The orbital velocities at the bottom in these waves were computed with equation (1).

Rance & Warren

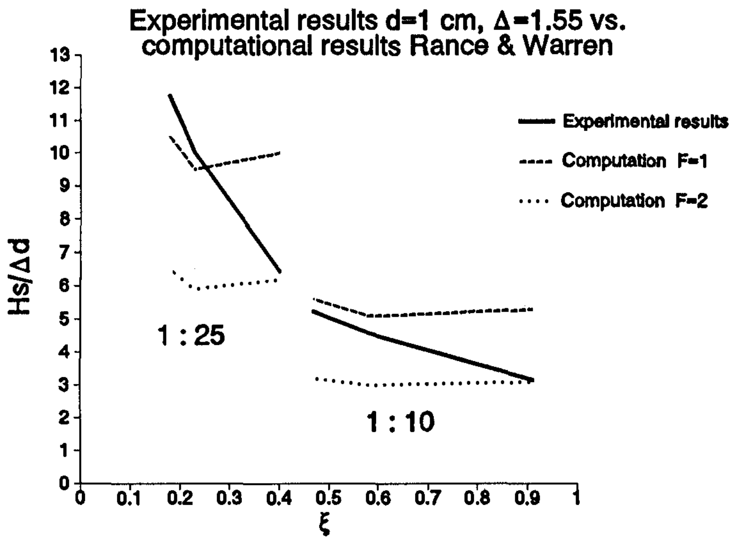

Figure 4 Comparison of experiments with computation according to Rance & Warren

The necessary stone diameter is first computed with the relation of Rance & Warren. With $\hat{u}_b = \omega * a_b$, equation (7) is rewritten as:

$$d_{50} = \frac{2.56 * \hat{u}_b^{2.5}}{\sqrt{T_p} * (\Delta\, g)^{1.5}} \tag{12}$$

This equation is valid for a horizontal bottom. The diameter in this formula is the median sieve diameter, d_{50}. In order to compare the computational results with the experiments, the nominal diameter (dn_{50}) is required, which is approximately $0.84*d_{50}$. The turbulent velocity, from equation (2), is simply added to the orbital velocity from equation (1). Together with a correction for the influence of the slope angle on the stability, the equation finally becomes:

$$dn_{50} = \frac{0.84 * 2.56 * (\hat{u}_b + F * q)^{2.5}}{\sqrt{T_p} * (\Delta\, g)^{1.5}} * \frac{\sin\phi}{\sin(\phi - \alpha)} \tag{13}$$

in which F is a calibration factor and ϕ is the angle of repose of the stones, here taken as 45°. q, as defined in equation (2), is taken as the turbulent velocity.

With this equation the necessary diameter along the slope is computed. The maximum computed diameter is used, which is equivalent to the use of a total-damage concept in the experiments, regardless of the location of the damage. For comparison, the results of the experiments with ρ_S = 2550 kg/m³ and dn_{50} = 1 cm are being used. Figure 4 shows the results for F = 1 and F = 2. The influence of the slope angle, as seen in the experiments, is also found in the computations, but the influence of the wave steepness is not reproduced correctly.

Jonsson/Sleath

The relationship as found from the results of Rance & Warren (equation (7), expresses the relation between the stroke of the orbital motion and the necessary diameter to prevent the incipient movement of the stones. This implicitely indicates the influence of the inertia of the orbital motion on the stone stability. The simple addition of a turbulent velocity to the orbital velocity, as done in equation (13), attributes the same influence to the turbulent velocity, which is not logic. Another approach is the following. Consider the forces on a grain in a flow field, see Figure 5. The shear force is exerted by the orbital movement, described with equations (9), (1) and (10). Equation (8) gives the relation between shear stress and stone size for incipient motion. This equation is now rewritten for the equilibrium of forces, where the turbulent velocity is assumed to generate a lift force, reducing the effective weight of the stone. In this way, the influence of turbulence is treated separately from the orbital motion with its specific relation for the friction factor, equation (10).

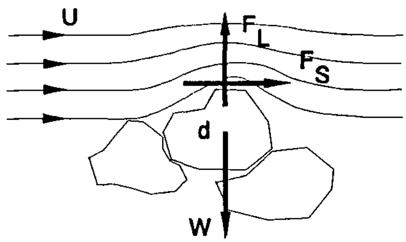

Figure 5 Flow forces on a grain

The equilibrium of forces then leads to the following proportionality:

$$\frac{1}{2}\rho_w f_w u_b^2 A_h \propto (\rho_s - \rho_w) g V - \frac{1}{2}\rho_w C_L A_h q^2 \tag{14}$$

in which A_h is a representative horizontal area, both for shear and lift. V is the volume of a stone and C_L the lift coefficient. Using equation (8), this leads to:

$$\frac{1}{2}\rho_w f_w u_b^2 = 0.056\,(\rho_s - \rho_w) g d - \frac{1}{2}\rho_w C_L q^2 \tag{15}$$

giving finally:

$$dn_{50} = \frac{0.84\,\frac{1}{2}\,(f_w \hat{u}_b^2 + F q^2)}{0.056\,\Delta\, g} * \frac{\sin\phi}{\sin(\phi - \alpha)} \tag{16}$$

in which C_L is replaced by F, a calibration factor in which both the lift coefficient and the transfer from wave energy dissipation into turbulence is included.

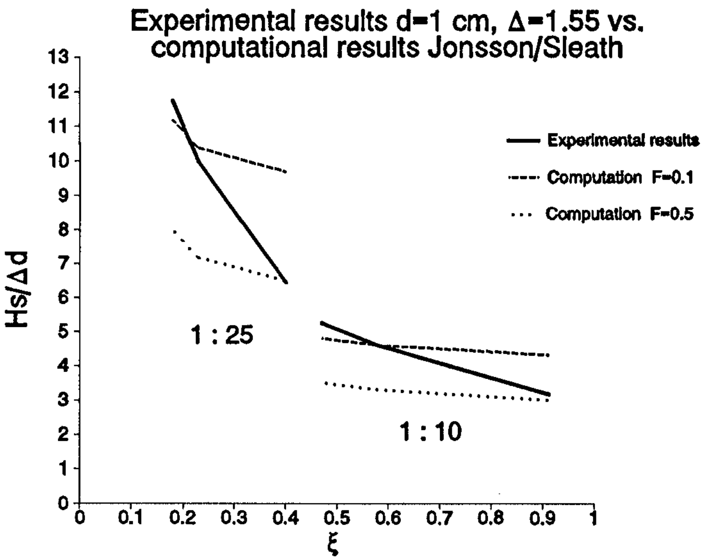

Figure 6 Experimental results compared with computations according to Jonsson/Sleath

Figure 6 shows the results for F = 0.1 and F = 0.5. The agreement is somewhat better than with the relationship of Rance & Warren. The influence of the slope angle is represented just as good as for the Rance & Warren relation, while the tendency of the influence of the wave steepness is qualitatively correct. The difference in stability between low and high values of ξ, however, for one value of the slope angle, is too small in the computations.

Variable friction factor in ENDEC

Until now, in ENDEC only one friction factor has been used, viz. 0.05, which is a relatively high value due to the rough bottom. Equation (10) leads to higher friction factors for steeper waves. Using different friction factors for different wave steepness, (f_w = 0.05, 0.04, 0.03 for s = 0.05, 0.03, 0.01, respectively) is justified, because of equation (10). For the calibration factor F in the computations 0.25 is used, being a value in between the two values of Figure 6.

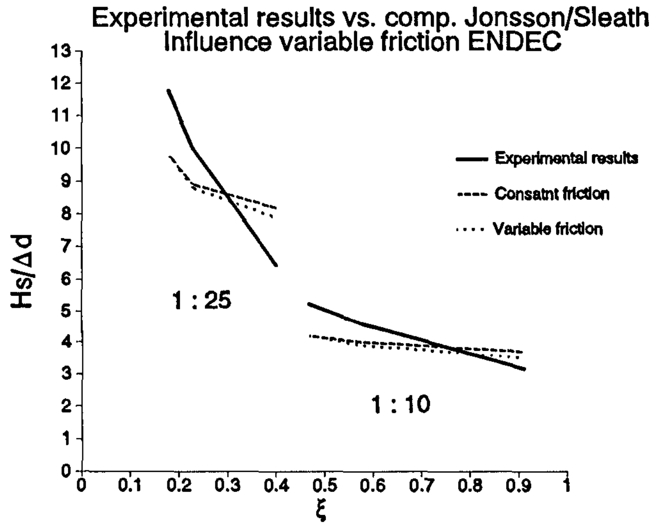

Figure 7 Variable friction factor in ENDEC

The results are now a little bit better, see Figure 7. The models being used are very simple. Considering the fact that the values of the calibration factor F are realistic with respect to the lift coefficient, the description is quite reasonable in a qualitative way. From the point of view of design practice, however, the results are still far from perfect.

The same underestimation of the favourable effect of high wave steepness was found in non-breaking waves, see Schiereck et al., 1994. The explanation has probably to be found in the negelection of the vertical flow pattern in breaking waves. Figure 8 shows a plunging breaker in which a jet obliquely to the bottom occurs, while the jet is absent in a spilling wave. This jet may have an unfavourable influence on the stone stability, which is not described in the models used here. This difference will only appear when using a 2-dimensional wave model for the wave motion on a slope. The agreement so far is encouraging enough to try to couple the experimental results to the water motion, using a better wave-velocity model.

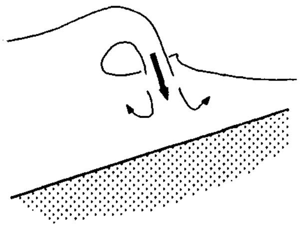

Figure 8 Plunging jet

6. EVALUATION

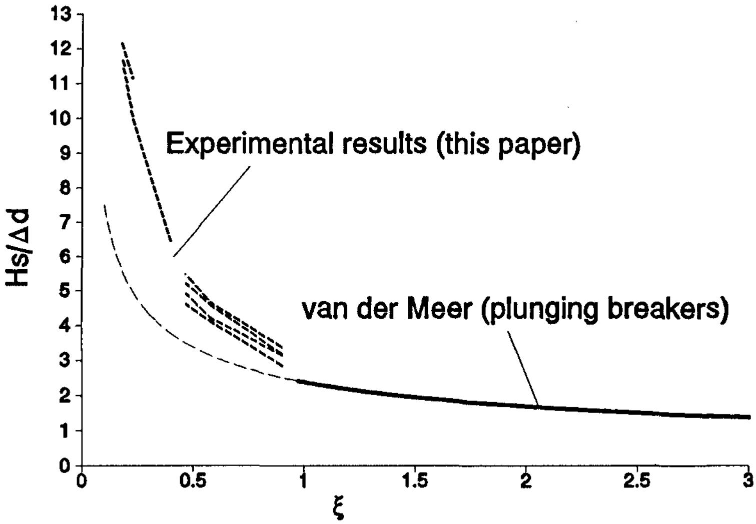

Figure 9 Experimental results compared with van der Meer formula for plunging breakers

Figure 9 shows the experimental results discussed before. The investigations were done in the range $\xi = 0.1 - 1.0$. The figure also shows the relation for stone stability on relatively steep slopes as proposed by van der Meer, 1988. This relation was validated for values $\xi > 1$. The trend for both situations is the same, which encourages further research in this field. Together with a 2-dimensional model, describing the wave motion on a slope in more detail than the models used in this paper, it should be possible in the future to give a satisfactorily accurate description of stone stability in (breaking) waves on slopes. For the time being, the experimental results as presented in this paper can be used as a first approximation for stone stability on mild slopes.

7. CONCLUSIONS

1- The computational results give a description that follow the trends of experimental results reasonably well in a qualitative way, when including the following elements from the physical process of the stability of stones in breaking waves on a mild slope:
- orbital movement (from linear wave theory, equation 1)
- wave shear stress (according to Jonssson, equation 10 and using equation 9)
- Rayleigh distribution (with shallow-water correction by Stive, equation 6)
- wave breaking and energy dissipation (according to Battjes/Janssen, equations 3, 4 and 5)
- turbulent velocities (according to Battjes, equation 2)
- stone stability (according to Sleath, equation 8)

2- The computed relation between stone stability ($H_S/\Delta d$) and breaker parameter ($\xi = \tan\alpha/\surd(H_S/L_0)$, is quantitatively insufficient. Probably the fact that the vertical velocities near the bottom in a plunging breaker were not taken into account, is the main reason for this. Other weak points are possibly the turbulence model used and the influence of turbulence on the stability of a single stone.

3- The experimental results seem consistent and are also in line with the (empirical) van der Meer relation for stability of stones on steep slopes. This ressemblance can be used as a basis for future research on stone stability in breaking waves on slopes in general.

4- The experimental results in this paper can be used for the design of a protective rock layer on a mild slope.

SYMBOLS

a_b	orbital stroke at bottom		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
D_B	energy dissipation due to breaking of waves		Nm/s/m^2
f_w	friction coefficient in waves		-
F	tuning factor		-
g	acceleration due to gravity		m/s^2
H	wave height		m
H_M	maximum wave height		m
H_S	significant wave height		m
h	water depth		m
k	wave number	$(k = 2\pi/L)$	1/m
k_s	equivalent sand roughness	$(k_s = d_{50})$	m
L_0	deep-water wave length	$(L_0 = gT_P^2/2\pi)$	m
M	mass		kg
n	number of displaced stones		-
q	turbulent velocity scale in breaking waves		m/s
Q_B	percentage of broken waves		-
S	damage		%
s	wave steepness	$(s = H/L_0)$	-
T_P	peak wave period of spectrum		s
$\hat{u}_b$	amplitude of orbital velocity at bottom		m/s
w	width of flume		m
α	slope angle of structure		-
Δ	relative mass density of material	$(\Delta = (\rho_s-\rho_w)/\rho_w)$	-
ϕ	angle of repose of stones		-
ρ_s	mass density of material		kg/m^3
ρ_w	mass density of water		kg/m^3
ξ	breaker parameter	$(\xi = \tan\alpha/\sqrt{(H/L_0)}$	-
$\hat{\tau}_b$	amplitude of bottom shear stress		N/m^2
ω	angular frequency	$(\omega = 2\pi/T)$	1/s

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