CHAPTER 112

STABILITY OF ROCK ON BEACHES

by

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ABSTRACT

Stability relations for rock on a mild slope are derived and compared with experimental results. It appears that for non-breaking waves the stability on a mild slope can be described with existing relations for stability on a horizontal bottom in oscillatory flow. For breaking waves, no existing relation can be used and a provisional empirical design rule was established. The results can be applied in designing outfall protections.

1. INTRODUCTION

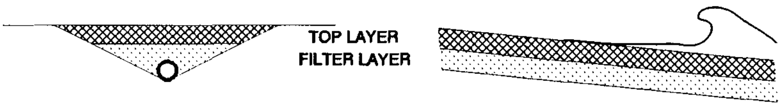

Figure 1 Outfall protection

Pipelines on the sea bottom are usually protected in order to prevent damage by anchors or erosion. When a pipeline crosses a beach, it often lays in a dredged trench, see Figure 1, is covered with stones including a filter layer and is again covered with the original sand. The protection then acts as a last defence in case of severe wave attack on the beach. For the design of such a protection, which can be seen as an armour layer on a mild slope, no design rule is available at the moment.

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Much information is available on static stability of rock on steep slopes, as occurring at breakwaters and revetments, see e.g. Hudson (1959) or van der Meer (1988). Information is also available on static stability in oscillating flow on a horizontal bottom, see e.g. Sleath (1978) or Rance & Warren (1968), while Naheer (1977) investigated the stability under breaking solitary waves. For mild slopes much is known about dynamic stability, e.g. for sand and pebble beaches, where the profile is reshaped when conditions change. About static stability on mild slopes, which is a rather unusual combination in coastal engineering, practically nothing is known. At Delft University of Technology two studies were done (Sistermans (1993) and Grote (1994)) to find a static stability relation for rock on mild slopes which can be used as a design rule for outfall protections.

2. LOAD AND STRENGTH

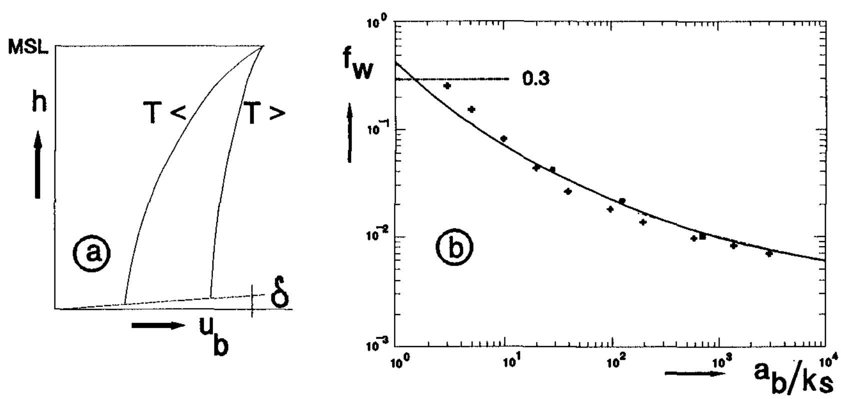

Figure 2 Orbital velocity and wave friction

The basic phenomenon in stability under waves on a mild slope, at least under non-breaking waves, is assumed to be the shear stress due to oscillatory flow:

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

with:

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

f_w and u_b depend on the wave height, H and period, T. Given a certain wave height, the longer the period, the larger the orbital velocity at the bottom, u_b, (Figure 2a).

With f_w, the friction coefficient, the relation is opposite: the shorter the period, the larger the friction coefficient, see Figure 2b, derived from Jonsson (1966) where f_w is given as a function of the orbital stroke at the bottom, related to the bottom roughness. This is caused by the fact that the boundary layer, (δ, see Figure 2a) under a longer wave is more developed, leading to a lower velocity gradient, causing a smaller shear stress (given a certain velocity). In CUR/CIRIA (1991) an expression by Swart is given to describe the relation of Figure 2b:

$$f_w = \exp[\,-6 + 5.2\,(\frac{a_b}{k_s})^{-0.194}\,] \qquad (f_{w\,max} = 0.3) \tag{3}$$

For the stability relation between shear stress and stone dimensions, the critical Shields parameter, as adapted by Sleath (1978) for turbulent flow is:

$$\frac{\hat{\tau}_b}{(\rho_s - \rho_w)\cdot g \cdot d} = 0.056 \tag{4}$$

d is the equivalent spherical diameter, in this paper approximated with d_{50}, the median sieve diameter, which is easily available and differs only a few percent from the spherical diameter. Equation (4) is partly based on experimental data by Rance & Warren (1968) from experiments in an oscillating flow tunnel. In this research, their results are, as an alternative for equation, (1), (3) and (4), described with:

$$\frac{a_b}{T^2 \Delta g} = 0.025\,[\frac{a_b}{d}]^{-\frac{2}{3}} \tag{5}$$

In breaking waves, for the time being, the same mechanism is assumed to work. But due to a complete change in the velocity field and the turbulence in a breaking or broken wave, it can be expected that some amplification factor on the computational results has to be applied to fit experimental data.

3. EXPERIMENTS

Set up

Experiments were done in a wave tank (length 40 m, width 0.8 m, depth 0.9 m) at the Laboratory of Fluid Mechanics at Delft University of Technology (DUT). On the concrete surface of a slope 1:25, stones were laid with dimensions ranging from d_{50} = 7 mm to 17 mm and densities ranging from 2400 to 2900 kg/m^3. 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 the stones and the 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 stability of the top layer.

The stones were laid in coloured strips of 0.2 m (in the wave direction) over the full width of the flume. By counting the number of stones displaced, n, after every test, the damage S was determined as a percentage of the total number of stones available in a strip: $S = n * d_{n50}^2 / A$ (%). For every strip a relation between n and the wave height at the toe of the slope was established and from these curves, the level S = 0.5 % was arbitrarily chosen as incipient motion.

The maximum number of waves in regular wave tests was 750. Irregular waves were generated according to a JONSWAP-spectrum; the number of waves was 2000 in irregular wave tests. The wave heights and spectra were determined at the toe of the slope. The water depth at that location varied from 0.7 m for regular waves to 0.6 m for irregular waves.

Scale effects

The flume in the laboratory is considered as a physical reality in itself, not as a model of some prototype with a scale 1:X. Computations are made for that reality and compared with the experimental results. However, in the end, the relations thus found have to be used for prototype circumstances. This is permissible only if no scale effects are present in the relations. Sources of scale effects in small flumes using water as a fluid, can be the viscosity of the water, the surface stress and air entrainment in breaking waves.
To avoid viscosity effects, the particle **Re**-number in the Shields-graph ($u_{*c}d/\nu$), should be more than about 600, indicating that the flow around the stones is turbulent. The minimum stone size of 7 mm was chosen to meet this demand.
Stive (1985) investigated the scale effects in breaking waves on a 1:40 slope and found no significant scale effects in wave heights and velocities on the slope for a wave height range from 0.1 to 1.5 m. The wave heights in the authors' experiments ranged from 0.1 m to 0.35 m.

4. COMPUTATIONS

Orbital velocities were computed with the linear wave theory. Although the circumstances on the slope are beyond the validity-range of this theory, LeMéhauté (1968) already showed that for orbital velocities at the bottom, the theory predicts measured values quite well. Figure 3 shows for various locations along the slope and wave steepnesses, s, in the authors' experiments, the comparison between

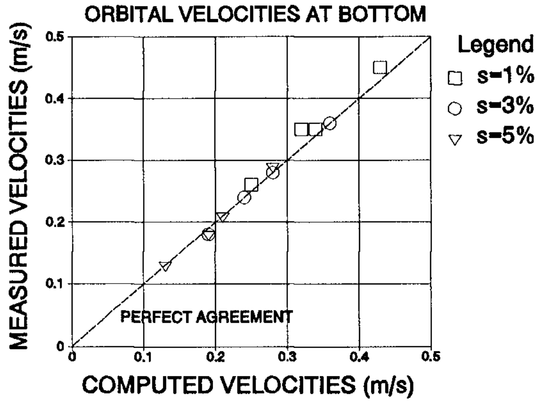

Figure 3 Comparison computed and measured orbital velocities in DUT wave flume

computed and measured maximum orbital velocities at the bottom. The measured values are in the upslope direction, since it appeared from the experiments that the first movement of the stones is in upslope direction. The agreement between computed and measured values is remarkably good.

The wave heights on the slope were determined, applying the shoaling coefficient:

$$K_{sh} = \frac{1}{\sqrt{\tanh kh \left[1 + \frac{2kh}{\sinh 2kh}\right]}} \tag{6}$$

The water depth at the toe of the slope in the flume cannot be considered as deep water. Hence the local wave height is computed as:

$$H_L = K_{sh,L} * H_0 = K_{sh,L} * \frac{H_{Toe}}{K_{sh,Toe}} \tag{7}$$

For irregular waves the same procedure is followed, now with respect to a significant wave height, H_S, and a peak period of the wave spectrum, T_P.

Tests were done with various stone sizes. For each water depth along the slope, the (local) wave height for the threshold of motion for the stone dimensions in a certain test, and the corresponding wave height at the toe of the slope were determined with the above mentioned relations and compared with the measurements.

5. NON-BREAKING REGULAR WAVES

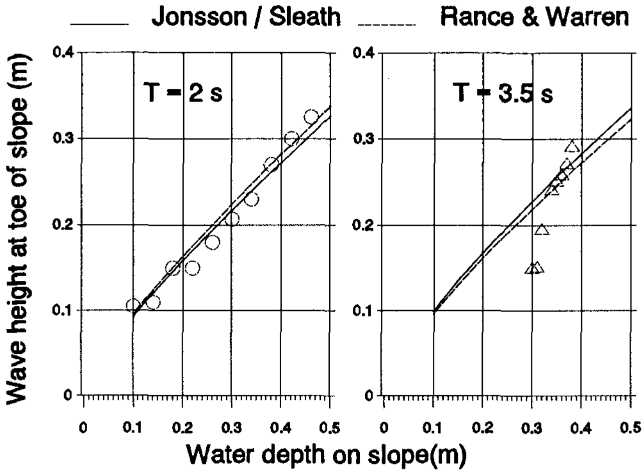

Figure 4 Wave height at toe of slope for incipient motion with constant T

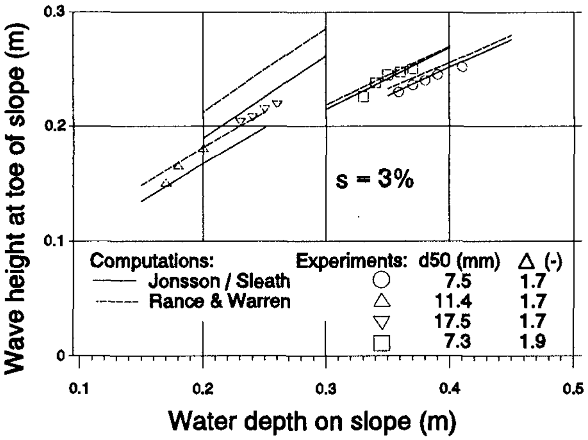

Figure 5 Wave height at toe of slope for incipient motion with constant s = 3%

Figure 4 shows the results for constant wave period (2 s and 3.5 s). The differences between the computation according to Jonsson/Sleath and Rance & Warren are small. The agreement for T = 2 s is quite good. For T = 3.5 s there is a deviation for water depths around 0.3 m. The wave steepness for which the threshold of motion was determined was 1 % or less, and the corresponding wave appeared to be unstable in the flume: the waves fell apart into two separate waves. Figure 5 shows the stability for a constant steepness, $s_T = H_T/1.56*T^2 = 3$ % and various stone sizes and densities. The difference between Jonsson/Sleath and Rance & Warren is again small for the smaller stones in relatively deep water. For larger stones the difference increases. This is caused by the fact that a large stone diameter gives a large friction coefficient, f_w, which is computed iteratively in Jonsson/Sleath.

In general it can be said that the assumed mechanism describes reasonably well the experimental results. In the following, all computations have been done with the method of Rance & Warren, being simpler than that of Jonsson/Sleath while the differences are small.

6. NON-BREAKING IRREGULAR WAVES

The stability mechanism in irregular waves does not differ from that in regular waves, but it is obvious that the higher waves cause the damage. In this research, the irregular wave field is described with a significant wave height and the peak period of the wave spectrum. From a first comparison between the results for regular and irregular waves, it appeared that, for incipient motion, in regular waves the wave height was about 50% higher than the significant wave height in irregular waves. Taking the Rayleigh-distribution as basis for the wave height distribution, this would mean that the 1% highest waves are responsible for the incipient motion ($H_{1\%} \approx 1.5H_S$). In shallow water, the wave height distribution is going to deviate from the Rayleigh-distribution. An expression for this deviation is given by Stive, see CUR/CIRIA (1991). This expression is, however, independent of the wave period, while in the experiments the difference between regular and irregular waves was larger for lower wave steepness.

A magnifying factor for the significant wave height, K_I, is now defined with a lower limit 1 and an upper limit depending on the water depth and the wave period:

$$\begin{aligned} H_{ML} &= K_I * H_{SL} = \left[1 + Constant * \tanh\left(\frac{H_M}{H_{SO}}\right)\right] * H_{SL} \\ &= \left[1 + Constant * \tanh\left(\frac{0.14 * L * \tanh(kh)}{H_{SO}}\right)\right] * H_{SL} \end{aligned} \tag{8}$$

H_M, is the breaker height according to Miche, see CUR/CIRIA (1991). For the constant in K_I, a value 0.6 was chosen. The result is a magnifying factor for H_{SL} along the slope between 1 and 1.6 depending on the depth and the wave steepness.

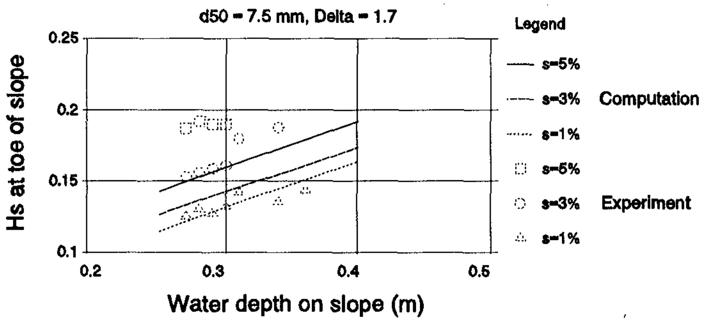

Figure 6 Regular non-breaking waves, $\Delta = 1.7$

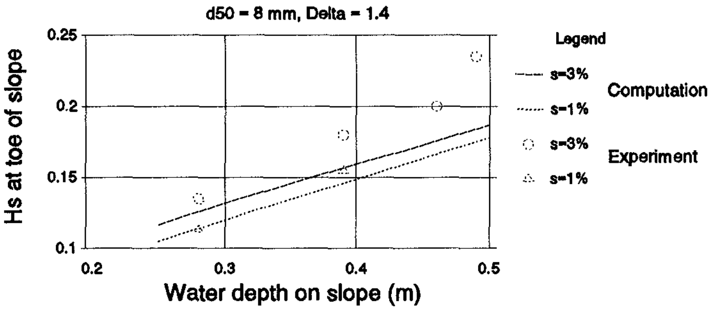

Figure 7 Regular non-breaking waves, $\Delta = 1.4$

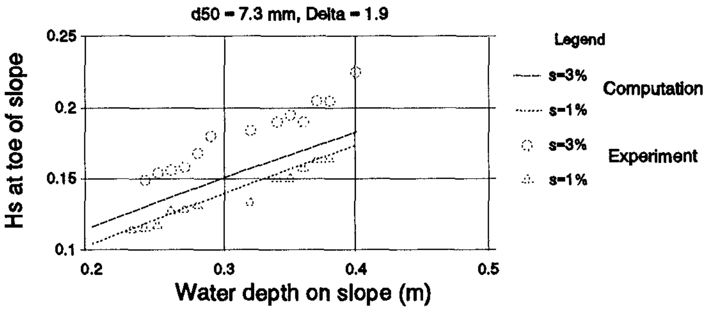

Figure 8 Regular non-breaking waves, $\Delta = 1.9$

Figure 6, Figure 7 and Figure 8 show the results for non-breaking irregular waves. It appears that for all stone densities investigated, the computations follow the trend of the experiments rather well; the influence of the wave steepness, however, is underestimated. The constant (0.6) in K_I was chosen such that it fitted the experimental results for irregular waves with s = 1%, being a practical lower limit for irregular waves. It appears that the representation of an irregular wave field with H_S and T_P, as used in shoaling computations, combined with the proposed magnifying factor, K_I, is too simple to describe correctly the whole process. For this first round, however, this is taken for granted, since the trend is indicated quite well and the final values to be used in the design rule are on the conservative side.

Note: The use of H_M, the local breakerheight according to Miche, seems somewhat peculiar in a formula for non-breaking waves. This is however deemed acceptable because, in an irregular wave field, there is a very gradual transition from non-breaking to breaking, while the mechanism at the point of breaking is not necessarily completely different. Moreover, the use of K_I, including H_M, has no other pretention than being a sensible way of curve-fitting.

7. BREAKING REGULAR WAVES

The assumed mechanism for the stability in breaking waves is again the shear stress under a wave. Starting with a given wave at the toe of the slope, the local wave parameters are computed and from these the stone diameter needed.

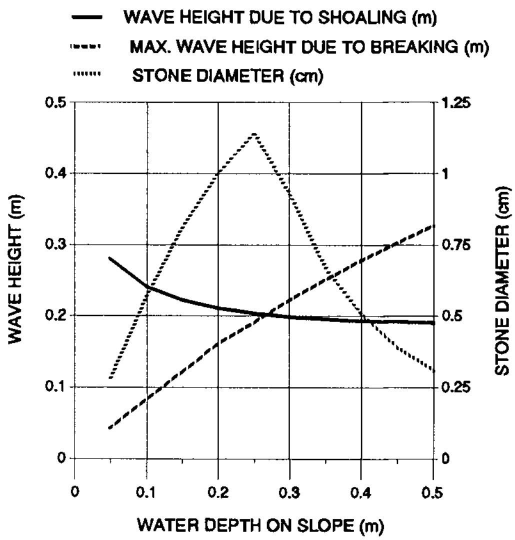

Figure 9 Computation of stone diameter in breaking wave

Figure 9 shows the computed wave heights due to shoaling and the breaker height according to Miche. The maximum of these two is used in the computation of the diameter, yielding a maximum value along the slope. It can be expected that this procedure will underestimate the necessary diameter, since the shape of the wave, the velocity field and the turbulence in a breaking or broken wave are unfavourable compared with non-breaking waves. Therefore, a second tuning parameter, K_B, comes in:

$$d_{Breaking} = K_B * d_{Non-Breaking} \tag{9}$$

The diameter that finally results from this procedure is taken as the diameter in breaking regular waves.

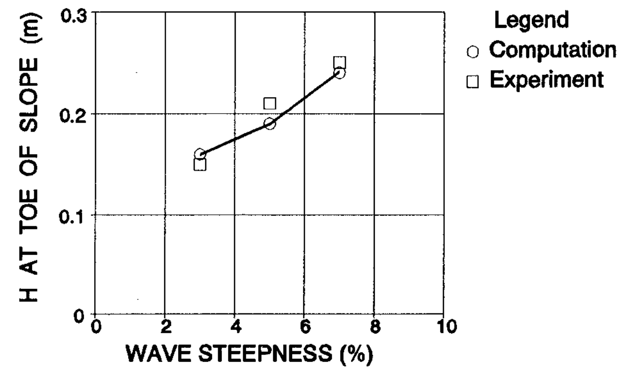

Figure 10 Comparison measured and calibrated values for regular breaking waves

Figure 10 gives the results of the comparison between computation and experiment, in which the above mentioned procedure was performed in a reversed order and iteratively (starting with a given diameter, which is the case in a flume experiment, computing the local wave height for incipient motion and hence the wave height at the toe of the slope). $K_B = 2$ is used in the figure, giving the best fit.

Here again it can be seen that a larger wave steepness leads to a more stable situation (larger wave height for incipient motion, see Figure 10). This can also be explained from the way of breaking of the waves. According to the common classification, all these waves belong to the spilling type ($\xi < 0.4$, see Battjes (1974)). There is, however, a gradual change in breaking characteristics and the lower the steepness of the waves in the experiments, the more they showed a plunging behaviour. In a plunging breaker the bottom is attacked by a jet, giving a more unfavourable situation compared to a "real" spilling breaker. The energy dissipation in a plunging breaker is more concentrated than in a spilling breaker, see Figure 11.

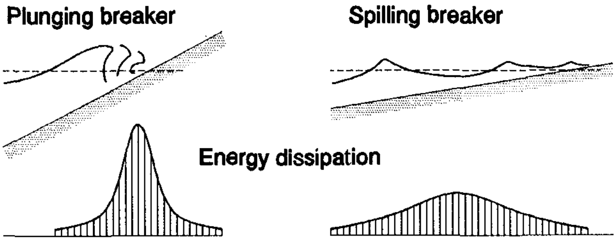

Figure 11 Energy dissipation in spilling and plunging breakers

8. BREAKING IRREGULAR WAVES

With a magnifying factor, K_I, for the irregularity of the waves and an amplification factor, K_B, to take the effect of breaking into account, the stability in breaking irregular waves is computed and the results are given in Figure 12.

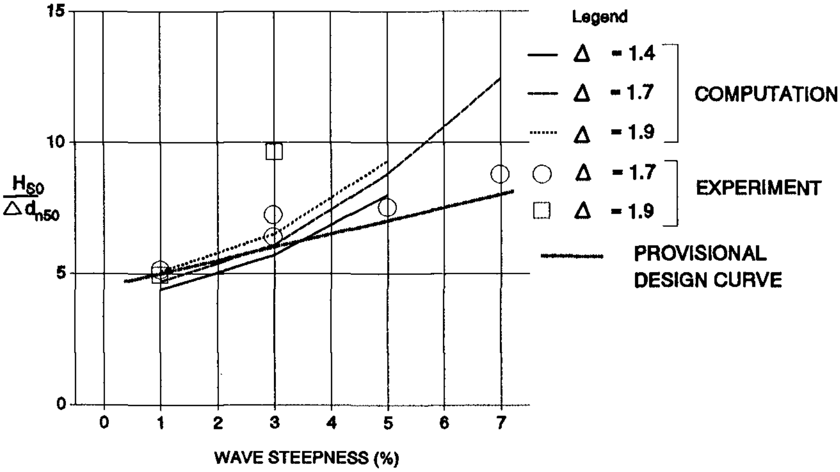

Figure 12 Comparison computed and measured values for irregular breaking waves

In this figure the stability is expressed as $H_{S0}/\Delta d_{n50}$ in order to compare it with the stability of rock on breakwaters or revetments. H_{S0} is a fictitious deep water wave

height, derived from equation (7) in order to have an unambiguously defined wave height. The nominal diameter (d_{n50}) is used instead of the sieve diameter because this is a more practical measure for large stones. The relation between d and d_n can be taken as an average for normal rock, see CUR/CIRIA (1991): $d_n = 0.84*d$.
From Figure 12 the following can be seen:

- The dimensionless parameter $H/\Delta d$, computed according to the procedure outlined above, still shows variation with Δ. This means that either the mechanism assumed in the computation is not correct or the use of this dimensionless parameter is not allowed.
- The rather scarce data do not fit very well with the computations.
- Typical values of $H/\Delta d$ for a wave steepness of 3 %, lie around 6 compared to 2 for revetments with steep slopes.

For the time being, the lower limit of the experimental results is taken as a conservative approach for a provisional design rule:

$$\frac{H_{SO}}{\Delta d_{n50}} = 4.5 + 50 * s_0 \qquad (10)$$

The tendency of this result is conformable to van der Meer's equation for stability on steep slopes, see van der Meer (1988). For a given slope angle, van der Meer's equation for plunging breakers gives an increasing stability number $H/\Delta d$ for an increasing wave steepness or a decreasing ξ. In Figure 13 both relations are given for a slope angle 1:25 (this is far beyond the range for which the van der Meer equations are valid, but it is just to show the mentioned tendency).

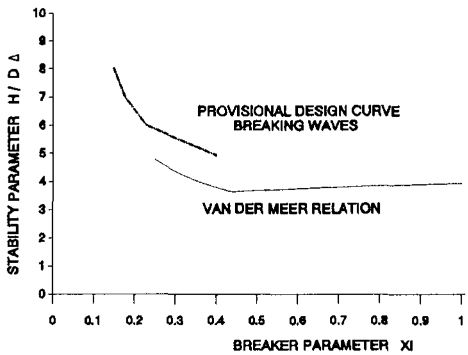

Figure 13 Comparison experimental results with van der Meer's equation

9. DESIGN RULE

The proposed design rule is now to be applied as follows:

- Start with the significant deep water wave height (H_{s0}), measured or from equation (7) and a wave height measured at any relatively deep water location
- Compute the shoaling of H_{s0} along the slope with equation (6) using the peak period of the spectrum, T_P
- Determine H_{ML} along the slope with equation (8)
- Compute the necessary diameter along the slope with equation (2) and (5)
- Compute the maximum diameter with equation (10)

Figure 14 gives the result for H_{s0} = 5 m, T_P = 10 s and Δ = 1.65.

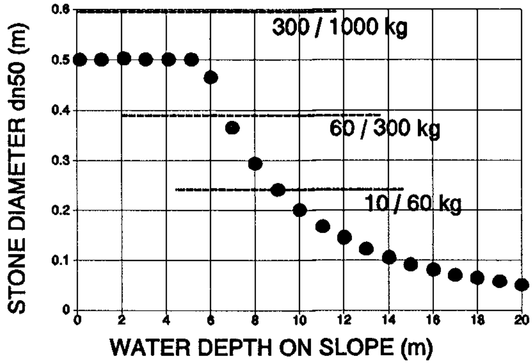

Figure 14 Design example

NOTES:

1 Equation (10) expresses the stability as a function of the wave steepness, s. Since the stability is closely related to breaking, it would be more appropriate to relate the stability parameter $H/\Delta d$ to the breaking parameter ξ. Only one slope was investigated (1:25). Using s instead of ξ is on the safe side for all slopes milder than 1:25, since a milder slope yields lower ξ-values leading to a more stable situation.

2 In the figure some limits of stone classes are given, which are common in the Netherlands. These are not essential, but are presented to show that for a practical case, a choice has to be made from material available.

10. RECOMMENDATIONS

The design rule, presented in the previous section, is the best approach available at the moment for stability of rock on mild slopes. Conservative choices had to be made; improvements are possible. Therefore the following is needed:

- A more complete description of the velocities and shear stresses on the slope in irregular waves. This can probably be obtained with a model that describes the behaviour of irregular waves on a slope, including bottom friction and breaking, see e.g. van der Meer (1990).
- A physical description of the breaker zone itself with a model that includes the shape of the waves, the velocity field and the turbulence. This could give more and better information on the attack on the slope than is possible with the orbital movement according to the linear wave theory.
- More systematic experimental data on stability in breaking waves, including other slope angles. In doing so, also an attempt can be made to make a better link between the relations in Figure 13, giving a complete picture of stability of rock on mild and steep slopes.

SYMBOLS

A	surface area of coloured strips		m^2
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
f_w	friction coefficient in waves		-
g	acceleration due to gravity		m/s^2
H_0	deep water wave height		m
H_L	local wave height		m
H_{ML}	maximum local wave height (Miche breaker height)		m
H_S	significant wave height		m
h	waterdepth		m
K_I	magnifying factor for irregular waves		-
K_B	amplification factor for breaking waves		-
K_{sh}	shoaling factor		-
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		m
M	mass		kg
n	number of displaced stones		-
S	damage		%

s	wave steepness	$(s = H/L_0)$	%
T_P	peak wave period of spectrum		s
u_b	orbital velocity at bottom		m/s
α	slope angle of structure		-
Δ	relative mass density of material	$(\Delta = (\rho_s - \rho_w)/\rho_w)$	-
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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