The effect of foreshore slope on breakwater stability

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The problem history

In case of coasts with steep foreshores coastal structures suffer more from damage than normally could be expected from given boundary conditions at deep water. For that reason in many guidelines it is recommended to apply a heavier class of rock in those cases; manufacturers of single layer units (like Accropode, Core-loc or Xbloc) recommend a lower K_d value in case of a steep foreshore. Unfortunately until recently there was no insight in the physical processes leading to this extra load. In the stability formula of VAN DER MEER [1988] shallow water and steep foreshores are not considered. The original formula (for plunging conditions) of Van der Meer reads:

$$\frac{H_s}{\Delta D_{n50}} = c_{plunging}\,\xi^{-0.5} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{1/5} \tag{1}$$

The coefficient $c_{plunging}$ has a value of 5.0

A first step into a more systematic analysis of problem of shallow water and steep foreshores has been the systematic research on the interaction between waves and structures in case of shallow foreshores by VAN GENT *ET AL*.[2003]. The main result of this research was that the load on the structure can better described by a shallow water spectrum at the toe of the structure. And because the longer periods are more relevant than the shorter periods, more weight should be given to the low frequency part of the spectrum. This can be done by using in stability formula as parameter for the wave energy not the T_p or T_{m0}, but the $T_{m-1,0}$. This parameter gives more reliable results both for run-up/overtopping formula as well as for stability formula.

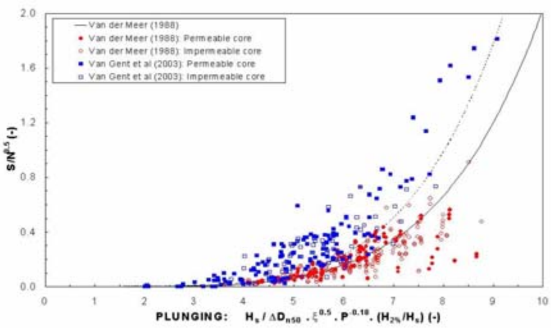

Figure1: Data of VAN GENT [2003] and VAN DER MEER [1988] (from VAN GENT[2004])

Van Gent stated that for shallow water conditions instead of the significant wave height it is better to use the 2% wave height, and to calculate the value of ξ with the spectral wave period $T_{m-1,0}$. This leads to the following equation for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = c_{plunging}\,\xi_{s,-1}^{-0.5} P^{0.18} \left(\frac{H_{2\%}}{H_s}\right)^{-1} \left(\frac{S}{\sqrt{N}}\right)^{1/5} \tag{2}$$

This formula has been calibrated on the dataset of Van Gent. Calibration of the formula leads to a value of $c_{plunging}$ of 8.4.

A comparable transformation can be carried out for surging waves.

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The present problem

As can be seen from Figure 1, even after correction of the data as indicated in equation (2), there is a significant difference between the dataset of Van der Meer and the dataset of Van Gent. Thorough investigation of both the test sets of Van der Meer and of Van Gent could not identify systematic differences in the modelling approach. The differences between the tests is that Van der Meer focused of tests with horizontal bed in front of the construction (80% of his tests) and deep water (the ratio H_s/d varied between 0.12 and 0.26), while Van Gent focused on slopes (1:100 and 1:30) and shallow water (the ratio H_s/d varied from 0.23 to 0.78m; few test were between 0.15 and 0.23). However, these effects are accounted for in the transformation from deep water spectra to shallow water spectra.

Recent tests on steep slopes

Because the stability of breakwaters at steep slopes is still a problem, it has been decided to focus some laboratory tests on this problem. A number of tests were done in the research flume of the Laboratory of Fluid Mechanics of Delft University of Technology. In the flume a 1:2 (permeable) breakwater has been constructed on a 1:8 foreshore and on a 1:30 foreshore. The toe of the breakwater was also made of riprap, but fixed with chicken wire to prevent any damage to the toe. Of course it is questionable if a 1:8 foreshore should be considered as a foreshore or as a part of the structure. However, these two slopes were selected in order to see clearly the differences in slope effects. The wave flume has full reflection compensation. Several series of tests were done, but relevant in this framework are two series.

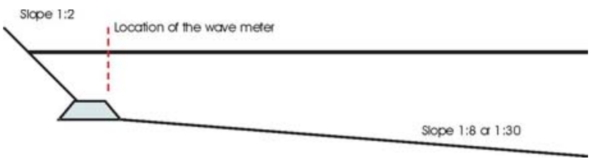

Figure 2: Test set-up in the laboratory

The basic setting of these series is a wave height at the wave maker $H_{m0,0}$ = 12.1 cm and a wave period of T_p = 1.6 seconds. The waterdepth is 66 cm, the rock had a size of D_{n50} 0f 1.57 cm and a specific density of ρ = 2789 kg/m^3 In the first series the wave at the wave board was kept identical, in the second series the wave at the toe of the structure was kept identical. In this second series, a wave spectrum was generated in such a way that the incoming spectrum at the toe of the breakwater was exactly the same for both the 1:8 and the 1:30 slope. The damage on the breakwater was investigated for a number of wave conditions. The results are presented in figure 3. As a standard procedure the wave at the toe has been decomposed in an incoming wave and a reflected wave. In the experiment described in figure 3 the (decomposed) incoming waves are made identical.

As expected, for equal deep water waves, the damage in case of a steep slope is more than in case of a gentle slope. However, also in the tests with identical waves at the toe of the breakwater, the steeper slope resulted in significantly more damage than the more gentle slope. Especially with a wave steepness in the order of 1.2 the difference in damage is significant.

Because the spectra are identical, and the damage is clearly not identical, this implies that the damage to the breakwater also has to depend on a wave parameter which is not *represented by the shallow water wave energy spectrum, and it has to be a parameter which is different for waves approaching over different foreshore slopes.*

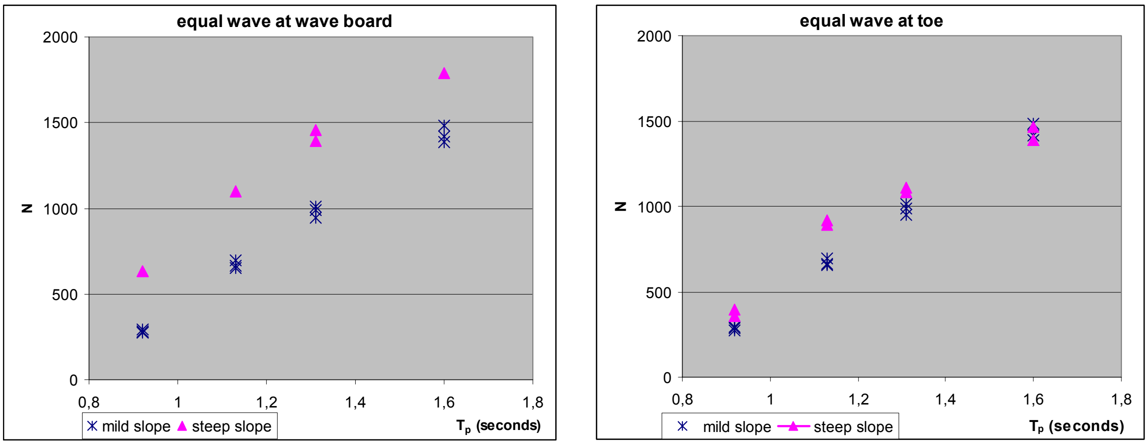

Figure 3: Results of damage tests, with identical shallow water spectra at the toe of the breakwater.

Because decomposition of the wave at the toe into a incoming wave and a reflective wave (especially for irregular waves) is rather complicated very near to a structure, a number of tests have been carried out focussing on the relation between waves at the toe for decomposed and undecomposed wave spectra. It was found that the above conclusion remains completely valid for both the decomposed as well as the undecomposed wave.

In case a physical relation is looked for which should relate the local hydraulic parameters to the stability of individual rocks, the undecomposed wave should be used. The rock is influenced by a certain hydraulic load, and it does not matter if load is caused by reflected or not-reflected waves. However, in case one is looking for a relation between the stability of rock slope in general as a function of an overall descriptor of the wave load, like a spectrum, one should use the decomposed waves.

Tests on acceleration in currents

Because earlier tests by a number of M.Sc. students at Delft University of Technology on propeller jets on slopes indicated that the acceleration might be relevant for the stability of stones, two lines of experiments were set up. In the first line focus was on permanent flow. In a flume a tapering was introduced. Because of this tapering the flow was accelerated. By keeping the decrease in width constant, and by varying the length of the tapering, the velocity in the tapering was equal for each test, but resulted in a different acceleration. For each combination of velocity and acceleration the critical condition for moving stones was determined. The tests showed that for a given stone size the critical velocity decreased with an increase of the acceleration. To describe this a stability parameter has been developed on the basis of the Shields parameter, but including the acceleration in the same style as is used in the Morison equation:

$$\Psi_{MS} = \frac{\frac{1}{2} C_B \rho d^2 u^2 + C_M \rho d^3 a}{\rho \Delta g d^3} = \frac{\frac{1}{2} C_B u^2 + C_M da}{\Delta g d} \qquad (3)$$

The entrainments function can be described as:

$$\Phi_E = Ed^2\sqrt{\frac{d}{\Delta g}} \tag{4}$$

in which E is n/AT, in which n is the number of stones displaced, A the area of the displaced stones and T the duration of the experiment.

Regression analysis shows that the relation between the critical load and the entrainment is given by:

$$\Phi_E = 6\cdot 10^{-6}\Psi_{MS}^{4.73} \qquad 0.2 \le \Psi_{MS} \le 1.4 \tag{5}$$

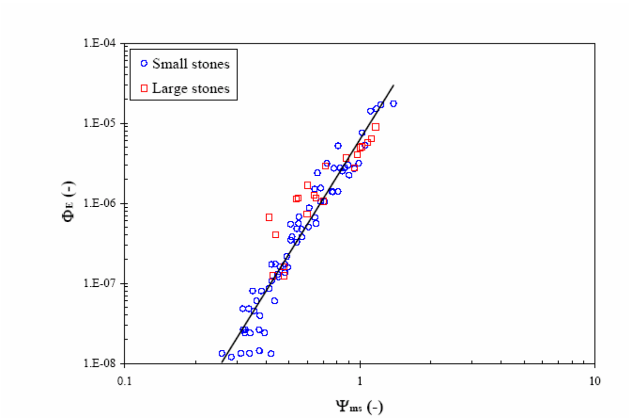

Figure 4: The dimensionless entrainment and Morison-Shields parameters for two types of stones [DESSENS, 2004]

The values of C_B and C_M were found as 0.10 and 3.92 [DESSENS, 2004]. The quotient C_M/C_B = 39.

A very general form of a stability formula is the stability formula of Izbash [1932]

$$d_{crit} = A\frac{u^2}{\Delta g} \tag{6}$$

in which u is a velocity near the stone, and A is a stability number. For permanent flow this equation can be rewritten as the Shield equation, introducing both an depth-averaged velocity as well as a logarithmic velocity profile. Equation (3) and (6) can be combined to

$$d_{crit} = A'\frac{\frac{1}{2}C_B u^2 + C_M da}{\Delta g} = A\frac{u^2 + \frac{C_M}{C_B}da}{\Delta g} \tag{7}$$

Tests on acceleration in waves

A similar test set up was made for the critical condition for movement of stones under waves. TROMP [2004] and TERRILE [2004] did tests on a 1:30 slope with regular waves. The flume was equipped with reflection compensation.

As was expected, at deep water the phase shift between velocity and acceleration is in the order of π/2. But in a shoaling wave, the moment of maximum velocity and maximum acceleration also move to each other. This decrease in phase shift is indicated in figure 5. Because of shoaling both the acceleration and the velocity increase. However, the acceleration increases more because of the increase asymmetry of the wave.

Applying equation (3) to wave situations leads for deeper water and for horizontal beds not to an lower critical velocity. Because of the phase shift between velocity and acceleration at the moment of maximum u the acceleration is zero, and consequently the component $(C_M/C_B)/a$ is also zero.

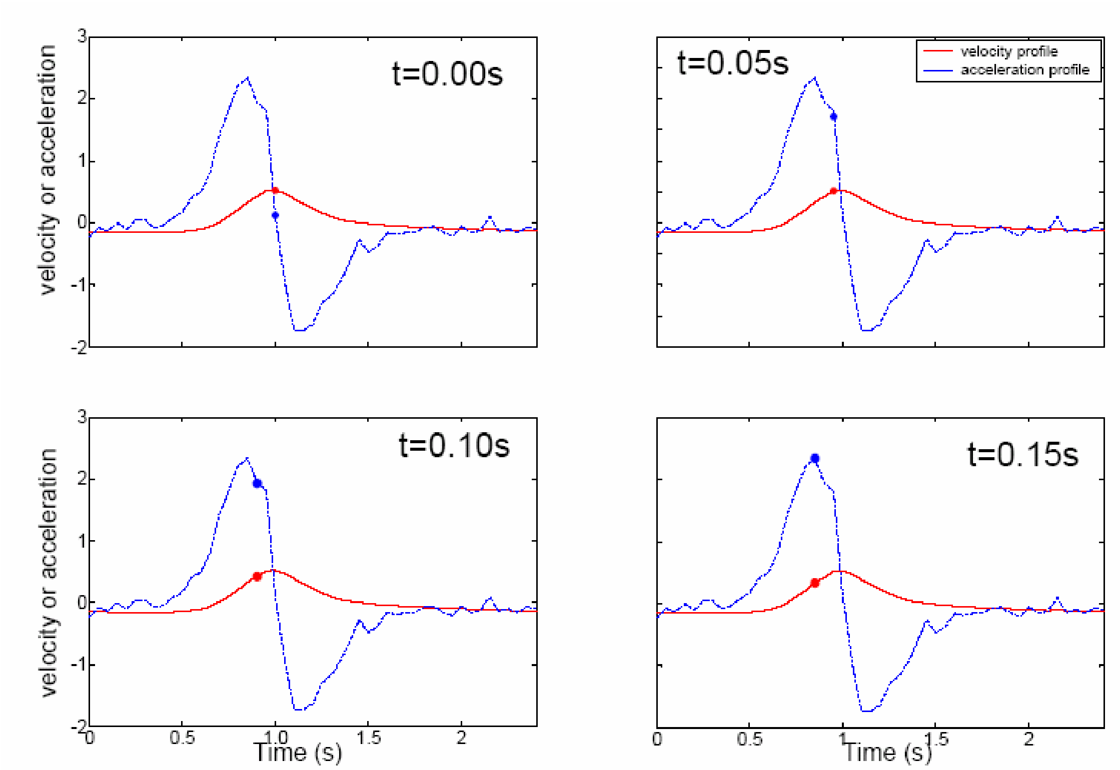

Figure 5: Instantaneous values of near-bed horizontal velocities and accelerations for a wave with H_0= 12.5 cm, h_0= 55 cm, T=2.5 s and X=18.68 m. The solid line represents the orbital velocity and the dash-dotted line the orbital acceleration. The time in the title of the plots is the time <u>before</u> passage of the wave top [TROMP, 2004]

During the tests Tromp made detailed video observations of movement of stones. The movements could be correlated to the observed near bed velocities and near bed accelerations. Figure 6 shows the movement of a stone as a function of the near bed velocities for two different waves, both with a wave height of 15 cm, but with two different periods.

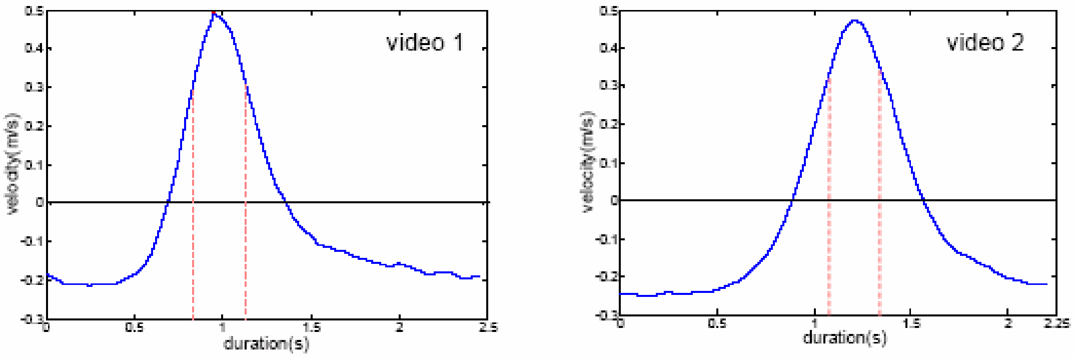

Figure 6: Velocity profile and duration of stone movement for a similar wave of 2.5 (video1) and 2.25 (video2) seconds [TROMP, 2004]

In figure 7 for the same situation the movement of the stone is plotted as a function of the acceleration. In the table below the results of this observation are summarised. It is clear that the start of motion of a stone is a function of both velocity and acceleration. This indicates that also for oscillatory motion a Morison-type of approach seems to be very logical.

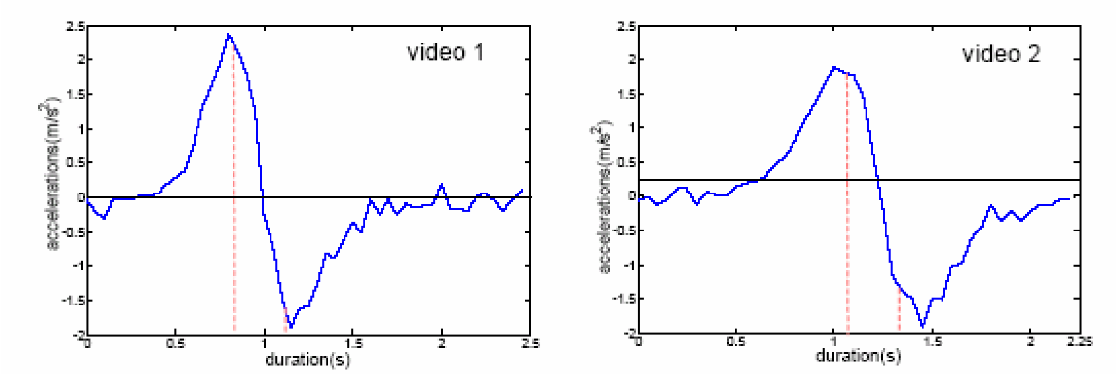

Figure 7: Acceleration profile and duration of stone movement for a similar wave of 2.5 (video1) and 2.25 (video2) seconds [TROMP, 2004]

	critical velocity	critical acceleration
H= 15 cm, T=2.50 s	0.32 m/s	2.2 m/s^2
H= 15 cm, T=2.25 s	0.39 m/s	1.8 m/s^2

This can be worked out, and when a large number of tests is available, with some regression analysis the coefficients of the Morrison equation can be determined.

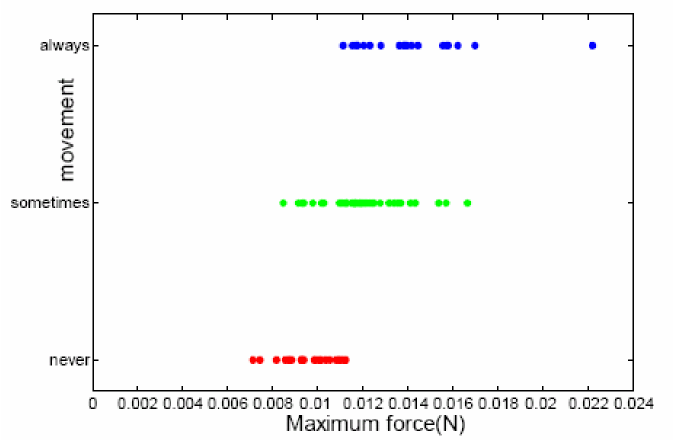

Figure 8: Movement/ no-movement of stones in one of the tests with stones of 9.6 mm, and calibrated Morison coefficients [TROMP, 2004]

As an example is given the test results for stones of 9.6 mm In figure 8 the force on a single rock is computed, using the Morison-equation (3) and the coefficients C_B = 0.55 and C_M = 3.75. These coefficients are different from the ones found in the experiment with permanent flow, but in the same order of magnitude. The quotient C_M/C_B = 7, which is lower than in case of permanent flow.

Apart from the approach via Morison type equation, also a stability criterion on basis of waveforms and a stability on basis of entrainment were elaborated. It is outside the scope of this paper to discuss this. For details is referred to TERRILE *ET AL*. [2005].

Relation to breakwater stability

From the above tests follows that the stability of rock is a function of both velocity and acceleration. In deep water and in case of horizontal beds there is a phase shift of $\pi/2$

between maximum velocity and maximum acceleration, which makes that the effect of acceleration can be neglected. However, in case of shoaling waves, especially on steep slopes, this phase difference is decreasing. At the same time the value of the acceleration is increasing relatively to the increase in the maximum velocity. Both effects make that acceleration is relevant for the determination of stability of rock on a sloping structure.

In the present stability equations the load parameter is a function of the energy density spectrum at the toe of the structure. But an energy density spectrum does not contain any information on the phase shift. Also the energy density spectrum does not contain any information on wave asymmetry. Therefore the present stability equations cannot distinguish between waves which have at the toe of the structure an identical energy density spectrum, but a different phase angle between velocity and acceleration. This phase angle depends, amongst others, on the slope of the foreshore.

Because of higher order effects the wave crests become steeper and the wave troughs become flatter. This increases the velocities and accelerations considerably. The wave steepness (as defined by the ratio between wave height and wave length) does not change, but when the steepness of only the part above the still water line is considered, this steepness increases significantly. The peakedness (kurtosis) of the waves is basically included in the wave spectrum, because it leads to a shift of energy to a higher frequency. In case the waves change from a pure sinus to a more trochoidal wave, the total energy in the spectrum remains the same, but some energy is shifted to lower frequencies. It is questionable if only using the $T_{m-1,0}$ is sufficient to account for this effect.

From the tests by Tromp and Terrile follows that, for a given velocity, the stability decreases because of the effects of acceleration. In order to take these effects into account one may start with the Van der Meer equation, and add one parameter. This parameter should have a value of 1 for deep water and horizontal foreshores. In case of a sloping foreshore this parameter should increase. Given the fact that the tests of Van Gent with structures on a steeper foreshore lead to a line which is in the order of 7 % higher, on can expect that in case of steep foreshores the coefficient should get a value of 1.1.

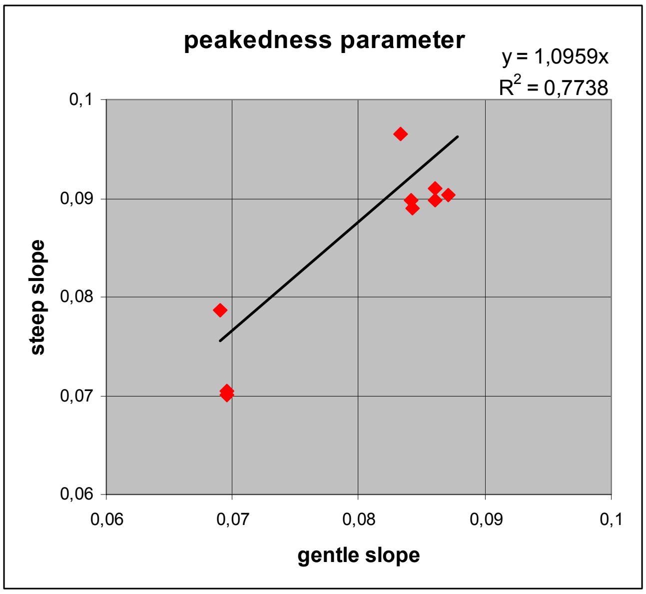

Figure 9: The peakedness parameter for both the gentle and the steep slope

This parameter should preferably be a function of the peakedness and the phase shift between acceleration and velocity. For the peakedness one may use the ratio:

$$H_{spike} = \overline{H^3} / \overline{H^2}$$

This parameter can be calculated for the wave observations at the toe for both the gentle and the steep foreshore. The result of this is plotted in figure 9. It shows that in this case the value of H_{spike} is in the order of 10% more for a steep slope than for a gentle slope. It seems that this parameter could be used to include in a stability equation to take into account the effect of a very steep foreshore.

Conclusions

It seems that the stability of rock in breakwater slopes above steep foreshores does not only depend on parameters described in a spectrum, but also on parameters like the peakedness and the wave asymmetry. A more universal stability equation should therefore include a parameter which describes these aspects. The peakedness of the waves is a promising parameter.

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