

On the stability of berm breakwaters in shallow and deep water.

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Abstract.

The paper describes laboratory tests on berm breakwaters in shallow water, e.g. in water depths where waves might break before they break on the breakwater. Analysis have been made of test results for breakwaters in shallow and deep waters and a unified design equation is presented for the berm recession of berm breakwaters in shallow and deep water.

Introduction.

Rubble mound berm breakwaters are increasingly used throughout the world. The main advantage of the berm breakwater is that lower stone weights are required on a berm breakwater than on a conventional rubble mound breakwater of quarried rock.

During the EU MAST II Berm Breakwater Structures project model tests were carried out for a berm breakwater, including a breakwater head, in deep water at the Danish Hydraulic Institute, DHI (1995), Juhl et al (1996), while tests in shallow water on a trunk section and including a head were carried out at SINTEF, Tørum (1997), as supplementary tests to the more extensive tests at DHI. In addition 2D tests were carried out at DHI, DHI (1996), Juhl and Sloth (1998). "Deep water" means in this context that the waves will not break until they break on the breakwater, while "shallow water" means that some of the larger waves will break before they arrive at the breakwater due to depth limitation. In the present paper we describe the shallow water tests carried out at SINTEF and give some brief results from them. We further include results from other tests series, notably from Andersen and Fleming (1991), DHI(1995), Lissev (1993), Tørum (1988) and Tørum (1997, 1997A) to arrive at an equation for the recession of the berm as a function of the stone diameter, the wave height and wave period. This equation can be considered as a design equation for the berm recession.

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Model Test Set-up.

The SINTEF "shallow" water model test set-up is shown in Figure 1. The waves were generated in 0.70 m water depth. After travelling for approximately 15 m, the waves climbed a slope of 1:30 to a horizontal plateau where the water depth was 0.25 m and where the breakwater model was placed.

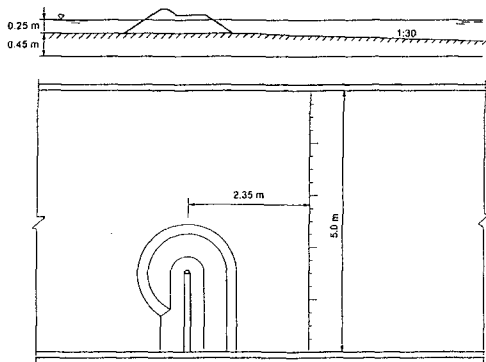


Figure 1. Test set-up.

Three different cross-sections were tested, A, B and C, Figure 2. Section A is almost a copy of the section tested at DHI, except that the water is shallower and except that the initial slope of the outer slope of the berm was 1:1.5 while this slope was 1:1.1 for the DHI tests. The gradation of the berm stones was such that $D_{15} \approx 0.018$ m, $D_{50} \approx 0.022$ m and $D_{85} \approx 0.030$ m.

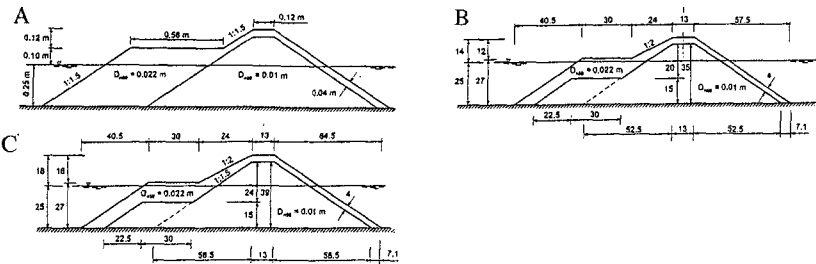


Figure 2. Tested cross-sections, A, B and C.

Wave measurements.

Wave measurements were carried out without the breakwater model being present in the flume. The wave measurements were carried out at the location of the centreline of the breakwater head with water depth 0.25 m, and in the deepest part of the flume, where the water depth was 0.70 m. The target spectra in "deep" water were JONSWAP spectra with an enhancement factor $\gamma = 3.0$.

Figures 3 and 4 show the relation between the measured significant wave heights, $H_s = H_{m0}$, and the measured zero-upcrossing periods, T_z , at water depths 0.70 m and 0.25 m respectively. The measured peak period, T_p , is also shown. However, the peak periods are not as "robust" as the zero-upcrossing periods as it may be somewhat arbitrary for which frequency the peak in the calculated spectrum will occur.

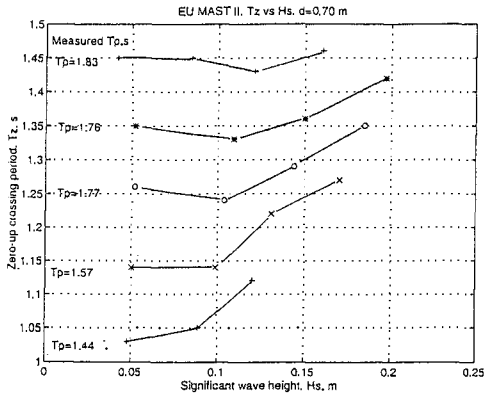


Figure 3. Measured zero-upcrossing periods T_z , peak periods, T_p , and significant waveheights, $H_s = H_{m0}$. Water depth $d = 0.70$ m.

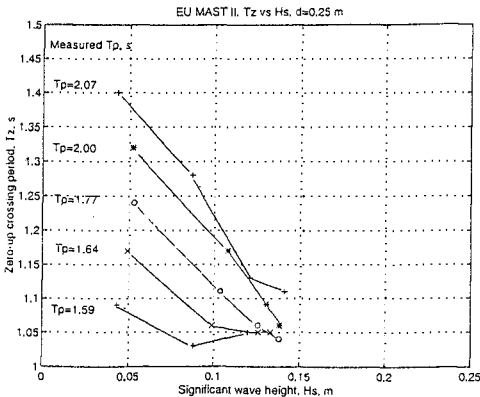


Figure 4. Measured zero-upcrossing periods, T_z , peak periods, T_p , and significant wave heights, $H_s = H_{m0}$. Water depth $d = 0.25$ m.

In shallow water the highest waves will break. Thus the ratio between the maximum wave height, H_{max} , and the significant wave height, H_{m0} , is expected to be lower in shallow water than in deep water. $H_{m0} = 4 (m_0)^{0.5}$, where m_0 = area under the spectrum. Figure 5 shows this ratio for different significant wave heights for $T_p=1.77$ s

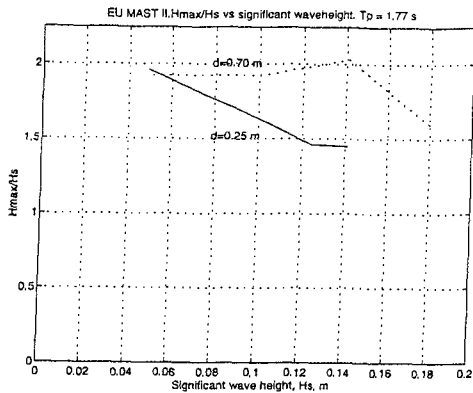


Figure 5 Ratio H_{max}/H_s ($H_s = H_{m0}$) for wave period $T_p = 1.77$ s and for water depth $d = 0.70$ and 0.25 m.

Test program.

The test program we used was approximately the same as for the DHI tests, DHI (1995), Juhl et al (1996) with respect to the number of waves for each wave step, except that we included a very long test run at the end of each test series. All tests were carried out with a target steepness of the waves of $s = 2\pi H_s/gT_z^2 = 0.05$, where g = acceleration of gravity. This is the same steepness as used for most of the DHI tests. After discussions with other partners in the EU MAST II Berm Breakwater Structures project we decided to use, as has been common practise, the zero up-crossing period in deep water ($d = 0.70$ m) and H_{m0} from shallow water (local wave height at $d = 0.25$ m) when calculating the steepness s . The test program is shown in Table 1. Here $D_{n50} = (W_{50}/\rho_s)^{0.333}$ and $\Delta = (\rho_s/\rho_w) - 1$, W_{50} = fifty percent of the stones are larger (and smaller) than W_{50} , ρ_s = specific mass of the stones and ρ_w = specific mass of the water.

Table 1. Test program for the shallow water tests at SINTEF.

$H/\Delta D_{n50}$	Number of waves.
2.05	2000
2.40	2000
2.85	2000
3.40	2000
≈3.85	2000
2.05	1000
2.40	1000
2.85	1000
3.40	1000
≈3.85	1000
≈3.85	10.000

The wave conditions during the shallow water testing at SINTEF are shown in Table 2.

Table 2. Wave conditions during the 3D tests in shallow water at SINTEF.

"Deep" water.	"Shallow" water				
T_s	H_m	$H_o = H_m / \Delta D_{\text{min}}$	T_s	s_d	s_s
1.05	0.086	2.05	1.05	0.049	0.049
1.14	0.101	2.40	1.06	0.049	0.058
1.24	0.120	2.85	1.09	0.049	0.064
1.35	0.138	3.40	1.09	0.048	0.074
1.45	0.160	3.85	1.10	0.048	0.084

As mentioned it was the wave steepness based on the "deep" water wave periods that was the governing factor for establishing the test program waves. However, it is the steepness based on the shallow water wave period that the breakwater "feels". Hence this steepness should have been used rather than the steepness based on the "deep" water wave period. In the unified analysis (see later) we have used the wave periods for shallow water.

The tests were run in the following way:

After building the breakwater model the wave/breakwater parameter H_o , the stability number, was increased in steps according to Table 1. 2000 waves were run for each step. Profiles were taken with a laser distance measurement system, generally after each step. The distance between each profile was 0.10 m and the distance between each measurement point in a profile was 0.02 m.

After this reshaping process with five H_o steps, H_o up to 3.85, the steps were repeated again, but now with 1000 waves in each step. Since no "damaging" effects occurred on Breakwaters A and C at the end of this sequence of runs, we continued to run 10.000 waves with $H_o = 3.85$.

For Breakwater B the planned test program was the same as for Breakwaters A and C. However, when running the tests a slight damage on the rear side due to wave overtopping occurred for $H_o = 3.40$ in the first wave step sequence (the sequence with 2000 waves for each step). For $H_o = 3.85$ the damage on the rear side was so large that we decided to terminate this step after approximately 1600 waves.

Test results for the 3D tests in shallow water at SINTEF.

We will present only some few of the results from this particular study and later include the results in an analysis of the reshaping of berm breakwaters were we also include results from other test series.

Figures 6 and 7 show an oblique "view" of Breakwater A before and after the testing, while Figure 8 shows one of the profiles before the test started and after completion of the test program for Breakwater A.

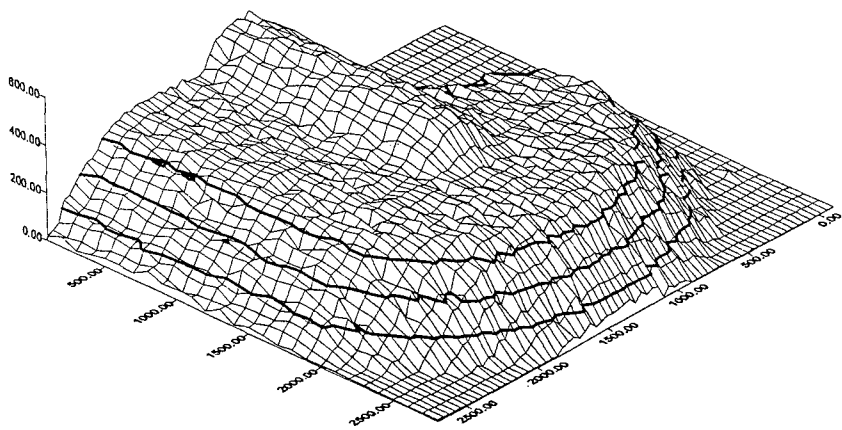


Figure 6. Oblique view of Breakwater A before testing.

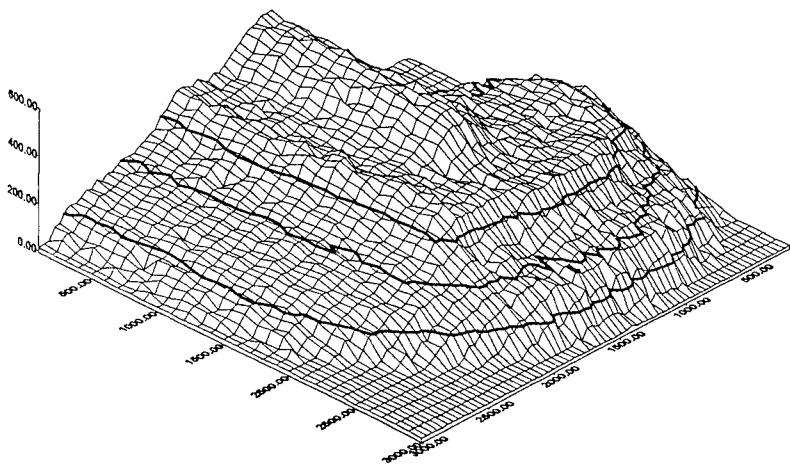


Figure 7. Oblique view of Breakwater A after testing.

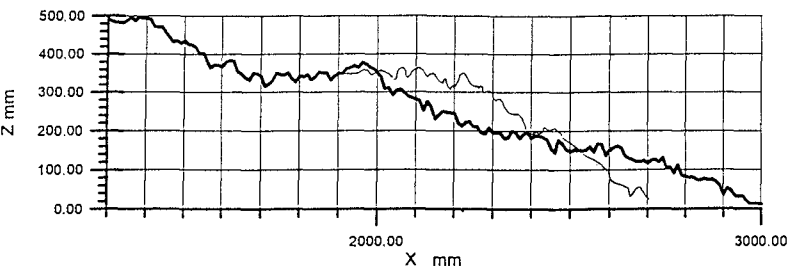


Figure 8. Breakwater A. Cross-sections at 1200 mm before and after testing.

Both Breakwater A and C were considered to be dynamically stable in the sense that no core material was visible after the testing had been completed and no damage had occurred on the rear side. Also it was noted that not much of the berm stones on the breakwater head had been thrown into the area behind the breakwater.

We took stone samples of the surface stone material at the still water line and at the toe of the berm after completing the test program for Breakwater C. It was interesting to note that the stones at the toe are significantly larger, (D_{n50}), than at the still water line. At first instance this observation may seem unexpected because the smaller stones are easiest to move. Westernen and Tørum (1997) measured the wave forces on a single armour unit placed at different positions along the slope of a reshaped berm breakwater. They found that the highest forces occurred above the still water line, and than to some extent as "impact" forces. This fact together with the observation of the smallest stones at the still water line support the concept that the stones are "knocked" loose above and around the still water and roll down the slope. The larger stones have a larger momentum and rolls more easy than the smaller stones. Hence the larger stones tend to move a longer distance than the smaller stones. The effect is probably the same as we see in a rock slide, where the largest stones are located at the toe of the slide.

Figure 8 reveals that only a part of the berm of Breakwater A was reshaped during the SINTEF "shallow" water tests. The DHI "deep" water test results, DHI (1995), Juhl et al (1996), show that for the same test program that almost the whole width of the berm was reshaped. When analysing the results with respect to an understanding of the differences, we saw that tests carried out under apparently similar conditions in different laboratories have given what we at first instance will consider as significant result differences. We will therefore review some of these test and include the results in a unified analysis.

Analysis of berm breakwater test results from different tests series.

The following analysis is mainly considering the two-dimensional case with waves normal to the breakwater axis.

The geometry of a berm breakwater is defined by notations as shown in Figure 9, while Figure 10 show notations for the reshaped berm breakwater. f_b = berm width, f_v = berm height, w = crest width, R_c = crest height, d = water depth, m and n defines the "inner" and "outer" slopes, Rec = recession of the reshaped berm, l_s = step length, h_s = step height, A_1 = eroded area and A_2 = deposited area.

Occasionally we will also see from berm breakwater test results that some of the berm stones will move above the original berm elevation. This phenomenon could be very pronounced for finer material than normally found in berm breakwater berms, e.g. van der Meer (1988), but is not very pronounced for berm breakwaters.

The most relevant non-dimensional parameters used for the analysis of berm breakwater test results are the following:

$$Ho = \frac{H_s}{\Delta D_{n50}} \quad (1)$$

$$HoTo = \frac{H_s}{\Delta D_{n50}} \sqrt{\frac{g}{D_{n50}}} T_z \tag{2}$$

Ho was introduced by Hudson (1958) (as the stability number N_s) and HoTo was introduced by van der Meer (1988) as a dimensionless wave period parameter. Typical values for berm breakwaters are $Ho < 3$ and $HoTo < 100$.

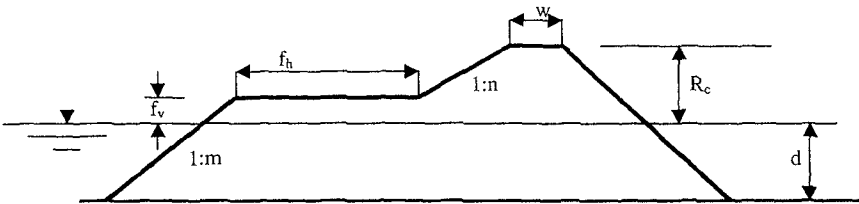


Figure 9. Geometry definitions of a berm breakwater.

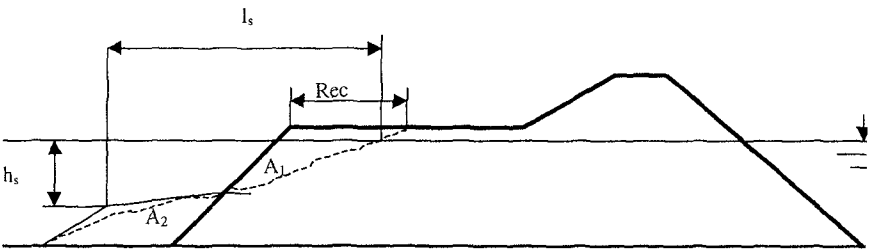


Figure 10. Reshaping parameters of a berm breakwater.

Figure 10 shows that from a continuity point of view the areas A_1 and A_2 must be equal for the two-dimensional case. van der Meer emphasised on the step length l_s and step height h_s (among other geometrical parameters) as a kind of "universal" parameters as they were found to be to a large extent independent of the original profile slope etc observed in his tests, at least for large values of $HoTo$ ($HoTo > 1000$). van der Meer (1988) obtained values for h_s and l_s as well as the shape of the profile from tests on straight slopes with $HoTo = 100 - 50.0000$. Based on some limited number of tests van der Meer gave also an indication of how h_s and l_s was dependent on the waterdepth. When the values of h_s and l_s and the profile shape are known it is a matter of trial and error procedure to find the recession such that $A_1 = A_2$. However, van der Meer (1988) did not cover so well the lower range of $HoTo$ ($HoTo < 100$) as found for berm breakwaters.

We have tried to measure values of h_s and l_s from different profiles from different berm breakwater test series. Our conclusion is that it not so easy to obtain values of h_s and l_s with any good accuracy. Hence it is not so easy to predict the

recession, Rec, with any accuracy either. We will therefore consider the direct measurement of the recession, Rec. This is a length parameter that can be measured with a reasonable good precision and it is a vital parameter for berm breakwater design.

Based on test results from different berm breakwater test series in "deep" water we have in the diagram of Figure 11 plotted the dimensionless parameter Rec/D_{n50} vs. HoTo . The results are from Andersen and Fleming (1991), DHI (1995) (DHI 3D tests), Lissev (1993), Tørum (1988), Tørum (1997, 1997A).

The main characteristics of the test series are the following:

Andersen and Fleming (1991):

Water depth $d = 0.67 - 0.90$ m, berm height $f_v = 0.10$ and 0.20 m, berm width $f_h = 0.65$ m, outer slope $1 : 1.1$, stone diameter $D_{n50} = 0.034$ m, wave steepnes $s = 0.05$

DHI (1995)

Water depth $d = 0.55$ m, berm height $f_v = 0.10$ m, berm width $f_h = 0.65$ m, outer slope $1 : 1.1$, stone diameter $D_{n50} = 0.022$ m, wave steepnes $s = 0.03$ and 0.05

Lissev (1993)

Water depth $d = 0.79$ m, berm height $f_v = 0.08$ m, berm width $f_h = 0.65$ m, outer slope $1 : 1.25$, stone diameter $D_{n50} = 0.034$ m, wave steepnes $s = 0.045$

Tørum (1988)

Water depth $d = 0.50$ m, berm height $f_v = 0.10$ m, berm width $f_h = 0.45$ m, outer slope $1 : 1.5$, stone diameter $D_{n50} = 0.029$ m, wave steepnes $s = \text{varying } 0.018 - 0.075$

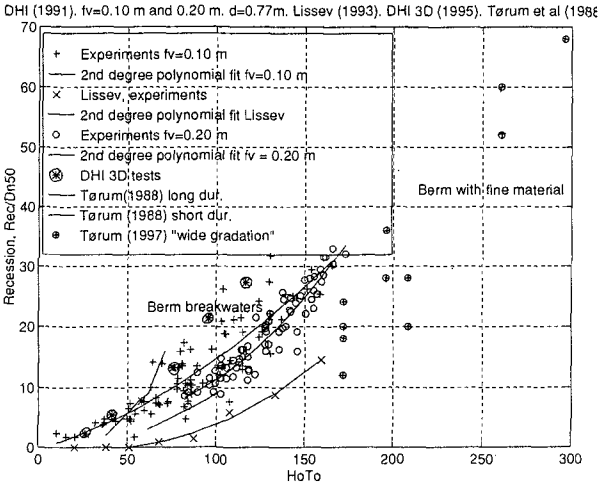
Tørum (1997A)

Water depth $d = 0.40$ m, berm height $f_v = 0$ m, berm width $f_h = 0.59$ and 1.09 m, outer slope $1 : 1.3$, stone diameter $D = 4 - 60$ mm, $D_{50} = 15$ mm (gradation curve values), wave steepnes $s = 0.05$

To some of the data points in Figure 11 a second degree polynomial fit has been provided, based on the "least square" principle. There are some minor differences in these test series with respect to test set-up and test programs. But they are not that different and we will consider the test series to be reasonable homogenous.

Based of van der Meer's concept we see that the recession, Rec, of a homogenous berm for a given wave condition could be dependent of the berm height, f_h , and the water depth h . We see however from Figure 11 that there is not much of a difference in results of the tests with $f_v = 0.10$ m and $f_v = 0.20$ m.

For "shallow" water there are fewer results than for "deep" water. We have in Figure 12 plotted data from tests at DHI (1996) and SINTEF, Tørum (1997A). We have for HoTo used the wave height and wave periods as measured at the shallow water location. For the SINTEF tests the T_z values were measured, while for the DHI tests the wave spectrum was measured and T_z has been taken as $T_z = T_{02}$.



+ Experiments $f_v = 0.10$ m from Andersen and Poulsen (1991).
 o Experiments $f_v = 0.20$ m from Andersen and Poulsen (1991).

Figure 11. Recession of the berm of berm breakwaters from different test series. "Deep" water.

If we compare the results of Figures 11 and 12 we see that the results are comparable when the local wave parameters are used to calculate $HoTo$.

There is a considerable scatter in the test results shown in Figure 11. The scatter is between different tests in the same series of tests at a specific laboratory and between test series in different laboratories. But the scatter is probably not more than we normally find for breakwater testing

We have not been able to resolve why there are differences and scatter in the results. We will thus presently consider the scatter of the data as "natural" and consider all results of equal value. Based on this consideration we have in Figure 13 plotted the results from Figure 11 and Figure 12 in the same diagram. We have in Figure 13 omitted the data from Tørum (1997A) since those data were from tests on aberm of quarry run material not typical for berm breakwaters.

We have provided a second degree polynomial fit to the data. We also tested out a third and a fourth degree polynomial fit, but these fits did not appear to be more accurate than the second degree polynomial fit.

The equation for the second degree polynomial fit is given by:

$$\frac{Rec}{D_{n50}} = 0.00073908(HoTo)^2 + 0.0498855(HoTo) + 0.604 \quad (3)$$

This equation may be considered as a design equation for the recession of the berm.

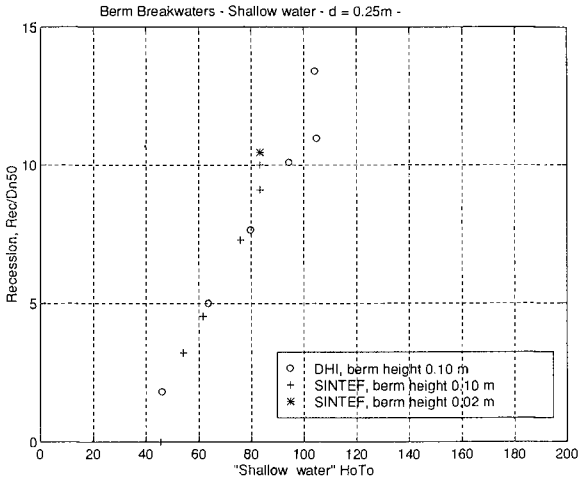


Figure 12. Recession of the berm of berm breakwaters from different test series. "Shallow" water. Note that HoTo is based wave parameters measured at the "shallow" water location of the breakwater.

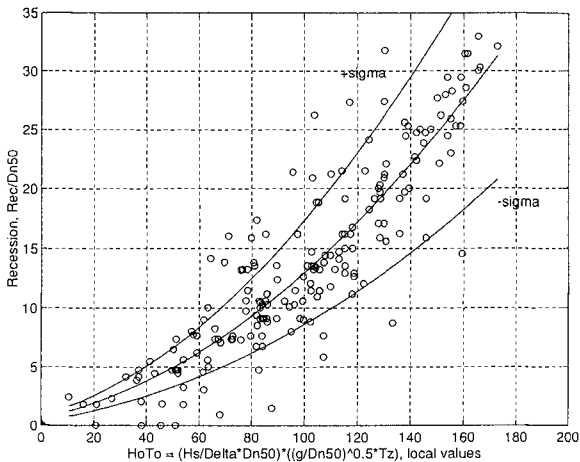


Figure 13. Recession of the berm of a berm breakwater in "shallow" and "deep" water as function of "local" HoTo-values. 2nd degree polynomial fit $\pm\sigma$

The scatter of the data has been analysed in the following way:

$$\frac{f - f_k}{f_k} = f(HoTo) \quad (4)$$

where f = datapoint for a given HoTo-value
 f_k = value after 2nd degree polynomial fit
 $f(HoTo)$ = function of HoTo

The result of this analysis is shown in Figure 14 where we have also indicated the standard deviation $\sigma = 0.337$. We have also included the "2nd degree polynomial fit" $\pm \sigma$ in Figure 13.

We have further in Figure 15 shown the data compared to a normal distribution function. The design equation, Eq. (3) and the scatter information may be used in a probabilistic analysis of berm breakwaters.

DHI (1996), Juhl and Sloth (1998), carried out tests on "islandic" type berm brakwaters. The idea is to place the largests blocks from the quarry on top of the berm as a composite breakwater. We have analysed the results of these tests in a similar way as we analysed the results from the tests with homogenous berms. For D_{n50} we used the D_{n50} for the largest stones on top of the berm. The results are shown in Figure 16 where we also have plotted the design equation, Eq. (3). The design equation works also reasonably well for the composite berm breakwater.

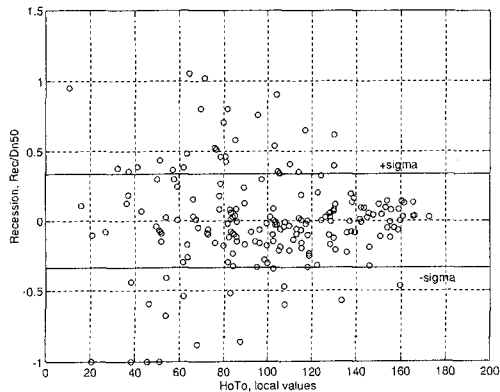


Figure 14. $(f-f_k)/f_k$ as function of HoTo (local values). The standard deviation $\sigma = 0.337$.

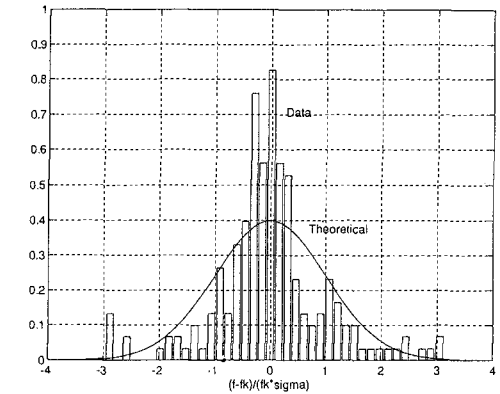


Figure 15 Standardised distribution of $(f-f_k)/f_k$ compared to a theoretical normal distribution.

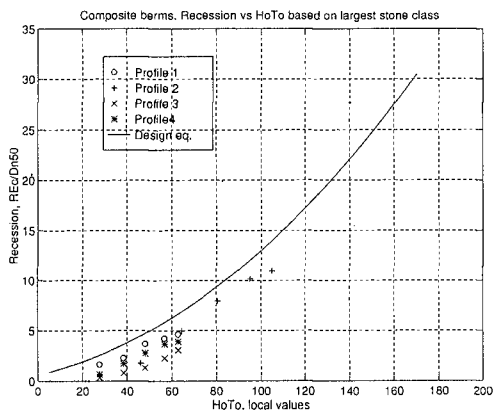


Figure 16. Recession vs. $HoTo$ for composite berm breakwaters. "Local" wave parameters. D_{n50} of largest stone class. Profile 2 is a homogenous profile. Design eq. is Eq (3).

Discussion and conclusion.

We have analysed two-dimensional test results and arrived at a simple design equation for the recession of the berm of a berm breakwater without major overtopping. This equation may be used, at least in the conceptual design phase, for berm breakwaters in water depths and for storm duration that are normally encountered for berm breakwaters.

If the waves are oblique to the breakwater, the lateral transport of the stones have to be considered, e.g. Alikhani et al (1996).

There is no well establish criteria for which stability number Ho or wave period parameter $HoTo$ the design should be accepted. Such criteria should apparently be linked to the mechanical strength of the stones.

Acknowledgement.

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