CHAPTER 76

Rear Side Stability of Berm Breakwaters

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Abstract

With the aim of providing improved methods for preliminary design of berm break-waters, a series of physical model tests and a parameter study with special emphasis on the rear side stability of a trunk section have been carried out at the Danish Hydraulic Institute (DHI). The model tests included different geometries of the berm breakwater profile and a range of wave conditions. For each profile, the wave condition resulting in sea side and/or rear side damage was determined. As a hydrodynamic description of the overtopping waves would be very comprehensive, and at present is not available, a surf similarity approach in combination with a force balance for the armour stones has been chosen. A parametric expression for the rear side stability has been established and found to be in fairly good agreement with the model test results.

Introduction

Existing experience with berm breakwaters provides some insight in the behaviour of the various parts of a berm breakwater. Most research during the last ten years has concentrated on the sea side stability of the trunk section, ie the development of the berm profile. No systematic work on the rear side stability has previously been reported and hence a comprehensive study focusing on the rear side stability of the trunk section was carried out at DHI. As it is still not possible to give an adequate hydrodynamic description of the overtopping phenomenon, eg in the form of a numerical model, it was decided to carry out a series of physical model tests and an associated parameter study.

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In the present study, the rear side stability is treated as a traditional static stability phenomenon. Emphasis has been put on inclusion of a large number of physical parameters enabling the description to cover a variety of berm breakwater designs: wave height and steepness, crest height, rear side slope, effective sea side slope, stone diameter, relative density and natural angle of repose.

The stability criterion established has been compared to the results of the model tests carried out and in addition to an expression given by Van der Meer and Veldman.

Model setup and procedure

A wave flume with a length of 65 m and a width of 1.8 m was used for model testing in four water depths of 0.67 m, 0.77 m, 0.87 m and 0.97 m. The profile tested is shown in Figure 1. Crushed stones were used both for the core, D_{50} = 0.011 m (50% exceedance), and for the berm and armour layer, D_{n50} = 0.034 m. D_{n50} is the nominal diameter given as $(M_{50}/\rho_s)^{1/3}$, where M_{50} is the mass of the stones (50% exceedance) and ρ_s is the density of the stones. The grading of the berm and armour material equalled D_{n85}/D_{n15} = 1.35 and the relative density equalled Δ = 1.68. The armour layer thickness on the crest and rear side was twice the value of D_{n50}. The model study covered variations in the following parameters:

- w_c, width of the crest, 0.175 m and 0.30 m
- R_c, freeboard of the crest, 0.20 m, 0.30 m and 0.40 m
- f_h, width of the berm, 0.45 m, 0.65 m, 0.85 m and 1.05 m
- f_v, freeboard of the berm, 0.10 m and 0.20 m.

Figure 1. Test profile.

Tests were carried out in test series with successively increasing wave height, H_{mo}, and wave period, T_{02}, from test run to test run, with a fixed fictitious wave steepness s_{02} = $2\pi H_{mo}/gT_{02}^2$ equal to approximately 0.030 and 0.044 respectively. Each test series consisted of approximately 1,000 irregular waves. The incident wave characteristics in front of the berm breakwater, H_{mo} and T_{02}, and the reflection coefficient were calculated using a multi-gauge technique.

The berm profile was measured after each test run. Damage to the seaward side of the berm breakwater was defined to occur when the entire top of the berm was eroded. Rear side damage was defined as a settlement of the rear side armour layer which in some cases was followed by an exposure of the core.

Table 1 gives an overview of the 23 tested profiles for which rear side damage was observed prior to or coincident with damage of the sea side. The wave conditions, H_{mo} and T_{02}, resulting in rear side damage are also shown in Table 1.

Table 1. Rear side damages.

Note: A is the berm area and $\tan\alpha$ the effective sea side slope, as defined below.

wc (m)	Rc (m)	fh (m)	fv (m)	h (m)	A (m²)	H_{mo} (m)	T_{02} (s)	s_{02}	$\tan\alpha$
0.175	0.2	0.45	0.1	0.97	0.25	0.176	1.67	0.0404	0.39
0.175	0.2	0.85	0.1	0.77	0.59	0.228	1.84	0.0431	0.36
0.175	0.2	0.65	0.1	0.97	0.47	0.211	1.78	0.0427	0.37
0.175	0.2	1.05	0.1	0.77	0.76	0.228	1.84	0.0431	0.29
0.175	0.2	0.85	0.1	0.97	0.68	0.184	2.08	0.0272	0.43
0.175	0.2	1.05	0.1	0.97	0.89	0.202	2.13	0.0285	0.30
0.175	0.3	0.45	0.2	0.77	0.25	0.186	2.06	0.0281	0.45
0.175	0.3	0.65	0.1	0.77	0.41	0.209	2.11	0.0301	0.38
0.175	0.3	0.65	0.1	0.77	0.41	0.219	1.81	0.0428	0.36
0.175	0.3	0.45	0.2	0.87	0.25	0.184	2.00	0.0295	0.42
0.175	0.3	0.65	0.2	0.77	0.44	0.210	2.12	0.0299	0.39
0.175	0.3	0.85	0.1	0.77	0.59	0.228	2.20	0.0302	0.33
0.175	0.3	0.65	0.2	0.87	0.47	0.235	2.17	0.0320	0.37
0.175	0.3	0.85	0.2	0.77	0.64	0.256	1.93	0.0440	0.31
0.175	0.3	1.05	0.1	0.77	0.76	0.232	2.24	0.0296	0.29
0.175	0.3	1.05	0.1	0.77	0.76	0.270	1.99	0.0437	0.29
0.175	0.3	1.05	0.2	0.77	0.83	0.271	1.99	0.0438	0.29
0.3	0.2	0.65	0.1	0.97	0.47	0.207	1.71	0.0453	0.37
0.3	0.2	0.85	0.1	0.97	0.68	0.222	1.78	0.0449	0.32
0.3	0.2	1.05	0.1	0.97	0.89	0.241	1.83	0.0461	0.28
0.3	0.3	1.05	0.1	0.77	0.76	0.238	2.21	0.0312	0.27
0.3	0.3	1.05	0.2	0.77	0.83	0.248	2.19	0.0331	0.30
0.3	0.3	1.05	0.2	0.77	0.83	0.279	1.97	0.0460	0.28

Data Analysis

During the tests, it was observed that the developed sea side profile can be characterised as three slopes: an upper steep slope, a lower slope close to 1:4 or flatter and finally a steep slope intersecting the seabed, see Figure 2. This is a well known observation confirmed by several other researchers.

meter

Figure 2. Example of development of sea side profile.

Rear side damage of a trunk section happens at almost fully developed sea side profile. A typical development is that a few stones just above still water level at the rear side are displaced downwards during a series of wave overtoppings (cf Figure 3), and after another few severe wave overtoppings, a settlement of the rear side armour layer occurs, possibly resulting in an exposure of the core.

Figure 3. Initiation of rear side damage.

The stability of the rear side depends on the sea side profile, crest height, crest width, rear side slope, stone diameter, relative density and natural angle of repose. It appears from the analysis of the present model test results that a variation with the crest width is not visible. New model tests with larger crest widths are presently being carried out in order to include this parameter as well. The following analysis is considered to be valid for the crests widths in Table 1, and hence w_c is not included in the analysis.

It is assumed that the speed of the overtopping water is the governing factor in determining the rear side stability. The speed at the crest, U_R, is chosen as a reference speed.

U_R can be found by putting the potential at the crest equal to the potential at the still water level at the seaward side:

$$U_R^2 = 2g \ (R_{ui} - R_c) \tag{1}$$

where R_{ui} is the run-up on a hypothetical slope of the sea side, with index i indicating a fraction of the waves. The hydraulic parameters are defined in Figure 4.

Figure 4. Definition of hydraulic parameters.

Pilarczyk (1990) applied the same reference velocity for the description of the rear side stability of dikes, resulting in a final stability criterion with large resemblance to the criterion derived below.

For the berm breakwater rear side, a traditional static stability criterion is applied. The stability of a single stone at still water level is expressed by a force balance parallel to the rear side slope, cf Figure 5:

$$F_D + W_s \sin\beta - \mu \ (W_s \cos\beta - F_L) < 0 \tag{2}$$

where F_D is the drag force, F_L is the lift force, W_s is the submerged weight, and β is the rear side slope angle. The resistance against rolling and sliding μ can be found as $\mu = \tan\phi$, where ϕ is the natural angle of repose.

This yields:

$$F_D + \mu F_L < W_s \ (\mu\cos\beta - \sin\beta) \tag{3}$$

The submerged weight equals:

$$W_s = \Delta\rho g \ D_{n50}^3 \tag{4}$$

where ρ is the density of water and D_{n50} is the nominal diameter.

Figure 5. Forces acting on a single stone.

The left side can be expressed as:

$$F_D + \mu F_L = (C_D + \mu C_L) \frac{1}{2} \rho U_R^2 D_{n50}^2 \qquad (5)$$

where C_D and C_L are force coefficients. For convenience, the nominal diameter is applied in the above expression. Combining (1), (3) (4), and (5) and re-arranging, the stability criterion now yields:

$$R_c > R_{ul} - \Delta D_{n50} \frac{\mu \cos\beta - \sin\beta}{C_D + \mu C_L} \qquad (6)$$

For a relatively flat sea side slope, it is assumed that the run-up can be expressed as a function of the surf similarity parameter, cf CIRIA/CUR (1991):

$$R_{ul} = a \, \xi_{02} \, H_{mo} \qquad (7)$$

where ξ_{02} is the surf similarity parameter (Iribarren number) given as:

$$\xi_{02} = \frac{\tan\alpha}{\sqrt{s_{02}}} \qquad (8)$$

where $\tan\alpha$ is the effective sea side slope and the factor a is a constant close to 1. In the following, a is kept equal to 1.

The effective sea side slope is for the present purpose defined as a straight line through the toe of the lower slope, where the influence of the breaking waves becomes significant, and up to the seaward face of the crest, cf Figure 4.

In general, the effective sea side slope represents the upper part of the berm area shaped by the incident waves. The effective sea side slope is considered to be

a good measure of the part of the berm area which is active in the wave deformation irrespective of the water depth in front of the structure.

For the specific test programme, the effective sea side slope for the cases with rear side damage has been plotted against the berm area, cf Figure 6. It is observed that the effective sea side slope decreases with the berm area.

Figure 6. Effective sea side slope vs berm area for the cases with rear side damage.

Combining (6), (7) and (8) gives for the stability criterion:

$$R_c > \tan\alpha \ \frac{H_{mo}}{\sqrt{s_{02}}} - \Delta D_{n50} \ \frac{\mu\cos\beta \ - \sin\beta}{C_D + \mu C_L} \qquad (9)$$

The above expression is made dimensionless by $H_{mo}/\sqrt{s_{02}}$.

$$\frac{R_c}{H_{mo}} \sqrt{s_{02}} > \tan\alpha \ - \left(\frac{H_{mo}}{\Delta D_{n50}} \ \frac{1}{\sqrt{s_{02}}}\right)^{-1} \frac{\mu\cos\beta \ - \sin\beta}{C_D + \mu C_L} \qquad (10)$$

For the observed rear side damages, the effective sea side slopes, tanα, have been divided into three equidistant intervals with the following limits: 0.27, 0.33, 0.39 and 0.45.

For the stone material applied, μ equals 0.9. For this value, the expression (10) has been calibrated to fit the observations. The best agreement was obtained with $(C_D + \mu C_L)$ equal to 0.08. Four different curves representing $\tan\alpha = 0.27$, 0.33, 0.39 and 0.45 respectively have been drawn in Figure 7. For all curves, the rear side slope equals the value of the actual test programme: $\tan\beta = 1{:}1.5$.

Figure 7. Stability of rear side.

Legend: Measurements: +: 0.27 < tanα < 0.33,
                                                        x: 0.33 < tanα < 0.39,
                                                        o: 0.39 < tanα < 0.45
                        Full drawn curves show the stability criterion (10)

It is seen that a fairly good agreement between the measurements and the stability expression is obtained. The measurements show that $R_c/H_{mo}\sqrt{s_{02}}$ increases with $H_{mo}/\Delta D_{n50} / \sqrt{s_{02}}$, which again shows that in the stability expression (10) the tanα term as well as the term including $H_{mo}/\Delta D_{n50} / \sqrt{s_{02}}$ are of importance.

For a specified wave condition, the rear side stability can according to (10) be increased in several ways:

- increase of crest height, R_c, which is the most traditional method
- increase of stone diameter, D_{n50}. In Norway, a berm breakwater has been constructed with larger stones on the rear side than on the sea side, cf Tørum et al (1990)

- increase of relative density, Δ
- decrease of rear side slope, $\tan\beta$

However, only the dependency on R_c has been studied experimentally.

The stability expression (10) bears some resemblance to the expression given by Van der Meer and Veldman (1992):

$$\frac{R_c}{H_s} \, s_{op}^{1/3} = K \qquad\qquad (11)$$

H_s is the significant wave height $\sim H_{mo}$
s_{op} is the fictitious wave steepness based on the peak period
K is a constant, which equals: $K = 0.25$ for start of damage
                                      $K = 0.21$ for moderate damage
                                      $K = 0.17$ for severe damage

The above expression (11) is based on a parameter fitting procedure applied to two different sea side geometries (one of them in two different scale ratios). Comparing to the stability expression (10), the major difference is that in (11), $R_c/H_{mo} \, s_{op}^{1/3}$ is constant, whereas the very similar quantity $R_c/H_{mo} \, \sqrt{s_{02}}$ in (10) depends on $H_{mo}/\Delta D_{n50} \, / \sqrt{s_{02}}$ and the effective sea side slope.

Conclusions

- Model tests with a range of berm breakwater profiles have been carried out at DHI.

- A parametric expression for the rear side stability has been made, cf (10).

- The expression includes the wave height and steepness, crest height, rear side slope, effective sea side slope, stone diameter, relative density and natural angle of repose.

- The measurements show that the rear side stability increases with a decreasing effective sea side slope. A decrease in the effective sea side slope can be obtained by increasing the berm area.

- Applying the surf similarity approach for a bermed profile gives reasonably good agreement between the derived expression for the rear side stability and the model test observations.

- New model tests with different crest widths are being carried out in order to examine the variation of the rear side stability for a wider range of this parameter.

Acknowledgements

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