Aalborg Universitet

Hydraulic Response of Rubble Mound Breakwaters

scale effects - berm breakwaters

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Appendix C

Existing Data from Previous Berm Breakwater Tests

In this chapter the existing tests on berm breakwater stability, overtopping, reflection and rear slope stability are described. The main part of existing data from stability tests with berm breakwaters has been carried out within the European Comission supported programme Marine Science and Technology (MAST), under the first and second programme (MAST 1 & 2) [Juhl et al., 1997]. Only very limited data exists on overtopping of berm breakwaters and no systematic study seems to exist.

In cases where the waves were measured at deeper water and not at the toe of the structure the wave parameters at the toe were calculated using the SWAN method [Holthuijsen et al., 2004]. The SWAN model gives generally very reliable estimates of wave heights even in very shallow water. However, SWAN does not give as reliable estimates of wave periods.

C.1 Instanes, 1987

2-dimensional physical model tests with a proposed berm breakwater cross section for the extension of the Årviksand harbour in Norwayin were performed in a wave flume by Instanes, 1987. The test setup is shown in Fig. C.1.

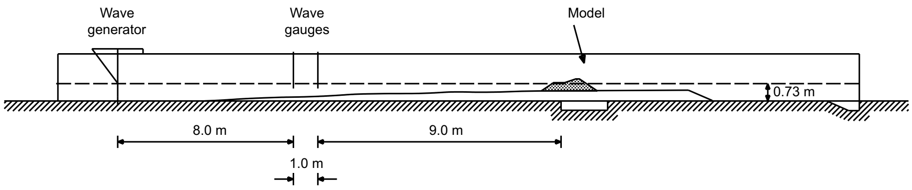

Figure C.1
Test setup used by Instanes, 1987.

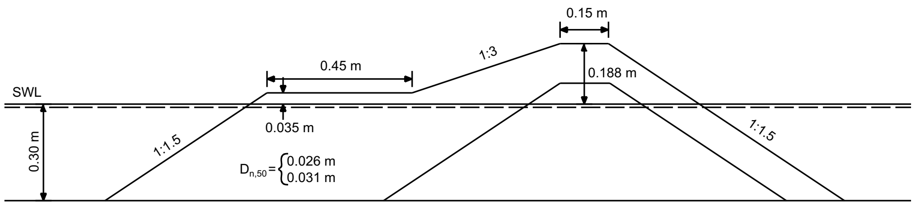

Figure C.2
Cross section tested by Instanes, 1987.

7 tests were performed with registration of front slope profile development. Due to limited storage capacity the wave parameters is given for the first 512 seconds out of a total length of the timeseries of 15 minutes. Waves were measured at deeper water using 2 wave gauges (see Fig. C.1) for seperation in incident and reflected waves using the method of Goda and Suzuki, 1976. At the measurement location the water depth was 0.54 m and the waves were non-gaussian with a skewness of 0.78 and kurtosis 3.82, and the highest waves were observed to break before the breakwater. Therefore the incident waves at the toe of the structure was calculated by the SWAN method. The skewness calculated by Eq. 4.24 is significantly smaller than mentioned by Instanes, 1987.

C.2 Tørum et al., 1988

Tørum et al., 1988 used the setup shown in Fig. C.3.

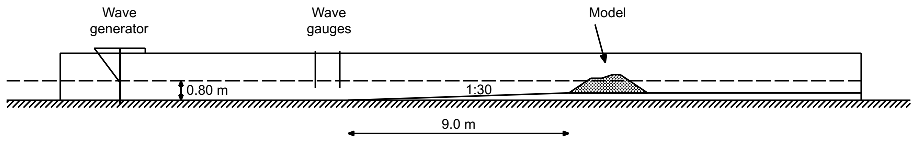

Figure C.3
Test setup used by Tørum et al., 1988.

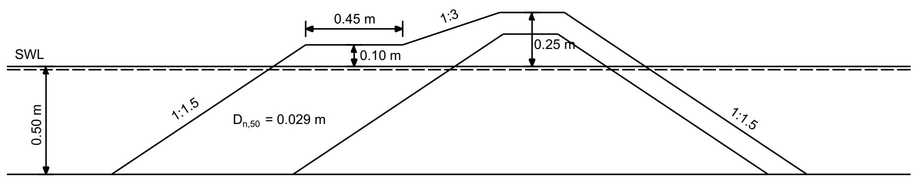

Figure C.4
Cross section tested by Tørum et al., 1988.

The waves were generated from the JONSWAP spectrum using a peak enhancement factor (γ) of 1 and 7 respectively. Tørum et al., 1988 found that a narrow spectrum gives slightly more reshaping than the wide spectrum with same H_{m0} and T_p. Van der Meer, 1988 found that the governing wave period was the mean period T_m and that the damage would be the same for same H_{m0} and T_m irrespective of the spectrum shape. The waves were measured on deep water and different wave shoaling and breaking could have taken place for the two γ values. The wave parameters at the toe were calculated from the SWAN method.

C.3 Van der Meer, 1988

Van der Meer, 1988 carried out a large test programme with all kinds of dynamically stable structures. Many of the tests were performed on a cross-section with a rather flat straight slope, but Test 380-395 are tests with berm breakwaters with down slope of 1:1.5.

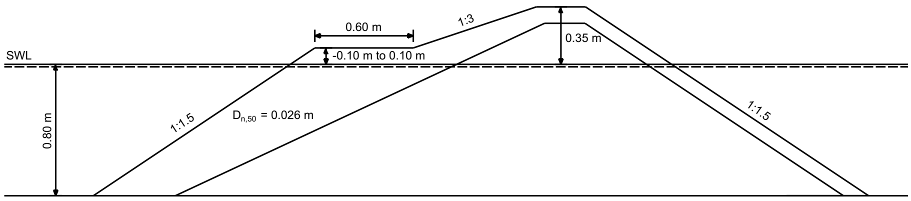

Figure C.5
Cross sections tested by Van der Meer, 1988.

The number of waves in each test was at least 3000, with intermediate profiles measured. A 1:30 sloping foreshore was used and the incident waves were measured at the toe of the structure using two wave gauges and the separation method of Goda and Suzuki, 1976.

C.4 Burcharth and Frigaard, 1990

Burcharth and Frigaard, 1990 performed three dimensional stability model tests with a roundhead and the adjacent trunk section for wave obliqueness from 0 to 30 degrees. Long-crested irregular waves generated from the JONSWAP spectrum using a peak enhancement factor $\gamma = 3.3$ were tested. The waves were measured with the structure in place performing no reflection analysis. The profile was initially a straight slope reshaping into a bermed profile. The model were constructed of one stone class (no core).

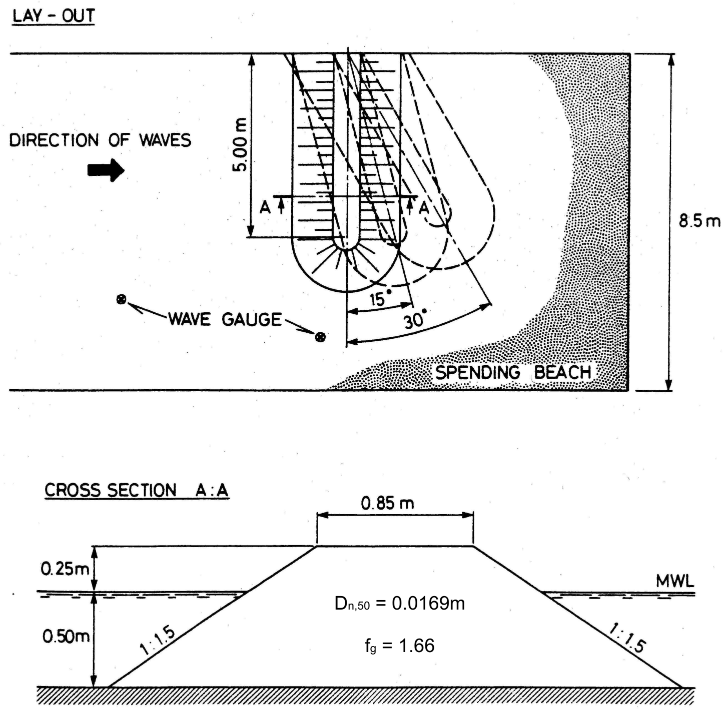

Figure C.6
Test set-up used by Burcharth and Frigaard, 1990.

C.5 Andersen and Poulsen, 1991

Andersen and Poulsen, 1991 performed a parametric study on stability of berm breakwaters and carried out approximately 100 tests. In each test they simultaneously tested two different cross-sections separated by a plate in the middle of the flume. The test programme covered the influence of wave height and steepness, berm width, berm elevation, crest freeboard and the water depth. Waves were measured approximately 27 meters from the structure, where the water depth was 0.15 m larger than at the toe of the structure and at the toe of the structure. The waves were measured by three wave gauges at the deeper part for reflection analysis and at the toe of the structure with no reflection analysis. It is not clear if it is the wave parameters at the toe or at deeper water that is given in the report, but it is assumed to be those at the toe as no reflection coefficients are given.

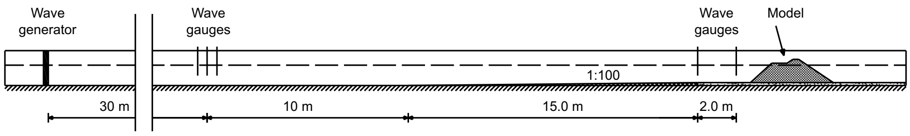

Figure C.7
Test setup used by Andersen and Poulsen, 1991.

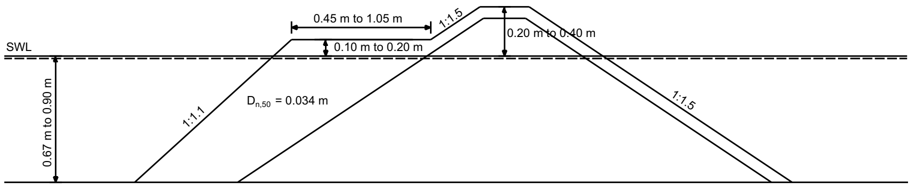

Figure C.8
Cross sections tested by Andersen and Poulsen, 1991.

C.6 Hall, 1991

Hall, 1991 investigated the influence of wave height, wave period, wave groupiness, number of waves, berm width, stone gradation factor and percentage of rounded stones on berm recession. Fig. C.9 shows the tested cross-sections.

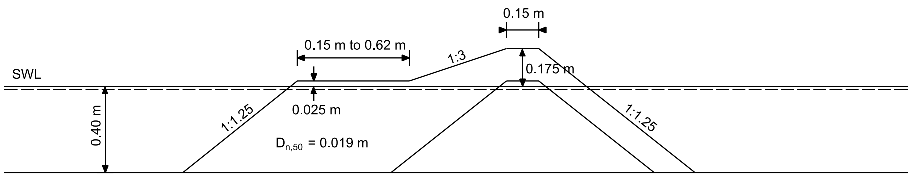

Figure C.9
Cross sections tested by Hall, 1991.

C.7 Lissev, 1993

Lissev, 1993 studied the influence of extending the core into the berm on the hydraulic response of a berm breakwater with homogeneous berm. A single cross-section was tested and front profile, overtopping, wave transmission, reflection and start of rear slope damage were measured. Lissev, 1993 concluded that the core could be extended into the berm with only little influence on the hydraulic response, if the top of the core is located 3-4 stone diameters below the final reshaped profile.

The waves were generated from the Pierson-Moskowitz spectrum and the waves were measured 8 meters from the breakwater on a flat bottom with the structure in place. Reflection analysis was performed using the method of Zelt and Skjelbreia, 1992 corresponding to the method of Mansard and Funke, 1980 as three wave gauges were used. The tests are performed with a rather large water depth to wave height ratio, so no or only very limited wave breaking is expected to have taken place. Therefore the waves measured should moreorless correspond to the waves at the toe of the structure. The overtopping volume was measured at the back of the crest.

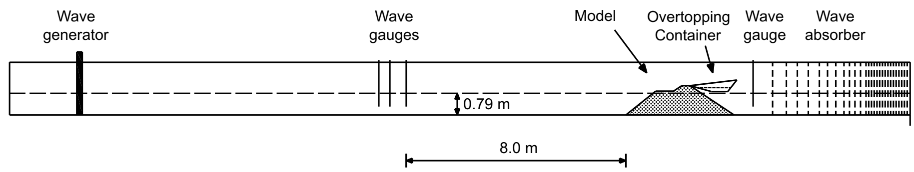

Figure C.10
Test setup used by Lissev, 1993.

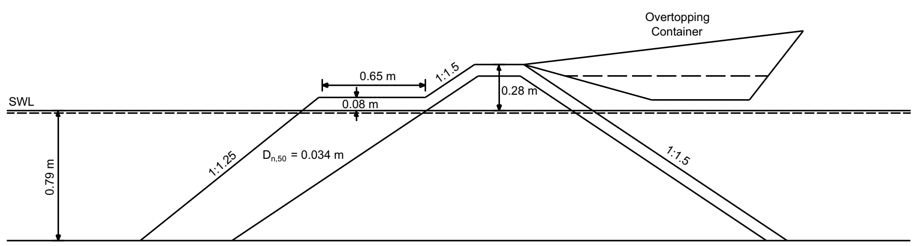

Figure C.11
Cross section tested by Lissev, 1993.

C.8 Viggosson et al., 1993

Viggosson et al., 1993 performed 3D physical model tests with the Keilisnes breakwater. The tested breakwater was a multi-layer berm breakwater constructed from five different stone classes. Stability and overtopping was measured in long-crested irregular waves. The waves were measured with the structure in place without performing reflection analysis. The overtopping was measured as shown in Fig. C.13 where the total volume of water overtopping the crest or flowing through the rather transparent crest was measured.

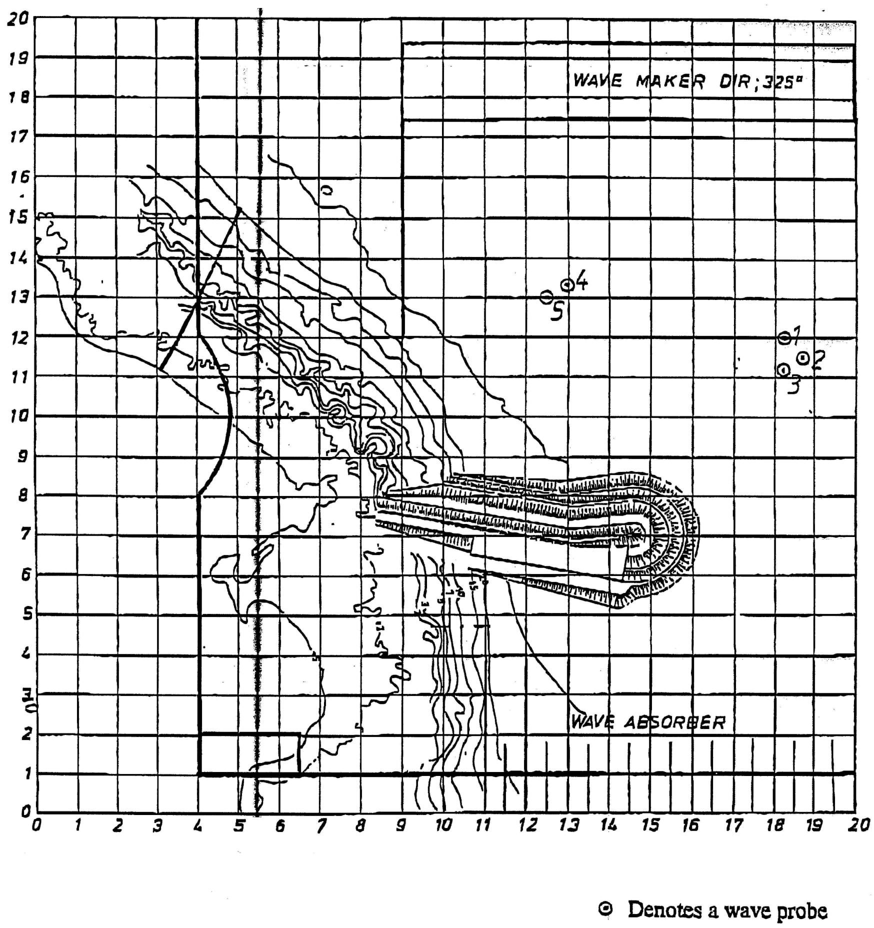

Figure C.12
Test setup used by Viggosson et al., 1993.

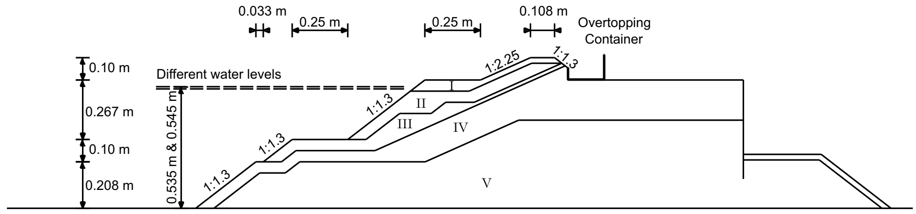

Figure C.13
Cross section tested by Viggosson et al., 1993.

C.9 Aalborg University, 1995

Aalborg University, 1995 tested stability and overtopping on a berm breakwater protecting the Heraklion Airport new runway. One of the tested cross-sections was a reshaping breakwater initially constructed as a straight slope. The berm was homogeneous and a rather impermeable core was used. The waves were generated form the JONSWAP spectrum with a peak enhancement factor $\gamma = 3.3$. The incident waves were measured with the structure in place using the separation algorithm of Mansard and Funke, 1980.

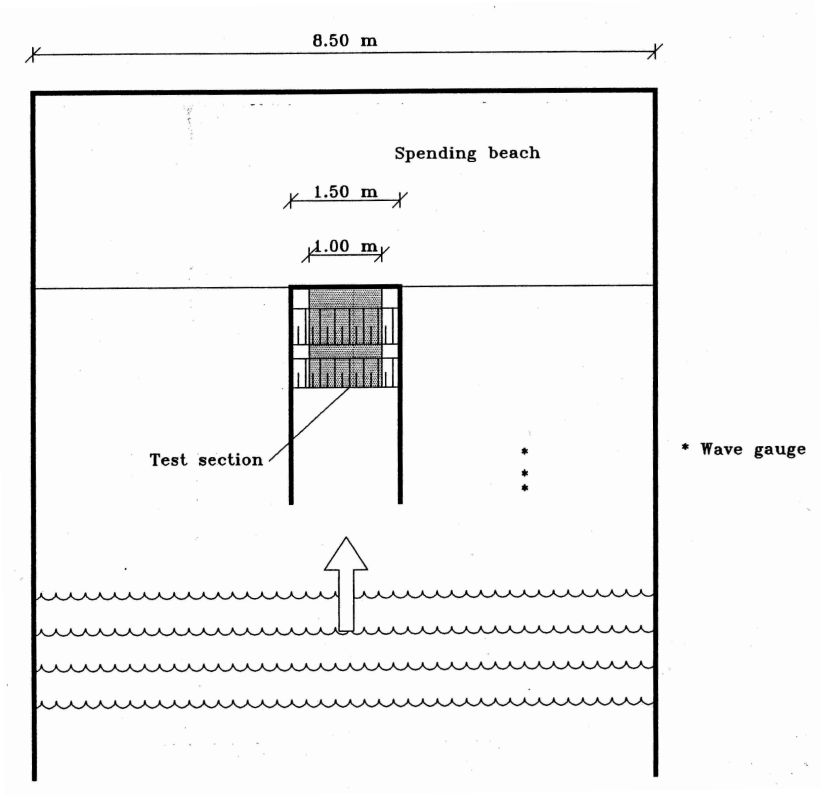

Figure C.14
Test setup used by Aalborg University, 1995.

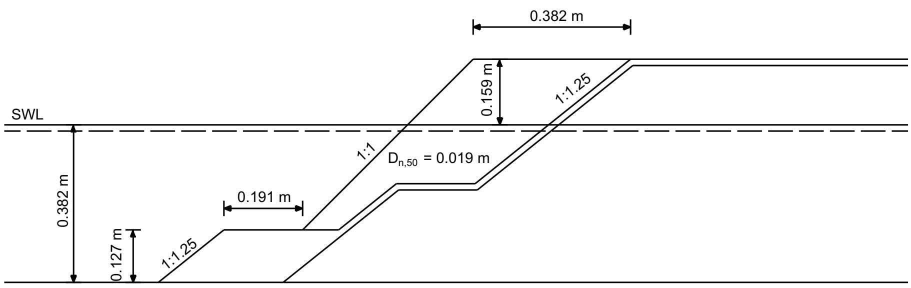

Figure C.15
Cross section tested by Aalborg University, 1995.

C.10 DHI, 1995

DHI, 1995 carried out 3D tests in a wave basin with a berm breakwater roundhead and the adjacent trunk section. The model tests focused on influence of the angle of wave attack (0-45°) on the reshaping process and stone movements at the trunk and at the roundhead of a berm breakwater with homogeneous berm. The generated waves were long-crested and the incident waves were calculated using 5 wave gauges.

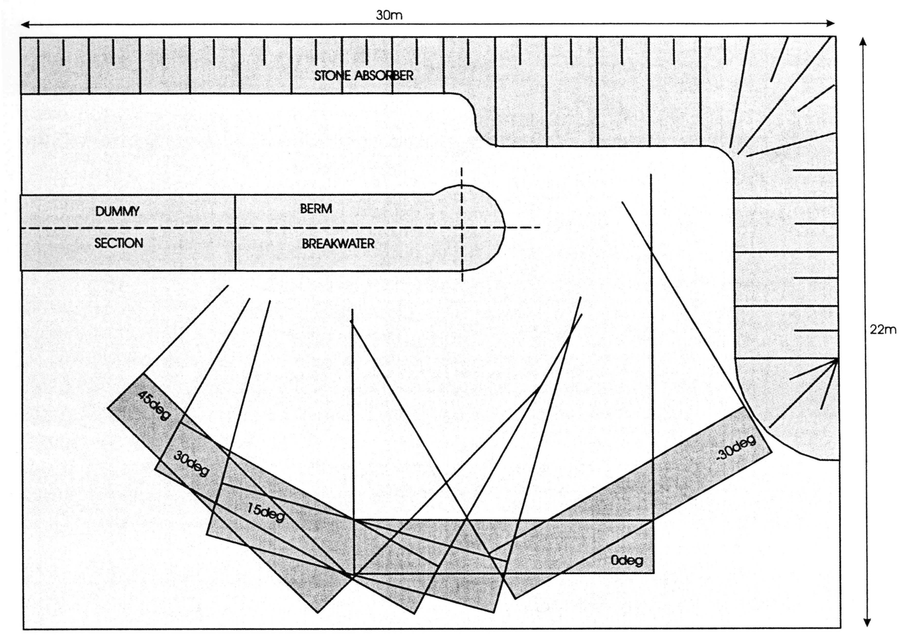

Figure C.16
Test set-up used by DHI, 1995.

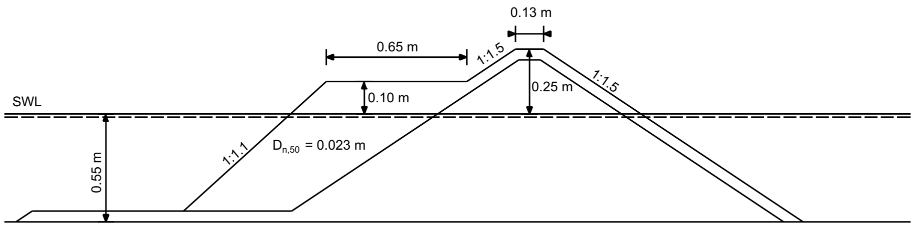

Figure C.17
Cross section tested by DHI, 1995.

C.11 DHI, 1996

DHI, 1996 studied profile reshaping and waves on the rear side generated by wave penetration through the structure and by overtopping on 12 different berm breakwater cross-sections. The test programme focus on:

- Influence of stone gradation on profile reshaping and overtopping for a berm breakwater with homogeneous berm by testing $f_g = 1.38$ and $f_g = 1.79$.
- Influence of contamination of finer material in the berm.
- Influence of separating the berm stones in two different stone classes to construct a two layer berm breakwater (Icelandic type berm breakwater).

The water depth at the toe of the breakwater was 0.25 m. The waves were measured both near the wave generator and 5 m from the structure on a water depth of 0.31 m using 5 wave gauges. Reflection analysis was performed. Shoaling and for the most severe sea states also wave breaking occured from the measurement location to the toe of the structure. Therefore the waves at the toe of the structure were calculated using the SWAN method.

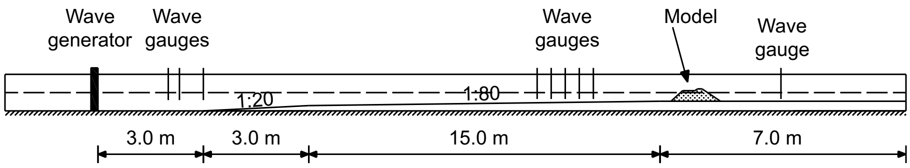

Figure C.18
Test set-up used by DHI, 1996.

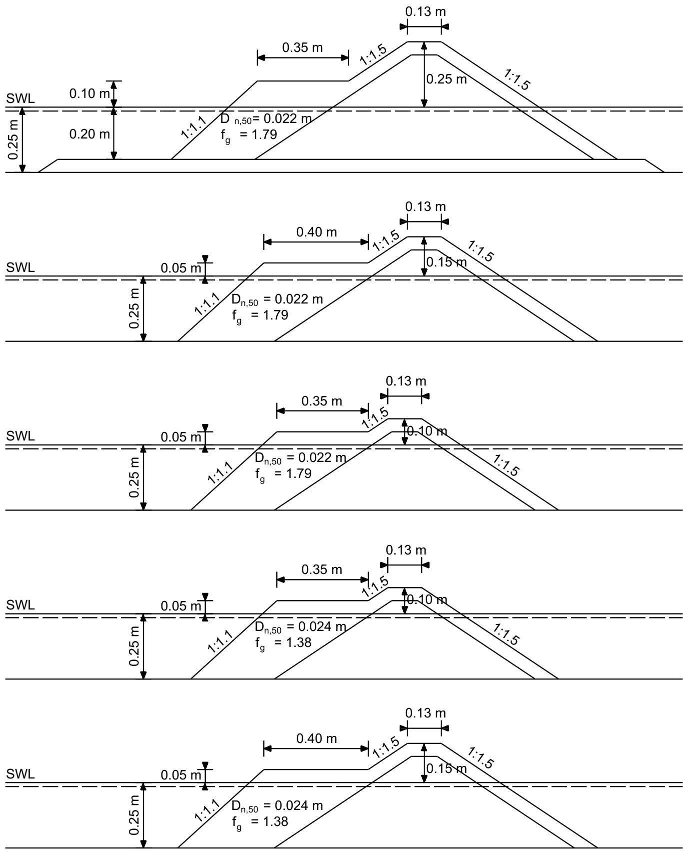

Figure C.19
Cross sections with homogeneous berm tested by DHI, 1996.

C.12 Kuhnen, 2000

Kuhnen, 2000 performed 3D model tests in a wide wave flume with the multi-layer berm breakwater used for the new Sirevåg breakwater. Kuhnen, 2000 measured scour at the breakwater head and the adjacent trunk section. Wave overtopping was measured at the crest at the trunk section, but the exact location of the ramp was not completely clear but it is assumed to be on the middle of the crest as a sketch in the report is illustrating. The waves were measured without the structure in place at the centerline of the structure (no reflection analysis).

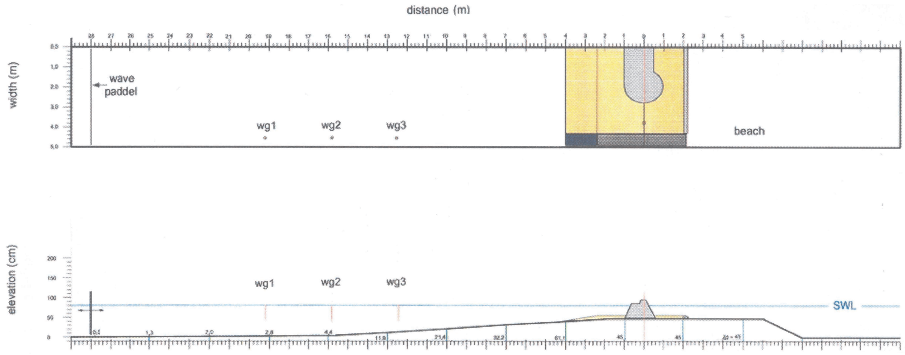

Figure C.20
Tested set-up used by Kuhnen, 2000.

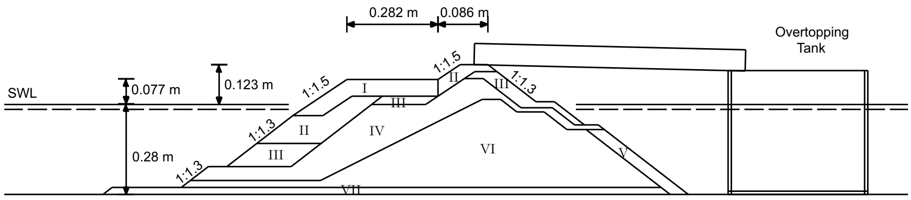

Figure C.21
Cross section tested by Kuhnen, 2000 (trunk section).

C.13 Porarinsson, 2004

13 test series with overtopping measurements were carried out in the same flume as used for the present tests. The same data analysis software package as used for the present data was used by Porarinsson, 2004. The tests can however be regarded as independent of the present data as other personal were constructing the model and performing the tests and data analysis.

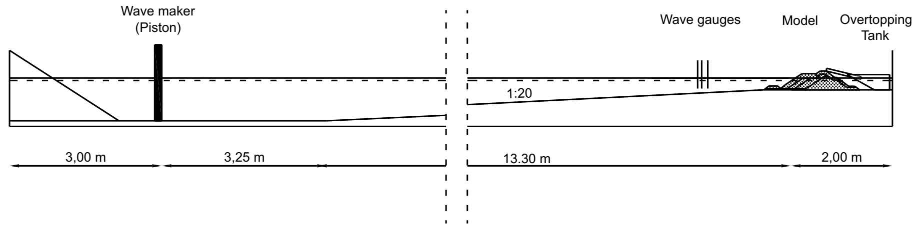

Figure C.22
Tested set-up used by Porarinsson, 2004.

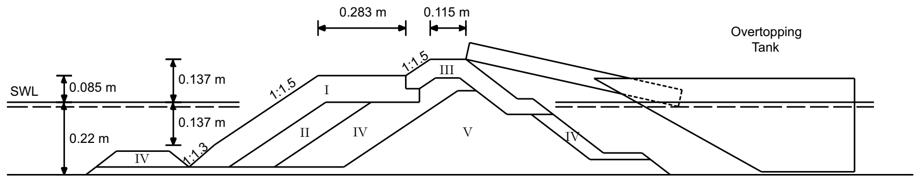

Figure C.23
Cross section tested by Porarinsson, 2004.

C.14 Summary

The available data on berm breakwater stability and overtopping used for comparison with present data is summarized in Table C.1 and C.2. For multi-layer berm breakwaters the parameters are related to the largest stone class. For approximately half of the data sets the waves were not measured st the toe of the breakwater. In these cases the waves at the structure has been calculated using the SWAN method. Not in all cases has reflection analysis been performed, but reshaping berm breakwaters give little reflection thus it is expected only to have little influence for the stability tests. However, for overtopping tests it is essential to have very accurate wave measurements at the toe of the structure due to the very non-linear overtopping process.

Study	No. stability data	No. overtopping data	R_c [m]	G_c [m]	B [m]	h_b [m]	$cot(\alpha_d)$	$cot(\alpha_u)$
Instanes, 1987	7	0	0.188	0.15	0.45	-0.035	1.5	3.0
Tørum et al., 1988	13	0	0.25	?	0.45	-0.10	1.5	3.0
Van der Meer, 1988	51	0	0.35	?	≈0.60	-0.1 to 0.1	1.5	3.0
Burcharth and Frigaard, 1990	5	0	0.25	0.85	0	-	1.5	1.5
Andersen and Poulsen, 1991	178	0	0.2-0.4	0.30	0.45-1.05	-0.2 to -0.1	1.1	1.5
Hall, 1991	108	0	0.175	0.15	0.15-0.62	-0.025	1.25	3.0
Lissev, 1993	12	11	0.28	0.30	0.65	-0.08	1.25	1.5
Viggosson et al., 1993	0	39	0.122 - 0.132	0.108	0.25	-0.032 to -0.022	1.3	2.25
Aalborg University, 1995	4	2	0.159	?	0	-	1.0	1.0
DHI, 1995	30	0	0.25	0.13	0.65	-0.1	1.1	1.5
DHI, 1996	50	0	0.10-0.25	?	0.35-0.40	-0.1 to -0.05	1.1	1.5
Kuhnen, 2000	0	3	0.136	0.086	0.282	-0.071	1.3-1.5	1.5
Porarinsson, 2004	7	13	0.137	0.115	0.283	-0.085	1.3-1.5	1.5

Table C.1

Range of parameters.

Study	s_{0m} [%]	H_{m0} [m]	N	h [m]	$D_{n,50}$ [m]	f_g	N_S	H_0T_0	$Re_D \cdot 10^{-4}$
Instanes, 1987	$7^{\star}$	$0.161^{\star}$	550-2200	0.30	0.026-0.031	1.36-1.40	2.9-3.5	61-80	3.2-3.9
Tørum et al., 1988	$2.5\text{-}6.7^{\star}$	$0.085\text{-}0.232^{\star}$	565-11400	0.50	0.029	≈ 1.5	1.6-4.4	44-139	2.6-4.4
Van der Meer, 1988	1.9-5.7	0.058-0.250	250-10000	0.80	0.026	1.5	1.4-6.0	27-296	1-9-4.0
Burcharth and Frigaard, 1990	2.0-3.8	0.10-0.20	3000	0.50	0.0169	1.66	3.6-7.2	114-380	1.7-2.4
Andersen and Poulsen, 1991	1.7-4.7	0.037-0.286	1000	0.67-0.9	0.0339	1.36	0.7-5.1	11-177	2.0-5.7
Hall, 1991	2.7-9.1	0.054-0.155	3200-4000	0.40	0.019	1.35-5.4	1.7-5.0	33-194	1.4-2.3
Lissev, 1993	4.0-4.5	0.068-0.276	1000-5000	0.79	0.0344	1.44	1.2-4.9	21-164	2.8-5.7
Viggosson et al., 1993	$0.8 - 4.1^{\star\star}$	$0.065\text{-}0.0896^{\star\star}$		≈ 0.54			1.4-2.2	34-74	1.9-2.3
Aalborg University, 1995	5.5-7.1	0.097-0.135	1000	0.38	0.0187	1.44	3.1-4.4	72-119	1.8-2.2
DHI, 1995	4.2-4.5	0.059-0.162	2000	0.55	0.023	1.80	1.5-3.9	30-121	1.8-2.8
DHI, 1996	$3.1\text{-}6.2^{\star}$	$0.074\text{-}0.127^{\star}$	2000	0.25	0.022-0.031	1.20-1.79	1.9-3.4	28-91	1.9-3.5
Kuhnen, 2000	4	0.100-0.133		0.25	0.022-0.030	1.14	2.2-2.9	59-90	2.2-3.4
Porarinsson, 2004	3.3-6.9	0.079-0.130	≈1200	0.22	0.043	1.3	1.1-1.7	18-40	3.8-4.8

Table C.2

Range of parameters. $\star$ = wave parameters calculated by SWAN. $\star\star$ = wave height reduced due to 40% reflection.