Table 1. Data of the waves and the calculated relative runup height on the experimental seawall based on the field observation.

Date	Time	① Nos. of Runp	② Nos. of Waves	③ Runup Count In	④ H_o ($H_{1/10}$) m	⑤ T ($T_{1/10}$) s	⑥ L_o ($L_1'_{10}$) m	⑦ Water Level +m	⑧ Water Depth dm	⑨ Runup Height Rm	⑩ H_o/L_o	⑪ d/L_o	⑫ R/H_o
'86. 11. 26	10	120	245	25	2.28	7.4	85.4	0.38	3.58	3.09	0.027	0.042	1.36
	11	125	233	23	2.16	7.5	87.8	0.37	3.57	3.12	0.025	0.041	1.44
	12	127	231	23	2.12	7.8	94.9	0.36	3.56	2.97	0.022	0.038	1.40
	13	117	213	21	2.58	8.9	123.6	0.32	3.52	2.99	0.021	0.028	1.16
	14	108	223	22	2.25	8.6	115.4	0.31	3.51	3.49	0.020	0.030	1.55
	15	109	210	21	2.25	9.2	132.0	0.27	3.47	2.75	0.017	0.026	1.22
11. 27	12	96	178	18	1.70	9.9	152.9	0.22	3.42	4.92	0.011	0.022	2.89
	14	89	199	20	1.32	9.5	140.8	0.15	3.35	4.68	0.009	0.024	3.55
12. 15	10	145	213	21	2.18	7.2	80.9	0.25	3.45	2.79	0.027	0.043	1.28
	11	125	210	21	2.38	6.7	70.0	0.27	3.47	3.07	0.034	0.050	1.29
	12	131	218	22	2.73	6.9	74.3	0.30	3.50	3.48	0.037	0.047	1.27
	13	126	233	23	2.61	6.7	70.0	0.35	3.55	3.09	0.037	0.050	1.18
	14	126	233	23	2.45	6.5	65.9	0.38	3.58	3.02	0.037	0.054	1.23
	15	118	245	25	2.33	6.5	65.9	0.38	3.58	3.14	0.035	0.054	1.35
12. 20	10	80	178	18	3.34	9.9	152.9	0.26	3.46	5.14	0.022	0.023	1.54
	11	92	159	16	3.66	9.2	132.0	0.26	3.46	5.29	0.028	0.026	1.44
	12	85	184	18	3.09	9.6	143.8	0.24	3.44	5.17	0.021	0.024	1.67
	13	95	164	16	3.45	10.2	162.3	0.23	3.43	5.12	0.021	0.021	1.48
	14	99	147	15	3.70	10.3	165.5	0.25	3.45	5.42	0.022	0.021	1.46
	15	89	144	14	3.22	10.3	165.5	0.28	3.48	4.93	0.019	0.021	1.53
'87. 1. 14	10	100	271	27	1.05	8.5	112.7	0.11	3.31	3.05	0.009	0.029	2.90
	11	98	253	25	1.36	8.8	120.8	0.11	3.31	3.18	0.011	0.028	2.34
	12	107	260	26	1.16	7.6	90.1	0.15	3.35	3.43	0.013	0.037	2.96
	13	107	251	25	1.14	9.1	129.2	0.18	3.38	3.03	0.009	0.026	2.66
	14	102	261	26	1.10	9.2	132.0	0.21	3.41	2.94	0.008	0.025	2.67
	15	100	233	23	1.29	9.4	137.8	0.24	3.44	3.14	0.009	0.025	2.43
2. 4	10	88	161	16	3.33	10.5	172.0	0.12	3.32	5.36	0.019	0.019	1.61
	11	82	164	16	3.06	9.6	143.8	0.07	3.27	5.46	0.021	0.023	1.78
	12	87	171	17	2.73	9.3	134.9	0.03	3.23	5.32	0.020	0.024	1.95
	13	87	161	16	2.59	9.4	137.8	0.03	3.23	5.42	0.019	0.023	2.09
	14	91	173	17	2.25	9.3	134.9	0.04	3.24	5.81	0.017	0.024	2.58
	15	80	191	19	2.46	9.5	140.8	0.03	3.23	5.21	0.017	0.023	2.12
2. 26	10	89	217	22	2.68	8.3	107.5	0.13	3.33	4.35	0.025	0.031	1.62
	11	105	212	21	2.79	8.7	118.1	0.19	3.39	4.45	0.024	0.029	1.59
	12	93	243	24	2.40	9.0	126.4	0.23	3.43	4.45	0.019	0.027	1.85
	13	89	244	24	2.36	7.9	97.4	0.28	3.48	4.26	0.024	0.036	1.80
	14	90	196	20	2.58	9.8	149.8	0.31	3.51	4.50	0.017	0.023	1.74
	15	89	199	20	3.00	10.7	178.6	0.32	3.52	4.73	0.017	0.020	1.58
2. 27	10	77	141	14	2.82	12.5	243.8	0.12	3.32	5.37	0.012	0.014	1.90
	11	75	145	15	3.03	12.9	259.6	0.16	3.36	5.12	0.012	0.013	1.69
	12	68	153	15	2.58	13.1	267.7	0.21	3.41	4.85	0.010	0.013	1.88
	13	82	159	16	2.59	12.8	255.6	0.28	3.48	4.73	0.010	0.014	1.83
	14	72	194	19	2.29	12.5	243.8	0.34	3.54	4.61	0.009	0.015	2.01
	15	83	154	15	2.65	11.9	220.9	0.35	3.55	4.32	0.012	0.016	1.63

Because of the great differences between number of recorded waves and observed runup waves to the experimental seawall, the author used the one-tenth highest wave and the same numbers of the highest wave runup, in the arrangement. For example, in the case of

the date the 20th December 1986,
time 14 o'clock

in the Table 1,

(1) the number of the observed runup waves to the experimental seawall in twenty minutes is 99.
(2) the number of the waves recorded in the wave recorder in twenty minutes is 147.
(3) calculated number of the one-tenth highest wave is 15. and the same number of highest wave runup are adopted.
(4) one-tenth highest wave height is 3.70 m.
(5) one-tenth highest wave period is 10.3 s.
(6) calculated offshore wave length L_o is 165.5 m.
(7) sea water level at that time is +0.25 m.
(8) water depth at the toe of the seawall is 3.45 m.
(9) the mean value of the highest 15 waves runup height, adopted in (3), measured from the sea water level is 5.42 m.
(10) wave steepness H_o/L_o (4)/(6)=3.70/165.5 = 0.022.
(11) relative water depth d/L_o(8)/(6)= 3.45/165.5 = 0.021.
(12) relative wave runup height R/H_o (9)/(4) = 1.46

The wave runup height is

5.42 m + 0.25 m = + 5.67 m ≑ + 5.70 m (Crown height)

Naturally, a number of runup waves overtopped the crown of the experimental seawalls, as shown in Photo 12.

4.3 Comparing the field observation data with laboratory test results

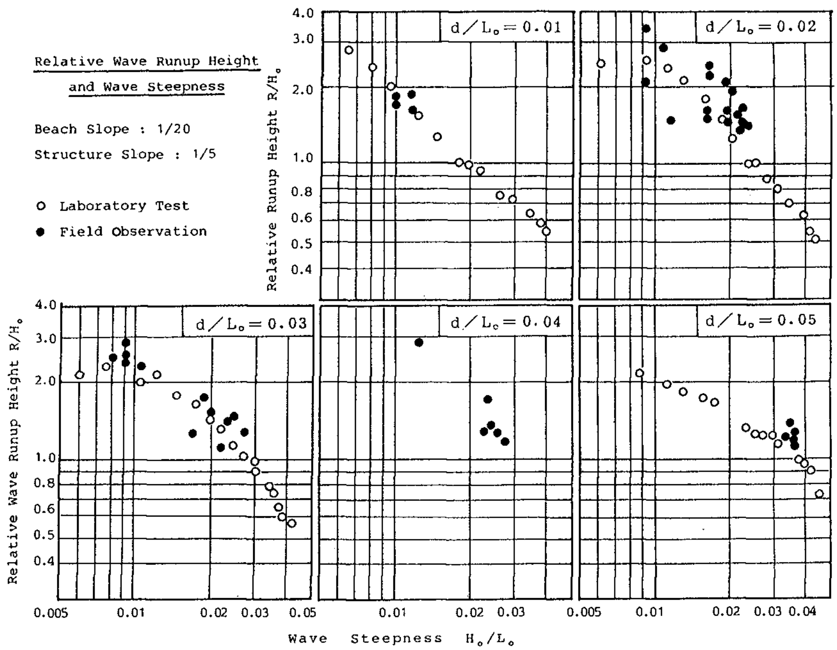

Fig.10. Comparison between the laboratory test and the field observation on the wave runup height on gentle slope seawalls.

The data of the field observation are ploted in Fig.10, together with the data of the laboratory test. Open circles are laboratory test and solid dots are field observation. At a grance, both of these data show a good agreement. The slope of the seawalls in the field are covered with armour units, but in the laboratory test all slope of model are smooth and impermeable. The waves in the laboratory test were regular waves. The wave recorder is 5 km away from the site of works, the runup waves are not the same waves as recorded by the wave recorder.

Although,there are many problems in these data and their caluculation processes, the author considers that these results on the gentle slope 1:5 seawalls are very useful for designing of the shore protection in future.

4.4 The works of fiscal 1987 was completed

Pnoto 13. The works of fiscal 1987 was completed. The left half are of fiscal 1986.

Photo 14. Heavy waves attacked the Kurobe Coast, and some of them overtopped the experimental seawalls.

On 6th November 1987, heavy winter waves attacked the Kurobe Coast, and some of them overtopped the experimental seawalls. However, the spray of sea water on the gentle slope seawalls were excessively small in comparison with that of existing vertical type seawalls with large mound of armour units.

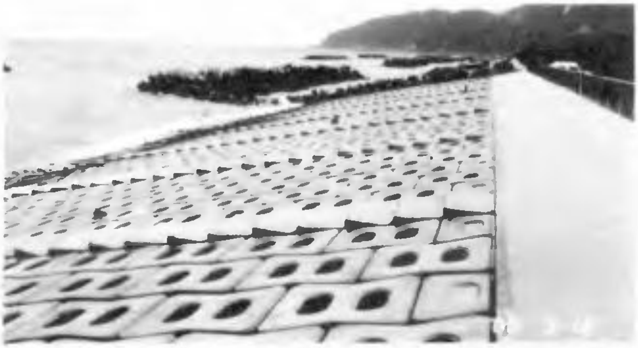

Photo 15. The latest view of the experimental seawalls.

After the construction of the experimental works, one or two winter season are over, but there has not been any trouble, damage or collapse, on the body, slope, covering blocks, foot and toe of the experimental works. At this time, these new type gentler slope seawalls are successful.

V. CONCLUSION

Many of existing vertical type seawalls have lost their foreshore, and wave dissipation works are not effective enough to prevent the overtopping of the splash and sea water mass. On the basis of the successful results and experiences obtained through the experimental works, the author proposed to reform existing vertical type seawalls to gentler slope seawalls with armour unit facing.

In conclusion, in order to maintain better coastal environment, structures on an erosive coast are desired to satisfy the following conditions:

(1) the wave reflection from the front slope should be minimized,
(2) scouring at the structure toe should be prevented as well as erosion of the foreshore,
(3) the whole structure should not collapse even if partial breakdown took place,
(4) the crown of the structures should not be too high,
(5) repairs and reinforcements should be easy, and
(6) the structure should be rather simple and not too costly.

In this view, the gentler slope seawall with rough and permeable front slope is one of the most relevant countermeasures against the beach erosion.

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CHAPTER 148

ON BERM BREAKWATERS

by
Alf Tørum[1], Steinar Næss[2],
Arne Instanes[3] and Svein Vold[4]

ABSTRACT

Two and three dimensional laboratory studies have been carried out on the stability of a berm breakwater concept. The study has to some extent been general and to some extent been connected to a project study of the stability of a berm breakwater for the fishing port of Årviksand, Norway.

INTRODUCTION

The main feature of a berm breakwater is that it has a rather thick cover layer of stones, relatively much smaller than on a conventional breakwater with one or two layers of cover blocks. The berm breakwater has been adopted several places as an economic solution when large cover blocks of natural stones are not available. It might also be an economical solution even when large cover blocks for a conventional breakwater are available.

The berm breakwater concept has become of interest in Norway in connection with plans for an extension of a breakwater at the Årviksand fishing port. Fig. 1 shows the layout of the harbour. The breakwater will be extended about 90 m into a maximum water depth of approxi-

1 Head of Research/Professor. Norwegian Institute of Technology/ Norwegian Hydrotechnical Laboratory, 7034 Trondheim, Norway.
2 Chief Engineer. Norwegian Coast Directorate. P.O. Box 8158 - Dep 0033 OSLO 1, Norway.
3 Research Engineer, Division of Structural Engineering, Norwegian Institute of Technology, 7034 Trondheim, Norway.
4 Research Engineer, Norwegian Hydrotechnical Laboratory, 7034 Trondheim, Norway.

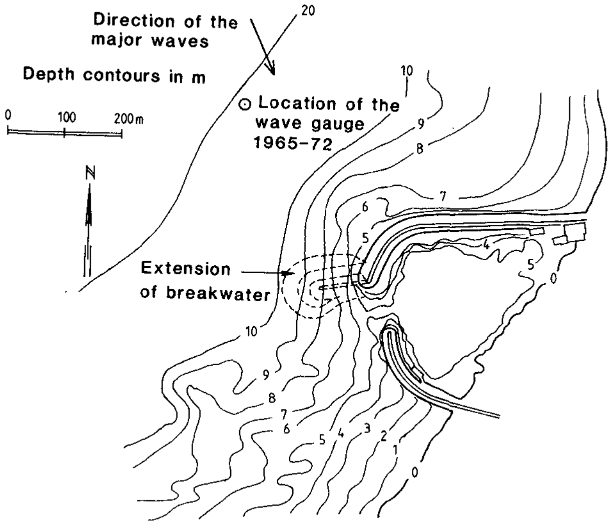


Fig. 1. Årviksand fishing port.

mately 13 m. The wave climate at the breakwater site has been evaluated from wave measurements during the time period 1965-1972 at 20 m water depth outside the harbour and from hindcast wave data in deep water from the time period 1955-1985 and refraction analysis. Based on the wave measurement and a Weibull distribution formulation the 50 and 100 year significant wave heights were estimated to be approximately 6.4 m and 7.2 m respectively. From the hindcast data and the refraction analysis the 100 year wave height was estimated to be approximately 6.2 m.

The differences in the estimated 100 year wave height reflects the uncertainty on the estimated wave climate which always is present at any harbour location on an open coast.

Based on a Weibull distribution fitting of the measured daily maxima, a simulation study has been performed based on a procedure described in [5]. The Weibull distribution which fits, by the method of moments, the data best has been used as the "parent" distribution. By a Monte-Carlo procedure daily maxima for seven-years of observation have been simulated one hundred times. Fig. 2 shows the probability density function for 50 year occurrence of significant wave height for water depth 20 m. The Weibull distribution parameter for the measured daily maxima were: Shape parameter 0.77, scaling parameter 0.660 m and location parameter 0.057 m. The shape parameter is rather low for this location and explains partly the large scatter of the estimated 50 year significant wave height through the simulation study.

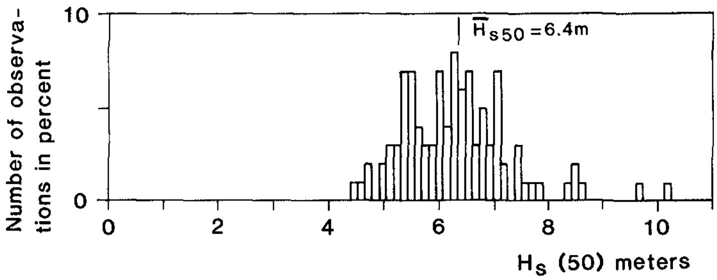


Fig. 2. Results of wave statistics simulation.

The uncertainty in the wave climate favours a breakwater design that is not too sensitive to the wave height with respect to stability. The berm breakwater is a concept that is of interest in this respect.

The stability results reported in this paper are partly from a student thesis work [1], to some extent related to Årviksand harbour, partly from a general investigation of the stability of berm breakwaters and partly from a project study for Årviksand harbour.

INTRODUCTORY TESTS ON THE STABILITY OF THE BERM BREAKWATER FOR ÅRVIKSAND HARBOUR

The existing north breakwater at Årviksand is built as a conventional breakwater with one layer of cover blocks of natural stones. The average block weight on the outer most exposed part is 10 tons. Some introductory tests showed that if the breakwater was extended to a maximum water depth of 12 m, a breakwater with 10 tons cover blocks will be stable for a significant wave height of 4.5 - 5.0 m. The estimated necessary block weight for a conventional breakwater to stand wave heights of Hs = 7.2 m would be 25 - 30 tons. Flume tests were then carried out on a berm breakwater design as shown in Fig. 3. One test was also carried out for a design shown on Fig. 4.

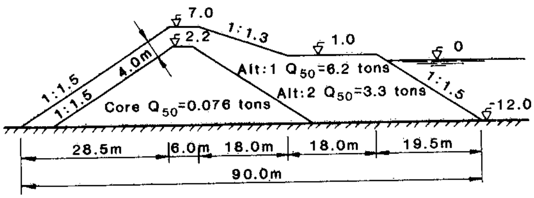


Fig. 3. Berm breakwater section - Introductory tests.

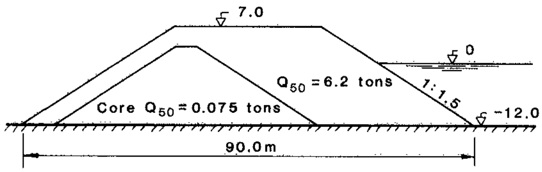


Fig. 4. Berm breakwater section - Introductory tests.

The model scale was 1:40. Two block weights in the berm were used: 1 average 3.3 tons, range 1 - 6 tons. 2 average 6.2 tons, range 1 - 14 tons.

Fig. 5 shows the flume with the breakwater model. The bottom configuration in front of the breakwater corresponded to the bottom configuration in front of the planned extension of the breakwater at Årviksand fishing port.

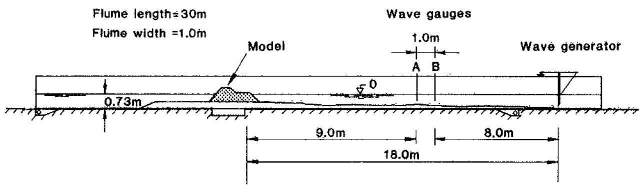


Fig. 5. Test flume.

During the tests two wave gauges were used. The water depth at wave gauge A corresponded to 20.8 m or approximately the water depth at the location of the wave gauge outside Årviksand in the time period 1965 - 1972.

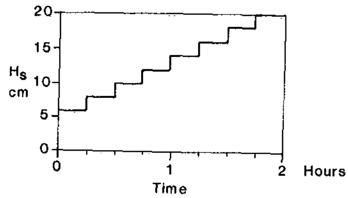


Fig. 6. Hs versus time.

The test programme used during the tests is shown in Fig. 6. The evolution with time of the significant wave height corresponds to a typical evolution during a heavy storm on the Norwegian coast. The wave spectrum was narrow with a peak period of 11.4 - 12 sec. The peak period was the same for all significant wave heights. The incoming and reflected waves were found by a procedure described by Goda and Suzuki [2]. It should be noted that for the highest significant waves the waves were non-Gausian. For exampel were the skewness 0.78 and the kurtosis 3.82 for a significant wave height of 8.3 m. The highest waves would then break before they came to the breakwater.

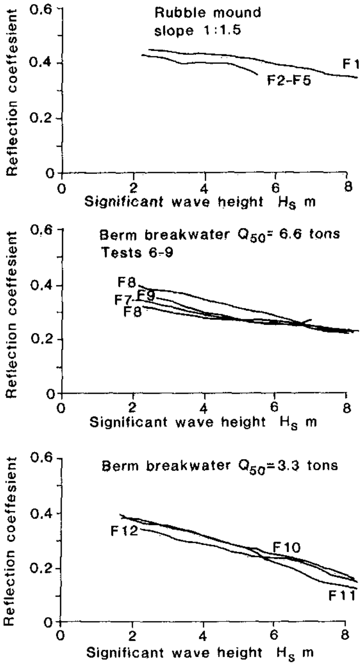
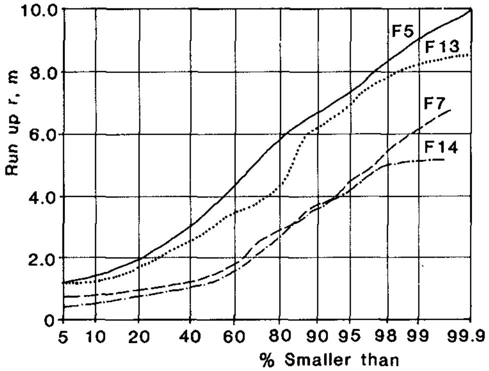


Fig. 7. Reflection coefficients.

The obtained reflection coefficients as a function of the significant wave height is shown in Fig. 7.

Run-up on the breakwater was also measured. Fig. 8 shows a run-up distribution. The run-up r is defined as the maximum run-up of each individual wave on the slope or within the breakwater.



————	Hs = 5.3 m	Rubble mound	10.3 tons	Test F5
······	Hs = 5.3 m	Berm breakwater*	6.2 tons	Test F13
— — —	Hs = 5.3 m	Berm breakwater	6.2 tons	Test F7
—·—·—	Hs = 5.3 m	Berm breakwater	3.3 tons	Test F14

* "Full" berm, Fig. 4

Fig. 8. Run-up

The run-up on the berm breakwater is much less than on the conventional breakwater. It is also seen that the run-up on the "full" berm breakwater, Fig. 4, is not much less than on the conventional breakwater. Hence it is concluded that a low berm is very efficient from the run-up point of view.

Figs. 9 shows profiles after the tests for the maximum significant wave heights 8.60 m and 8.32 m respectively were completed.