CHAPTER 148

ON BERM BREAKWATERS

by
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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-

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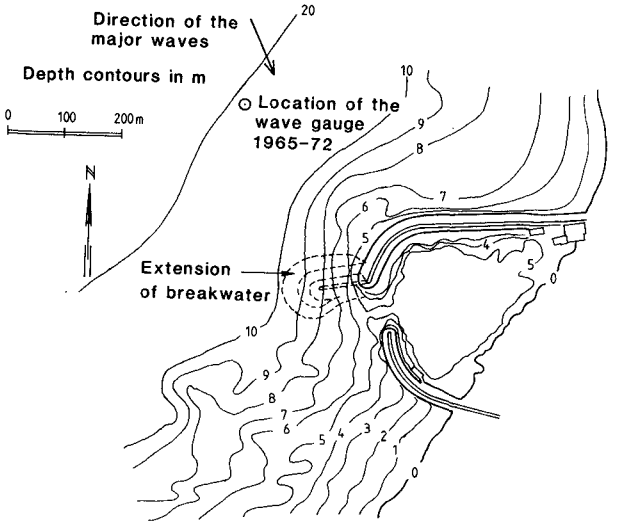


Fig. 1. Årviksand fishing port.

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.

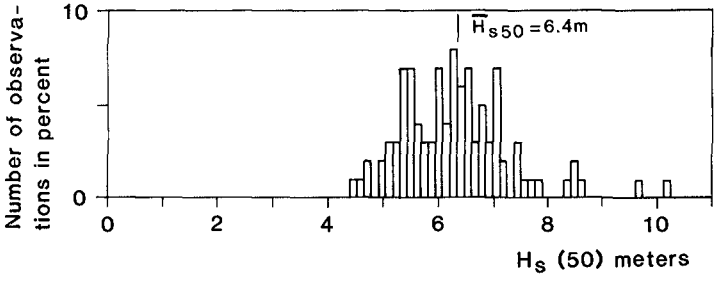


Fig. 2. Results of wave statistics simulation.

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.

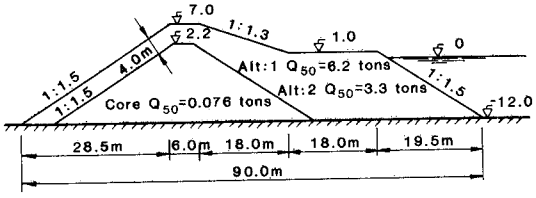


Fig. 3. Berm breakwater section - Introductory tests.

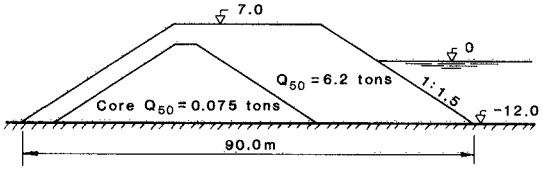


Fig. 4. Berm breakwater section - Introductory tests.

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.

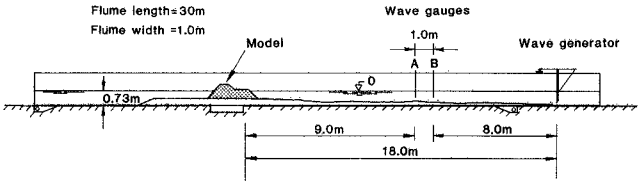


Fig. 5. Test flume.

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.

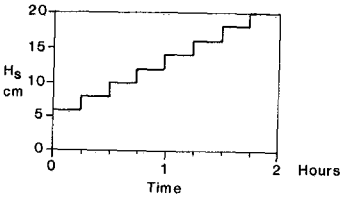


Fig. 6. Hs versus time.

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.

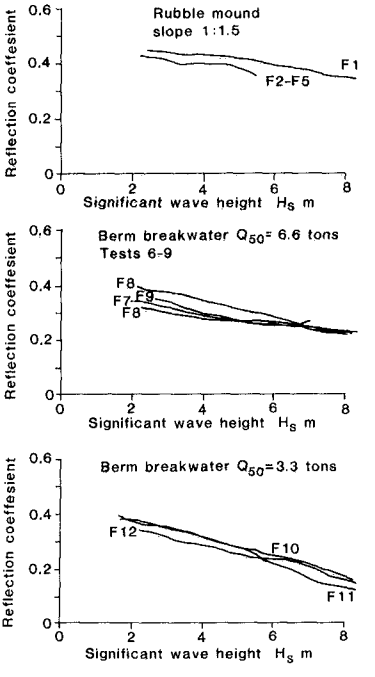
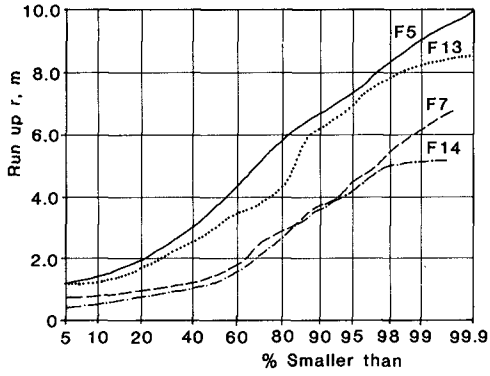


Fig. 7. Reflection coefficients.

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.



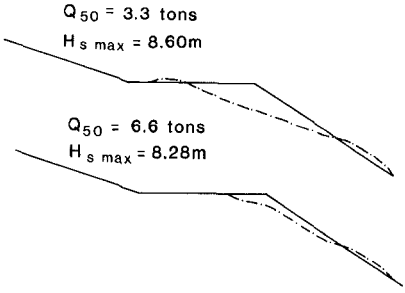
———	Hs = 5.3 m	Rubble mound	10.3 tons	Test F5
·····	Hs = 5.3 m	Berm breakwater*	6.2 tons	Test F13
— — —	Hs = 5.3 m	Berm breakwater	6.2 tons	Test F7
—·—·—	Hs = 5.3 m	Berm breakwater	3.3 tons	Test F14

* "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.



——— At the start of the test
·-·-·- After completion of the tests

Fig. 9. Cross-section

The results of these introductory tests indicated that a berm breakwater could be build with an average block weight in the range 3.3 - 6.6 tons. The berm breakwater showed also less sensitivity to the variation in the significant wave height than the conventional breakwater. There was not a sudden collapse as normally is the case for a conventional breakwater. This is very useful for a location where there is a significant uncertainty on the wave climate. However, it was felt that there was a need for a more general flume study on the stability of the berm breakwater concept.

GENERAL TEST SERIES

The objective of these tests was to investigate more closely how the berm evolved with the wave height, wave spectrum and duration of the "storm". The general test series were conducted in the same flume as shown in Fig. 5 in scale 1:40. The tested cross-section is shown in Fig. 10. Two berm widths were tested: 30 and 45 cm (12 and 18 m).

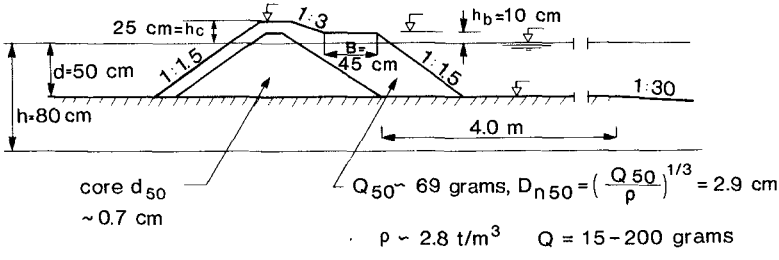


Fig. 10. Berm breakwater - General tests.

The "peak" period Tp corresponded to 12 and 16 sec. The spectrum corresponded to a JONSWAP spectrum. Two values of the peakedness parameter γ = 1.0 and γ = 7.0 were used. The wave height was varied in steps in different ways.

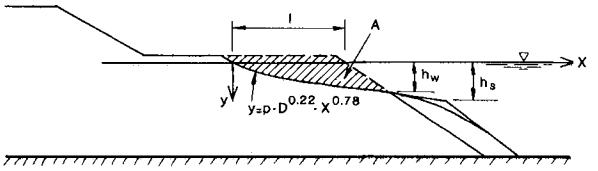


Fig. 11. Definitions.

The change of the berm is essential for the consideration of the stability of the breakwater. This change has been quantified by the number S introduced by van der Meer [3] as damage to a conventional breakwater. With reference to Fig. 11, S is defined as;

$$S = A/(D_{n_{50}})^2$$

where:

A = cross sectional area removed from the berm

$$D_{n_{50}} = \left[\frac{Q_{50}}{\rho_s}\right]^{1/3}$$

Q_{50} = 50 % of the stones has a weight larger than Q_{50}

ρ_s = spesific density of the stone material

The recession of the berm, l, see Fig. 11, has been non-dimensionalized as

$$l/D_{n_{50}}$$

If $l/Dn_{50} > B/Dn_{50}$, where B = berm width, inacceptable damage may occur.

In conventional breakwater design there is a consideration on static-stability. This means that the requirement is that the stones should not move at all. Sand and gravel beaches may on the other hand be dynamically stable. That is the individual grains may move heavily, but the profile does not in the long term sence change. This is also true for a berm breakwater. The profile is then dynamically stable. This means that the individual stones may move up and down the slope,

but the profile is stable in the long term sence. This applies especially to a two-dimensional profile, when the waves are coming normal to the breakwater. If the wave attack is oblique to the breakwater, stones can also move along the breakwater front. In this case the breakwater may not be dynamic stable.

The present program included basically two test series: One series, when the spectrum shape and peak period were varied, had a time evolution of the significant wave height, as shown in Fig. 12, corresponding to a typical evolution of an intense storm on the Norwegian open coast. In the other test series the duration of the tests for each variation of the wave height was much longer. The tests were carried out with no "repair" work on the breakwater after each wave height step had been completed. The program included one test, A, with a berm width of 30 cm (12 m). The remaining tests, B, were all for a berm width of 45 cm (18 m).

	Prototype		Model	
A	Tp = 12 sec	γ = 1.0	Tp = 1.9 sec	γ = 1.0
B 1-2	Tp = 12 sec	γ = 1.0	Tp = 1.9 sec	γ = 1.0
B 3	Tp = 12 sec	γ = 7.0	Tp = 1.9 sec	γ = 7.0
B 4	Tp = 16 sec	γ = 1.0	Tp = 2.53 sec	γ = 1.0
B 5	Tp = 12 sec	γ = 1.0	Tp = 1.9 sec	γ = 1.0

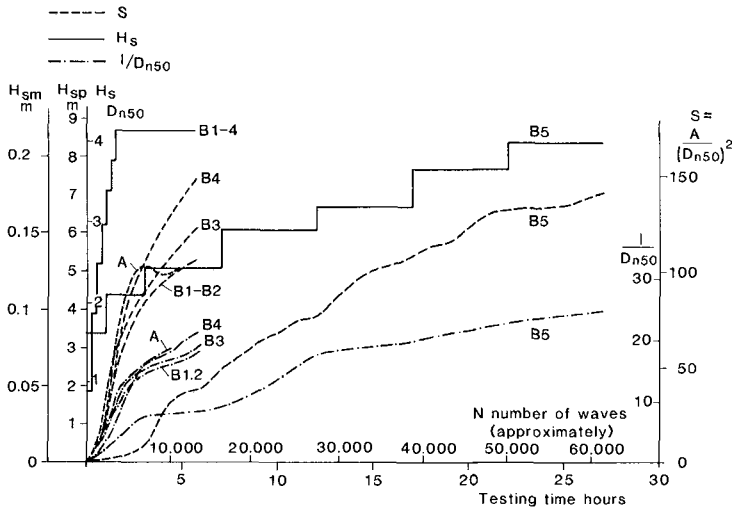


Fig. 12. General tests - Results.

The evolution with time and wave height of S and $1/Dn_{50}$ is also shown in Fig. 12.

It is seen that the value of S is depending on the peak period as well as on the width of the wave spectrum. It has also previously been found for exampel by Carstens et al. [4] that a narrow spectrum give a harder attack on a conventional rubble mound breakwater than a wide spectrum provided that the peak period is the same and provided that the significant wave height is the same. Van der Meer [3] found that the significant period was the average zero crossing period T_{02}. According to van der Meer [3] damage to a breakwater will be the same provided that the significant wave height is the same and the average zero upcrossing period is the same, irrespective of the spectrum shape.

However, on a conventional breakwater the damage is depending also on the Irribarren number

$$\xi = tg\alpha/(2\pi H_s/g\ T_{02}^2)$$

where α is the breakwater slope and $T_{02} = \sqrt{m_0/m_2}$, where mn is the nth moment of the spectrum. A berm breakwater has not a well defined slope. However, the results presented in Fig. 12 indicate that the "damage" is depending on the peak period Tp.

In the Figs. 13 and 14 are shown typical evolution of the profiles for two test series. B2 is for the shorter duration tests and B5 is for the longer duration tests.

From Fig. 13 and Fig. 14 is seen that there seems to be a significant point located at a distance hw as shown in Fig. 11. The intersection between the profile and the original profile seems to be more or less at this point, irrespective of the scour of the slope.

Van der Meer [3] has investigated many of the parameters of importance for the evolution of the slope. He also gives an equation for the profile below the water line as indicated in Fig. 11. With the local origo given as the intersection between the still water line and the profile the profile is given by

$$y = p \cdot D^{0.22} \cdot x^{0.78} \qquad (1)$$

It is not clearly stated by van der Meer which diameter, D, whould be used, but it is assumed that it is the diameter Dn_{50}. The coefficient p is primarily a function of the wave steepness, $2\pi Hs/(gT_{02}^2)$. For the present test parameter values van der Meer´s results give $p \approx 0.5$. With this value of p we have shown in Figs. 13 and 14 the profile as given by equation (1). For some reason there is an apparent difference between the profiles as obtained in the present study and the study by van der Meer [3]. Van der Meer carried out most of his tests in

relatively deeper water than used in the present study. However, van der Meer also carried out some tests with shallower water and states that there should not be any significant effects of the water depth, h, if h/Hs > 2.2, which is the case in the present study.

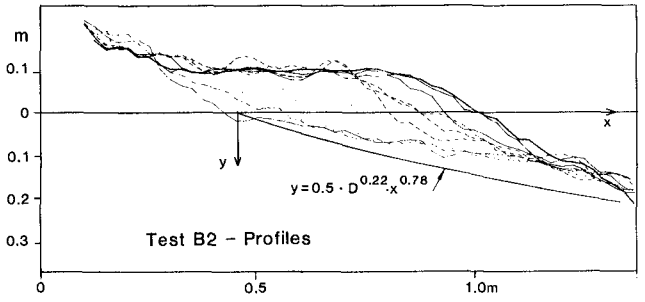


Fig. 13. Profile evolution.

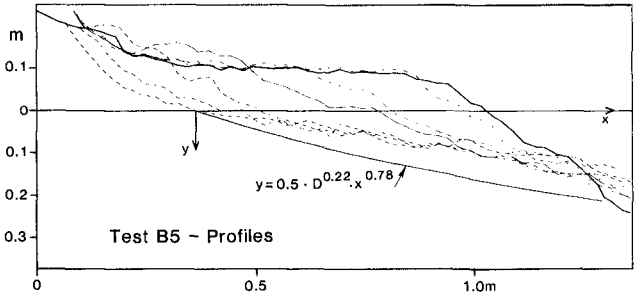


Fig. 14. Profile evolution.

There is obviously a time effect in the development of a dynamic stable profile. If we for example take the evolution with time of the wave height for the tests B1 - B4 (this evolution could be a typical evolution for the Norwegian coast) the value of l/Dn_{50} is equal to the berm width value B/Dn_{50} = 15.5 after approximately 6000 waves. The maximum significant wave height is then corresponding to $Hs/\Delta Dn_{50} \approx 4.0$, where $\Delta = \rho s/\rho w - 1$, ρw = specific density of the water.

For the long duration tests, B5, we have l/Dn_{50} = 15.5 after approximately 11 hours testing time or after approximately 25000 waves. The maximum significant wave height is then $Hs/\Delta Dn_{50} \approx 3.0$. This means that the design of the breakwater should be for $Hs/\Delta Dn_{50}$ somewhere between 3 and 4.

The tests with the 12 m wide berm (A-test) showed a berm

recession evolution about the same as for the tests with the wider berm (B-tests). However, due to the narrower berm, the core material finally became visible. This did not occur for any of the B-tests.

THREE-DIMENSIONAL STABILITY TESTS FOR THE BREAKWATER AT THE FISHING PORT OF ÅRVIKSAND

Based on the previous studies, the design shown in Fig. 15 was tested in a three-dimensional model of Årviksand fishing port. The model had a length scale of 1:60 and with a layout as shown in Fig. 1. The tests were run with high water + 3.0 m. The waves referred to were measured at a location corresponding to the location of the wave gauge during the field measurements in the time period 1965 - 1972. The peak period for the waves was Tp = 14 sec and the wave spectrum was approximately a JONSWAP spectrum with γ = 7.0. The time variation of the significant wave height was as shown in Fig. 16.

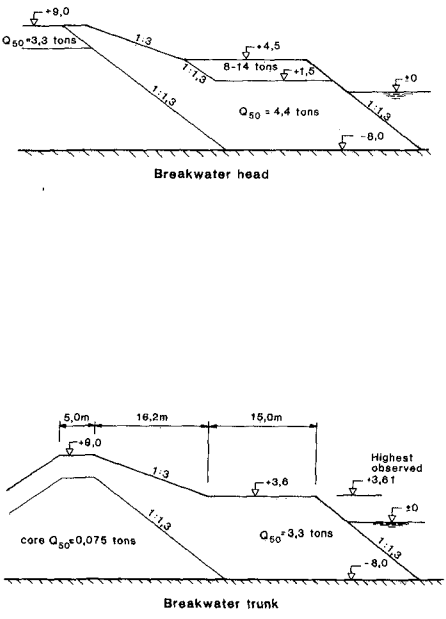


Fig. 15. Tested design for the Årviksand fishing port.

The first tests were carried out with a crest height + 9.0 m, or 6 m above the still water level.

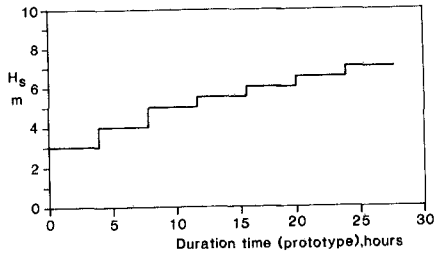


Fig. 16.

Fig. 17 shows the contours of the breakwater after 3.5 hours of testing time, or approximately 10.000 waves, with a maximum significant wave height of 7.0 m. It is seen that there is a damage on the back side of the breakwater due to overtopping at the middle part of the breakwater. The location of the damage is due to a consentration of the waves due to refraction towards the middel part of the breakwater.

The wave crest was then raised by 1.0 m to 10.0 m, or 7 m above the still water line. The breakwater was then tested for the same wave program, Fig. 16, as the previous design. Fig. 18 shows the contour map for the tested breakwater after 3.5 hours testing time. Although there were motions of stones up and down the breakwater slope as well as along the breakwater, mainly landwards, there was not any major damage to the breakwater. Some stones even moved over the breakwater crest.

Since storm duration is essential for consideration of the dynamic stability of the berm breakwater it is essential to consider statistically all the major storms the breakwater will encounter during its expected life time. The water level variation has also to be considered. The result of such a "fatique" study is not included in this paper.

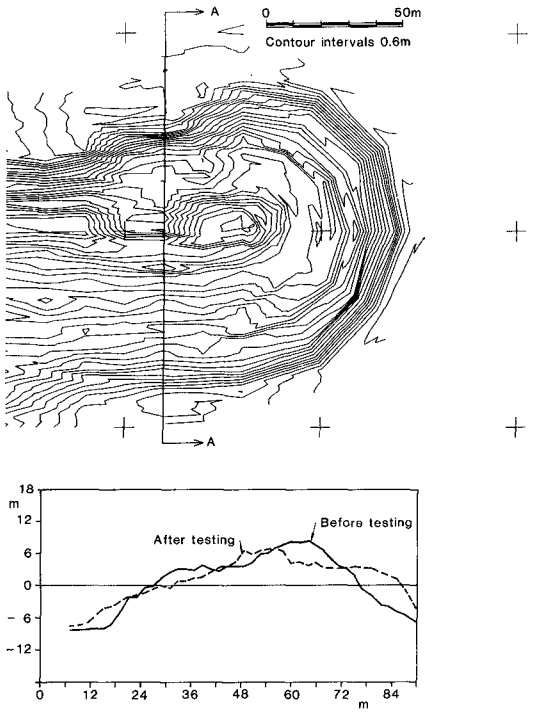


Fig. 17. Section A-A. Crest height + 9.0 m.

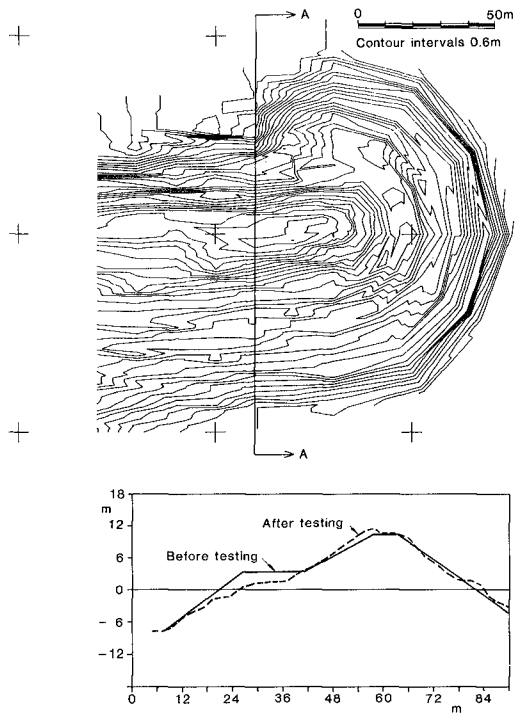


Fig. 18. Section A-A. Crest height + 10.0 m.

CONCLUSION

Based on the present study it is preliminary concluded that a berm breakwater should be designed for $Hs/\Delta Dn_{50}$ = 3 - 4, provided that the water depth ratio h/Hs > 2.5.

A berm breakwater should be subjected to a "fatique" analysis. It is essential that a three-dimensional study is conducted on the stability of berm breakwaters.

ACKNOWLEDGEMENT

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