

STABILITY TESTS OF THE EUROPOORT BREAKWATER

H. BERGE and A. TRÆTTEBERG

River and Harbour Research Laboratory at the Technical University of Norway,
Trondheim, Norway

Summary.

This paper deals with model tests conducted at the Delft Hydraulics Laboratory (DHL) and the River and Harbour Laboratory at the Technical University of Norway (RHL) for the design of the Europoort Breakwater.

A series of tests with regular waves was conducted at the DHL from which the design of the breakwater was decided. The chosen design was tested with irregular waves based on in situ observations. Wave spectra, wave height distributions and the joint distribution of wave height and period were specified. These tests were conducted at the RHL, and some tests were repeated at the DHL.

It has been commonly assumed that the destructive effect of a train of regular waves on a breakwater in model is equal to the effect of irregular waves with a significant wave height corresponding to the height of the regular waves.

* The tests showed that for this particular breakwater the irregular waves represented a more severe wave attack than the regular waves.

INTRODUCTION

This paper deals with stability tests in model of the Euro-poort Breakwater conducted at the Delft Hydraulics Laboratory (DHL) and the River and Harbour Laboratory at the Technical University of Norway (RHL). The tests were run both with regular and irregular waves.

Fig. 1 shows the outer part of Europoort. An 8 km long breakwater extending from the south will protect new industrial areas with adjoining harbour basins. The dry land will be separated from the breakwater with a channel. This permits a considerable amount of overtopping, and the breakwater has been designed with a very low crest. Fig. 2 shows typical cross sections for the deeper and for the more shallow parts of the breakwater.

The sea bed consists of sand with a low stability against erosion and a low bearing capacity. The jetty is therefore constructed with a wide fill with flat slopes. The jetty is protected with cubical blocks in two layers placed pell mell. The design wave height is 8,5 metres.

****** The model tests consisted of tests with regular waves in order to determine the general design of the breakwater and the necessary block weight. These tests were conducted at the DHL.

DHL had planned to check the results in a wave basin equipped with a new wave generator which could produce irregular waves, but it became apparent that the new equipment would not be in operation early enough to finish the tests before the deadline for the investigation.

At RHL equipment for producing irregular waves has been in operation since 1964, and RHL was asked to conduct these final tests.

This paper mainly deals with the tests which offer a possibility for comparison of results obtained with the use of regular

and irregular waves, respectively.

** TEST WITH REGULAR WAVES

The tests were performed in scale 1:60 with a cross section as shown in Fig. 3. (All elevations given in this paper are referred to New Amsterdam Ordnance Datum (NAP)). The stability was investigated with cubes with different dimensions and specific densities. For every particular cube dimension the average wave period and water level were varied independently.

By way of example Fig. 4 shows the results of a series of tests. The damage is described in qualitative terms according * to a system of certain standard criteria used at the DHL. The system not only takes into account the number of blocks which are removed from the armour, but also from where in the armour the blocks are removed. Thus the damage is rated higher if a number of blocks are removed from a concentrated area on the breakwater than if they are removed from different places more evenly distributed over the whole armour surface.

From each series of tests the test giving the minimum stability, at a damage between "none" and "slight" was used to calculate the stability number, $K/f(\alpha)$ from the formula

$$G = \frac{K}{f(\alpha)} \frac{\gamma_b H^3 d}{\left(\frac{\gamma_b}{\gamma_w} - 1\right)^3}$$

The computed maximum values of $K/f(\alpha)$ (minimum stability) for each series of tests are listed in the table.

G tons	γ_b t/m ³	H _d m	H _{destr} /H _d	K/F (α)
27,5	2,2	5,5	1,68	11,3·10 ⁻²
21,5	2,2	5,4	1,82	9,4·10 ⁻²
23,0	2,6	6,9	-	9,7·10 ⁻²
20,3	2,7	6,8	1,73	10,4·10 ⁻²
20,3	2,7	6,8	-	10,4·10 ⁻²
13,0	2,8	5,8	1,72	12,0·10 ⁻²
8,0	2,8	5,5	1,59	9,0·10 ⁻²
25,0	2,8	7,3	1,70	11,8·10 ⁻²

Average values: $K/f(\alpha) = 10,5 \cdot 10^{-2} \pm 11\%$

$$H_{destr}/H_d = 1,7 \pm 1,5\%$$

According to these values the design block weight, G = 43 tons with a specific weight of 2,65 t/m³, was selected for the design wave of 8,5 metres.

Also some tests with more shallow and less exposed parts of the breakwater were carried out in regular waves. In Fig. 5 is shown the respective average K/f(α) values for all cross sections concerned. Along the shallow part of the breakwaterscours scour is expected to occur in front of the breakwater. The graph shows the results under the condition of the original horizontal sea bed untouched and under the condition that scour has taken place. In the case of scour the models were tested with a 1:4 sloping bottom * in front of the structure from the original level of -7 and -10 metres down to -10 and -14 metres respectively.

TESTS WITH IRREGULAR WAVES

Cross sections.

The breakwater cross sections in Fig. 2 are the ones selected on basis of the tests in regular waves, but for minor modifications as the result of the tests with irregular waves.

At RHL a number of 4 cross sections were tested in irregular waves, cross sections at depths -20, -15.5, -12 and -10 metres. The shift from the deep water to the shallow water design will be at depth -12 metres. At depths less than approximately -5 metres the breakwater is constructed entirely with sand.

Tests at RHL.

Test equipment. The RHL wave channel is shown in Fig. 6. The waves are generated by a paddle moved by two oil hydraulic pistons. The paddle movement is controlled by a servo system with a voltage reference input and position and velocity feedback. For generating irregular waves the input signal is synthesized on magnetic tape from a white noise generator connected to electronic filters. With the known transfer function of the wave channel the filters are adjusted to give a wanted wave spectrum in the channel.

The wave generator works within a range of wave periods of 0.5 - 5 sec. and can produce waves with a maximum height of approximately 0.5 metres.

The wave generating system also includes a fan capable of producing wind with a maximum velocity of 10 m/sec.

****** Waves. At the site, waves have been recorded continuously a considerable period, and a well specified wave programme could be put forward as a basis for the tests. These specifications consisted of wave spectra, wave height distributions and joint distributions of wave height and period.

Fig. 7 shows the two main types of spectra used in the model tests, a Neumann spectrum and a narrower one.

The spectra were run with three different peak frequencies corresponding to wave periods of 10, 12 and 14 second in the prototype.

In Fig. 8 the dimensionless wave height distributions of model and prototype are shown, while Fig. 9 shows a diagram which expresses the joint distribution of wave height and period. The H-T correlation is derived from a wave record by plotting the ratio of the mean apparent wave period, T_i , according to the zero upcrossing convention, in intervals of $H/H_E = 0,5$ and the mean apparent period of all waves, T_m , against the wave height parameter H/H_E . The two curves envelope H-T correlations of waves outside Europoort, and the plotted values are examples of H-T correlations of the model waves. All wave data from the model are obtained from records of 200 successive waves.

The prototype wave conditions were found to be reproduced satisfactorily in the model.

The models. The test arrangement is shown in Fig. 10. The tests were run using a scale of 1:36.

The wave basin is constructed for a water depth of approximately 1.0 metre, and a model bottom was constructed consisting of a slope 1:30 up to the correct sea bed level, whereafter the bed was kept horizontal.

Two cross sections with widths 1.0 metre were tested simultaneously, each positioned adjacent to the glass panels. With the different sea bed levels on each side of the basin the cross section became as symmetrical, but this did not have any significant * influence on the wave conditions.

Test. For the deep water sections at depth -20 and 15.5 metres the stability of the given design was to be investigated.

For the breakwater in shallow water special precautions have to be taken against erosion, and the breakwater will be founded on a dredged bottom below the original sea bottom level. To keep the cost of dredging at a minimum the berms are to be as high as possible below the still water level. The criterion for the stability of the berm was that no rock be washed into the front armour or carried away from the berm on the harbour side. In ad-

dition to the study of the armour layer of 39 ton blocks an objective of the tests was to find the maximum crest elevation of the berms.

During a test the significant wave heights used were 4, 5, 6, 7, 8, 8.5, 9, 9.5 metres, each run for a period equal to 10 hours in prototype.

The tests were conducted with water levels of + 0.5 and + 1.5 metres.

Test at DHL.

Test equipment. The system for generating irregular waves in the DHL channel works approximately on the same principles as that of the RHL, i.e. the signal from a noise generator is filtered to give an input to a servo controlled wave paddle, which generates a wanted wave spectrum in the channel. The control system of the wave generator is also designed to accept a wave record as input signal.

Waves. In the tests three types of spectra were used, and each spectrum was run with peak frequencies corresponding to wave periods of 10, 12 and 14 seconds in prototype. The spectra are shown on normalized form in Fig. 11.

The A- and B spectra corresponds to the spectra used at the RHL, and in addition a very wide spectrum (C) was used.

Tests. The tests were run in scale 1:60 with two cross sections at depth -20.0 and -15.5 metres. The tests were run at water levels + 0.5 and + 1.5 metres.

TEST RESULTS

The results of the tests conducted at DHL and RHL are shown in Fig. 12-18. The line in the graphs, illustrating the effect of regular waves, is drawn on basis of the average values of $K/f(\alpha)$ and H_{destr}/H_d from the tests in regular waves which gave the lowest stability.

As the structure was supposed to be stable up to the design wave height the damage was limited in all cases, and it is difficult to draw conclusions of a general nature about the influence of spectral shape, wave period etc. on the damage.

Also, the conclusions about the stability are of course limited to the particular breakwater design and under the particular bottom conditions tested. However, at present very few tests on breakwaters have been conducted, in which the effect of regular and irregular waves can be compared, and it might be of some interest to discuss the results in some detail.

The typical development in the 43 ton armour layer during a test was as follows: Already at a wave height of 4-5 metres the blocks began to rock in the highest wave, resulting in a slow settling in the armour and core. This process continued for increased wave attack, and the thickness of the armour on the crest decreased. In most tests just a few blocks were removed from the armour layer, and after the completed test the breakwater seemed impaired only to a small degree.

As a result of the tests with irregular waves it was decided to compensate for settling in the core by increasing the initial crest elevation.

By comparing the tests which have been run both at DHL and RHL it seems that no systematic difference can be traced in tests performed in the scales 1:60 and 1:36.

It has been commonly assumed that the destructive effect of a train of regular waves on a breakwater in model is equal to the effect of irregular waves with a significant wave height corresponding to the height of the regular waves.

By comparing the results of tests run with regular and irregular waves it can be seen that in this case the damage appeared in the armour layer at a lower significant wave height than the height of the regular waves. This is apparently due to the few high waves present also for lower significant wave heights.

The difference in stability is somewhat greater than indicated by the average minimum curve as this curve is based on the results from the tests in regular waves giving the lowest stability. To illustrate this all results obtained in regular waves have been evaluated for the condition $G = 43 \text{ t}$ and $\gamma_b = 2,65 \text{ t/m}^3$ and plotted in Fig. 19. In addition to the average minimum curve, the curve corresponding to the average of all results is drawn. In Fig. 20 all results obtained in irregular waves for the cross sections at depth -20 and -15,5 metres respectively, have been plotted for comparison with the average stability obtained in regular waves.

Also the stability of the berms seemed to be somewhat lower than observed in regular waves.

The increase of damage was, however, not considered to necessitate any change in the breakwater design.

In Fig. 21 is shown a diagram of damage vs. wave height for different wave spectra. The damage is expressed by the number of blocks which had to be placed on the model in order to restore the original shape. The number of blocks used for construction was approximately 250. In test of breakwater stability scatter is inherent, and the shaded area between results from two tests run under identical conditions illustrate the scatter in the test series. In view of the scatter, it is not possible to draw any conclusions on the effect of spectrum shape and peak frequencies on the stability.

FINAL COMMENTS

The tests described in this article were not meant to give results beyond those necessary to draw conclusions about this particular breakwater for a given range of wave dimensions. For this particular case, within the observed range of damage, irregular waves seemed to represent a more severe wave attack than regular waves with heights equal to the significant wave heights of the irregular waves.

In stability tests conducted at the RHL previously (Ref. 1, 2) relationships between damage and types of spectra have been indicated. It was shown that irregular waves, depending on the spectrum width, can be more or less dangerous than regular waves. Under certain conditions regular waves have been shown to be considerable more destructive.

On the basis of the sum of experience made so far, the conclusion seems to be that the factors which influence the stability of a breakwater are many and complex and vary within wide ranges from project to project. The best basis for breakwater design is still model testing, preferably with irregular waves.

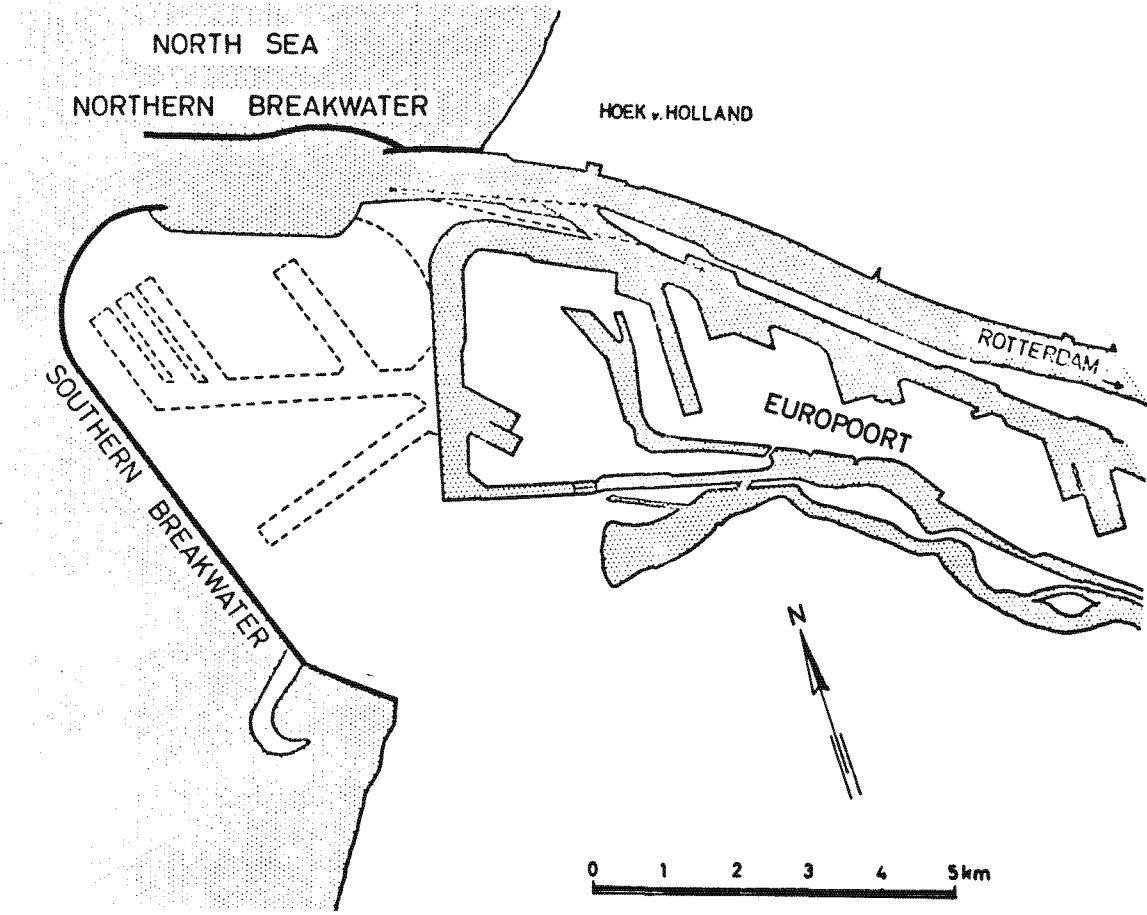


Fig. 1. Plan Europoort.

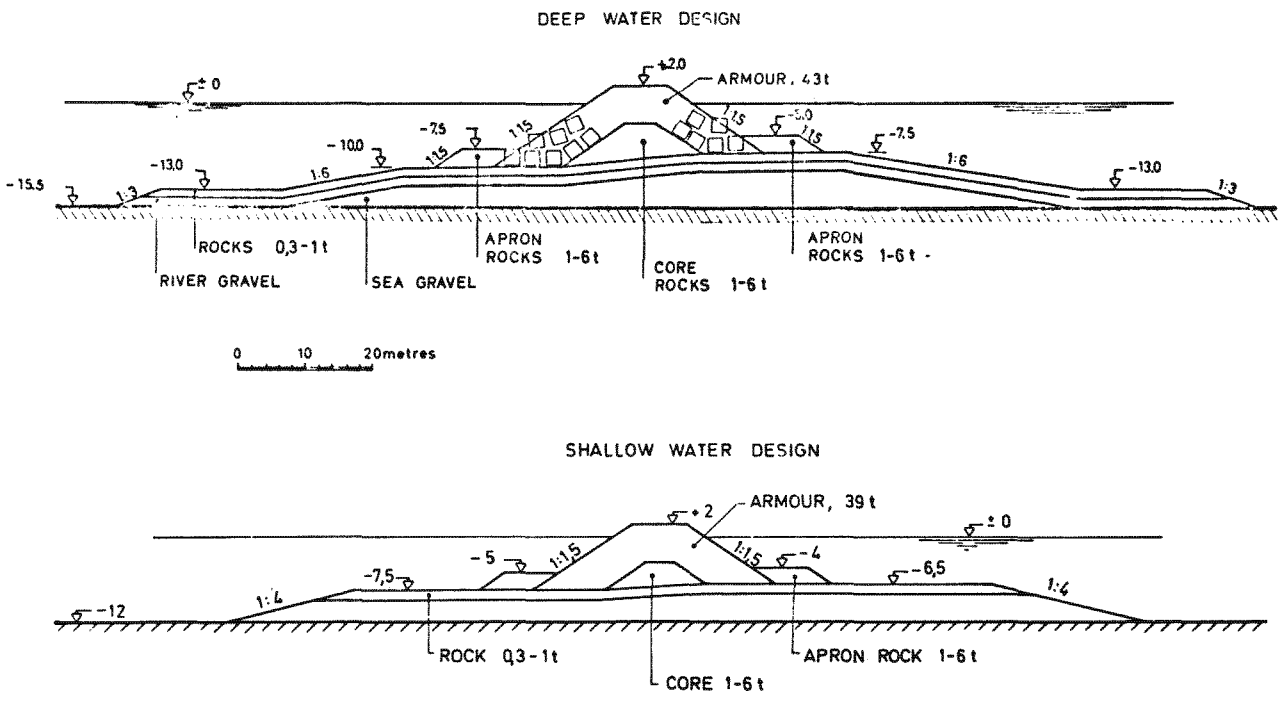


Fig. 2. Typical cross sections.

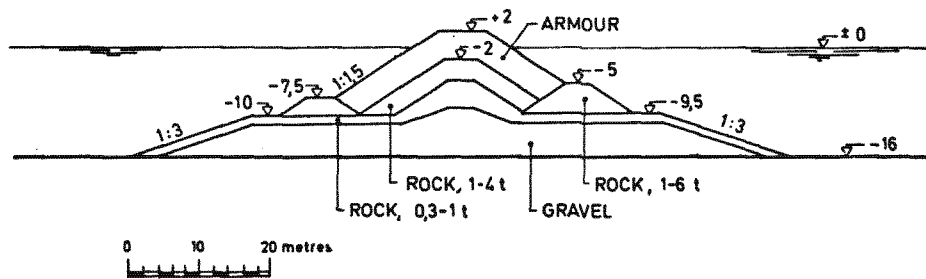
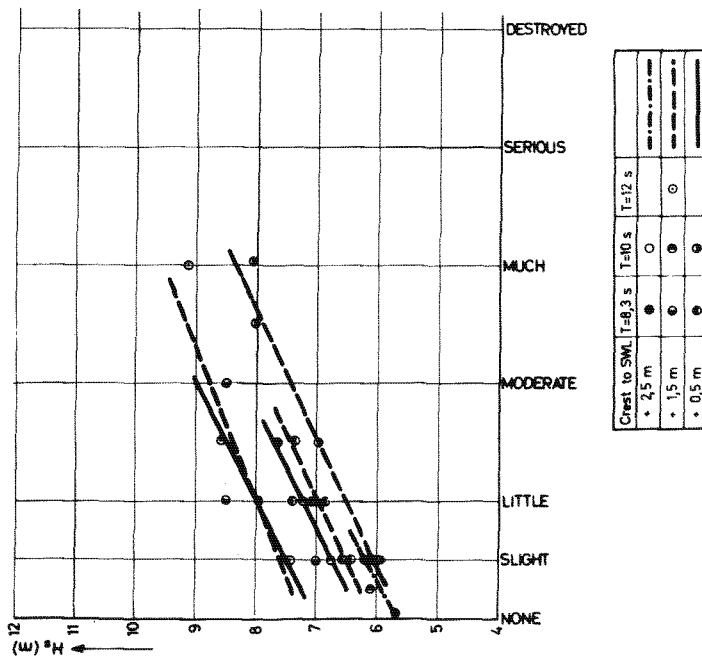


Fig. 3. Cross section. Tests with regular



** Fig. 4. Example of test results, regular waves. DHL.

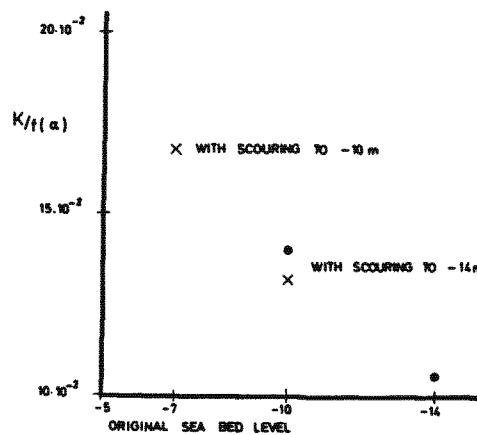


Fig. 5. Stability as a function of sea bed level. Regular waves. DHL.

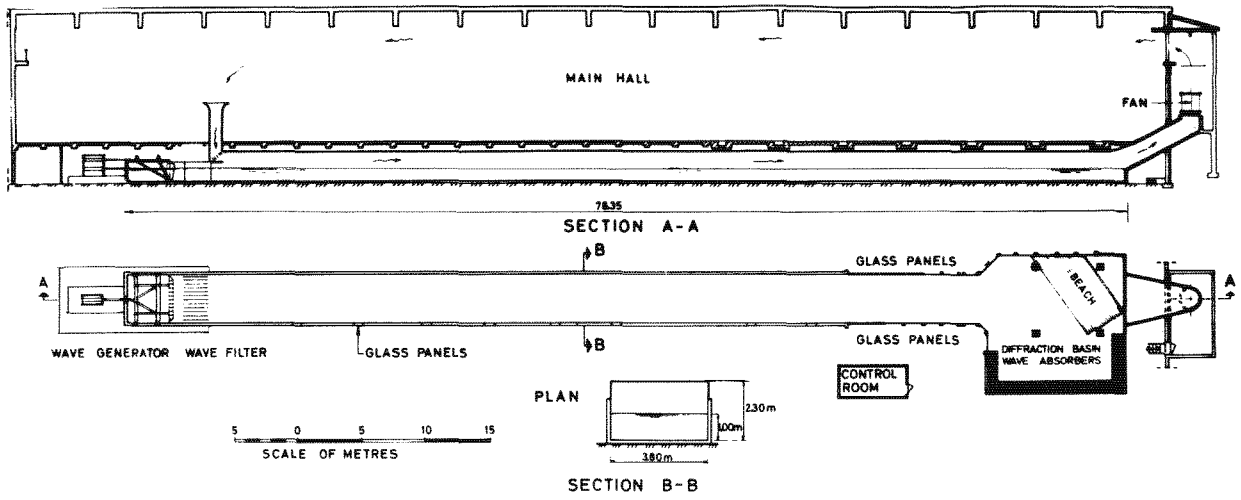
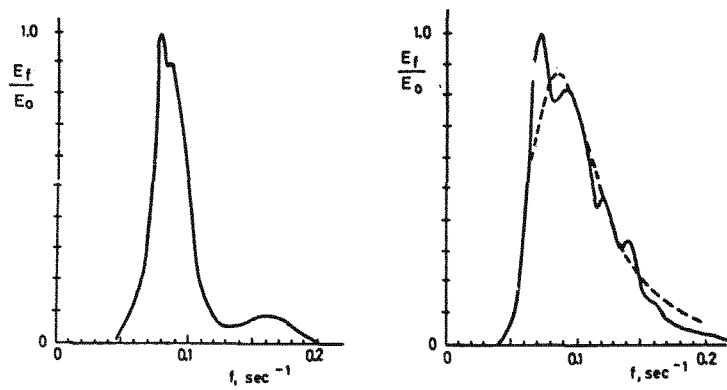
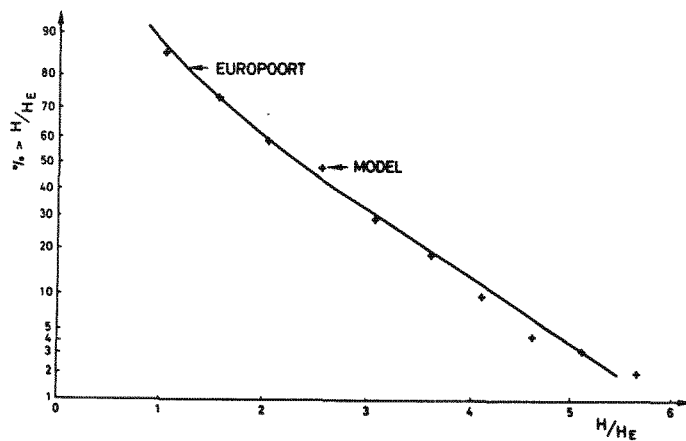


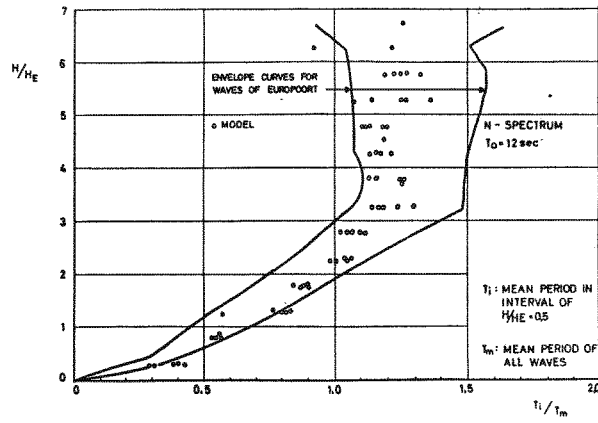
Fig. 6. Wave channel. RHL.



** Fig. 7. Wave spectra. RHL.



** Fig. 8. Wave height distribution. RHL.



** Fig. 9. Correlation between wave height and period. RHL.

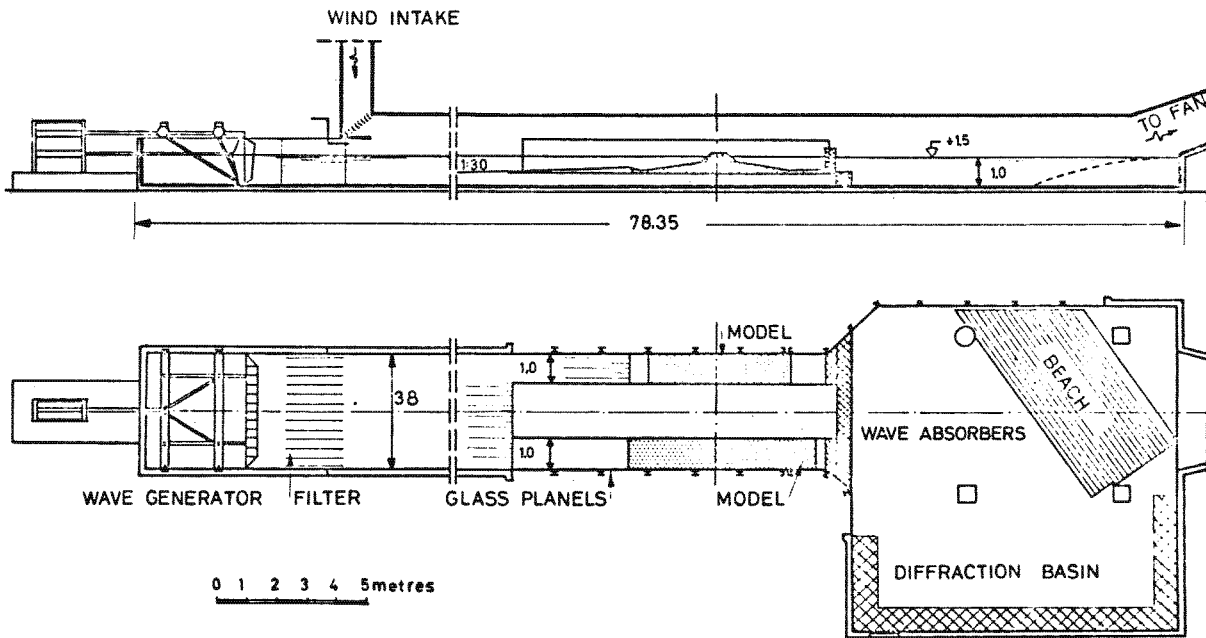


Fig. 10. Test arrangement. RHL.

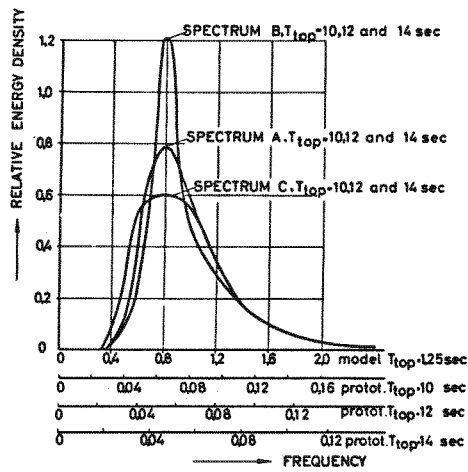


Fig. 11. Wave spectra. DHL.

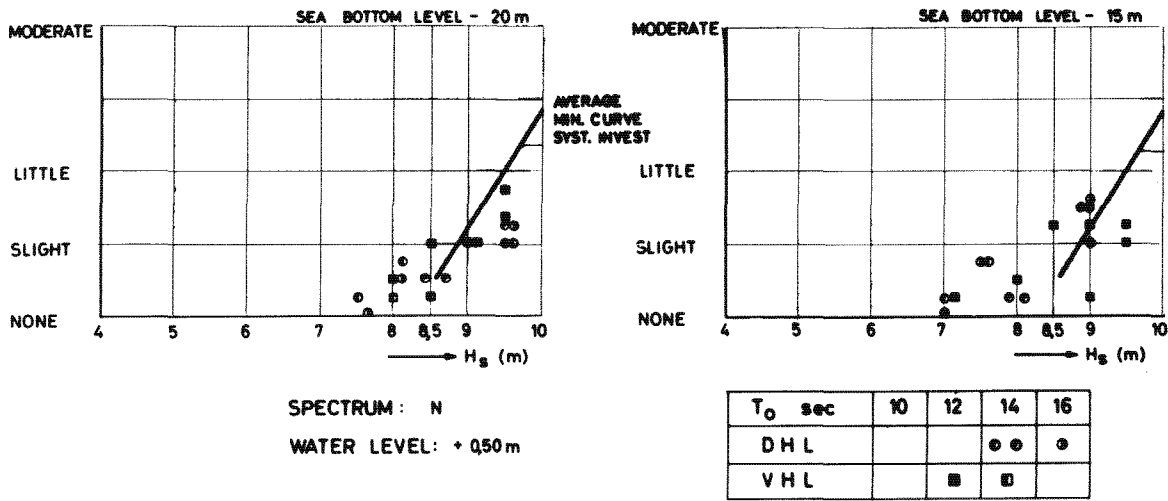


Fig. 12. Test results. Irregular waves.

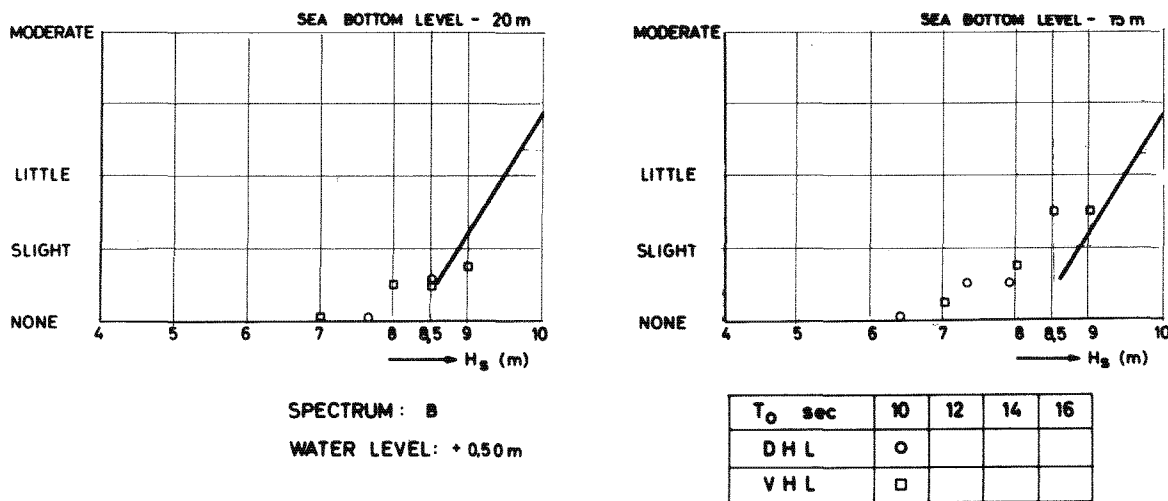


Fig. 13. Test results. Irregular waves.

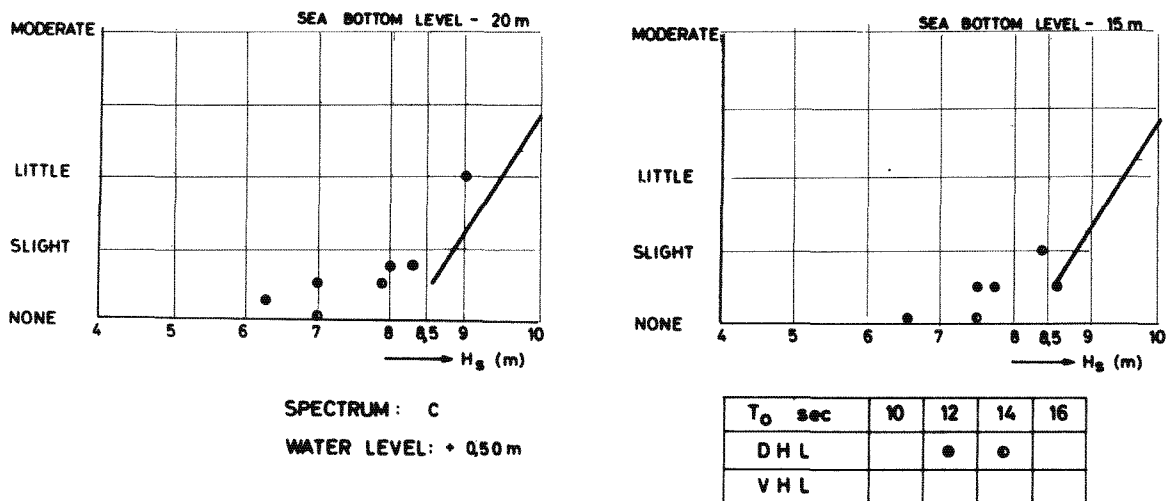
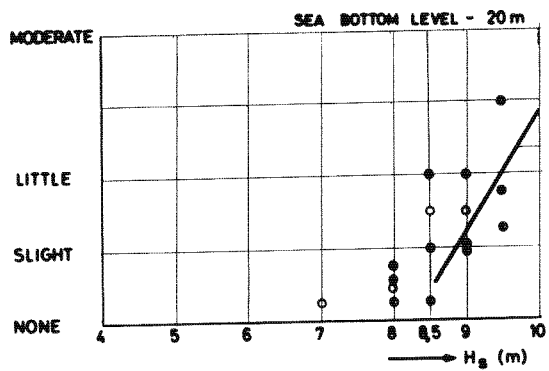
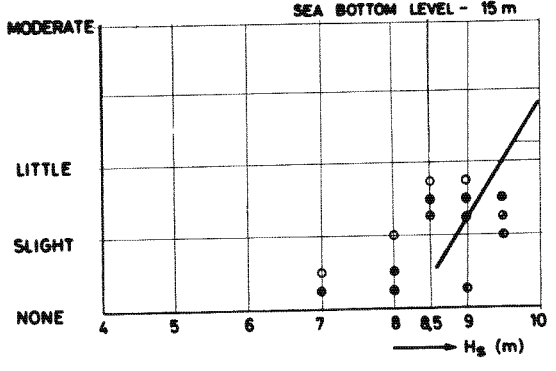


Fig. 14. Test results. Irregular waves.

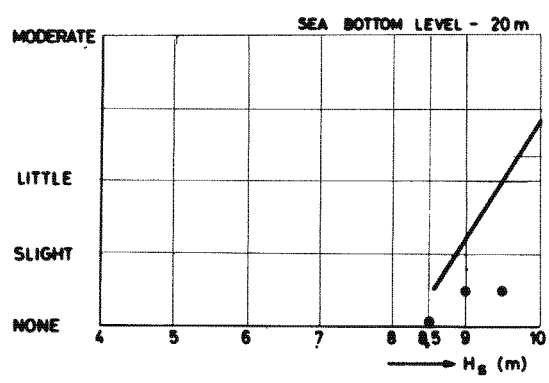


SPECTRUM: N
WATER LEVEL: +0.50m

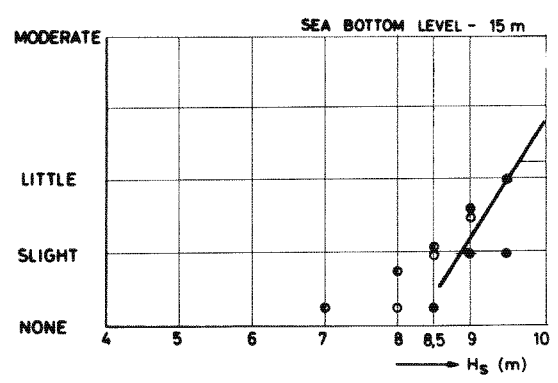


T ₀ sec	10	12	14	16
DHL				
VHL	○	●●	●	

Fig. 15. Test results. Irregular waves.

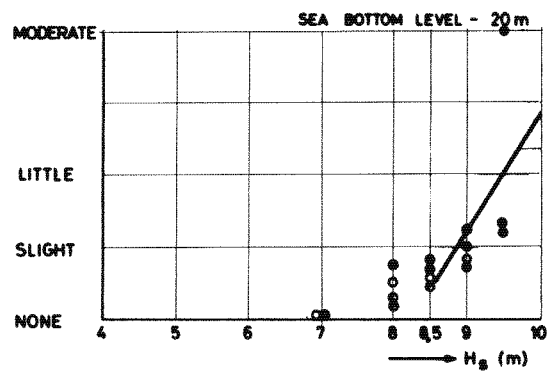


SPECTRUM: N
WATER LEVEL: +1.50m

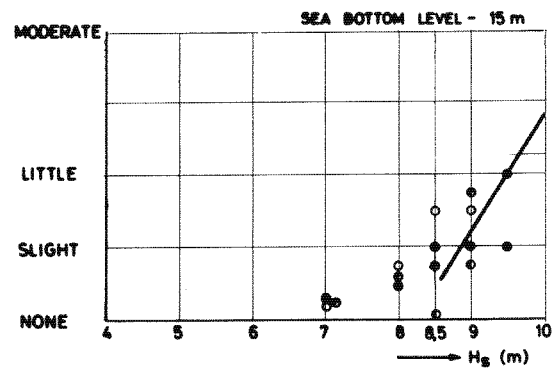


T ₀ sec	10	12	14	16
DHL				
VHL	○	●	●	

Fig. 16. Test results. Irregular waves.



SPECTRUM: B
WATER LEVEL: +0.50m



T ₀ sec	10	12	14	16
DHL				
VHL	○	●	●●	

Fig. 17. Test results. Irregular waves.

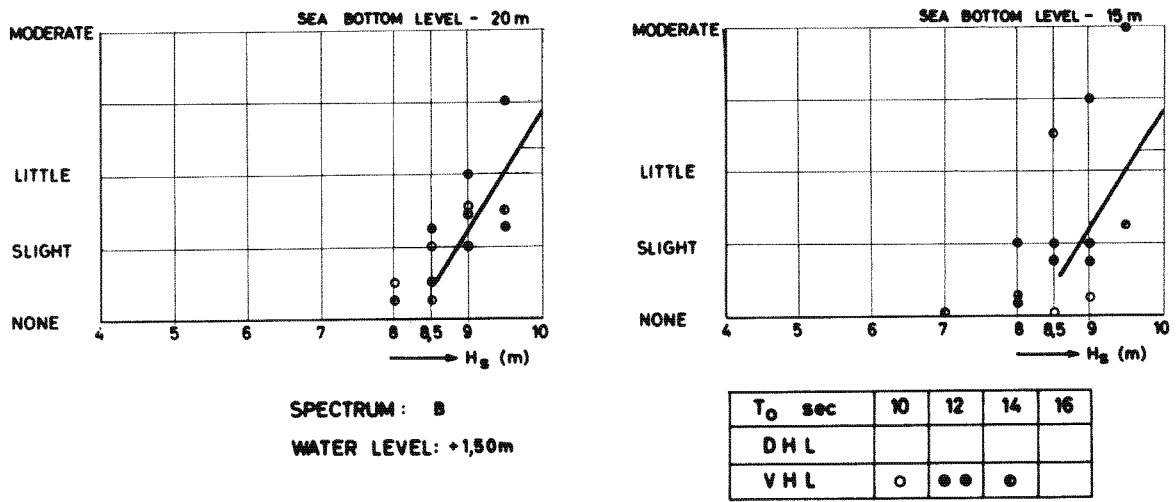
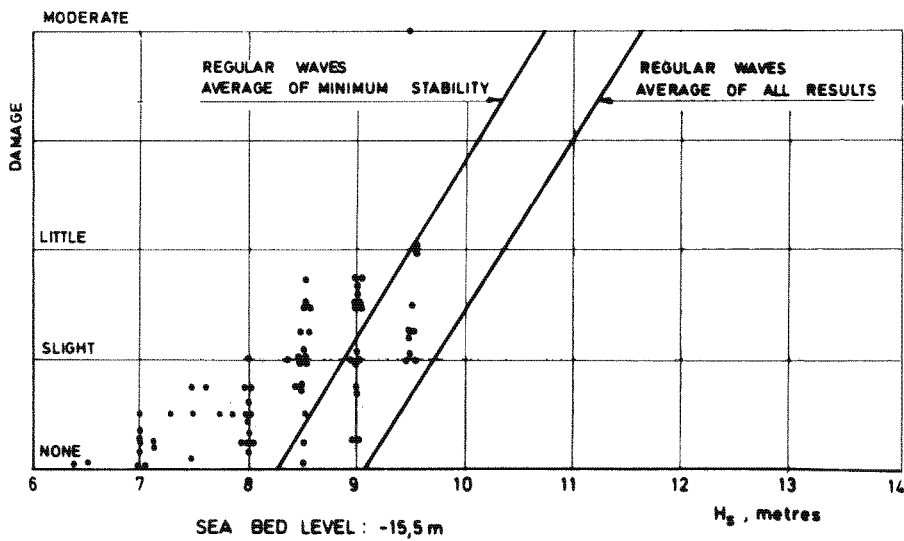
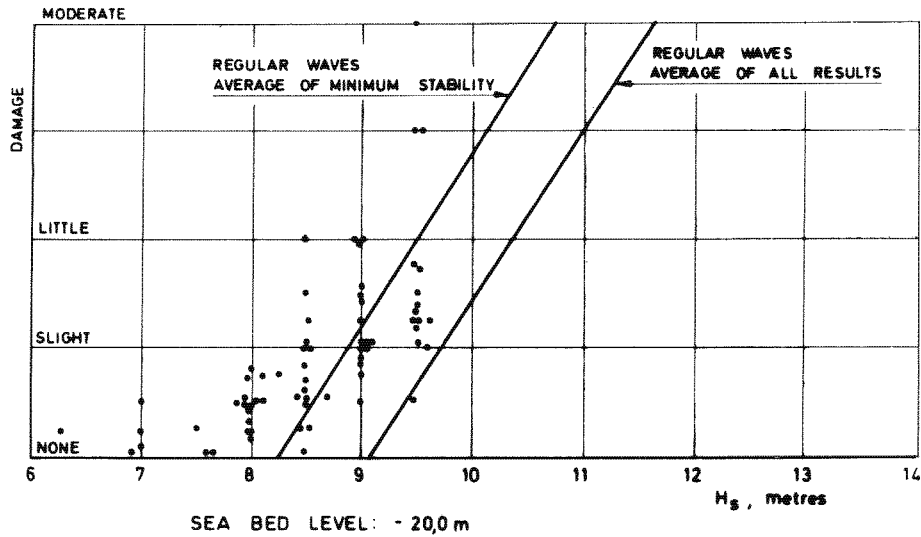
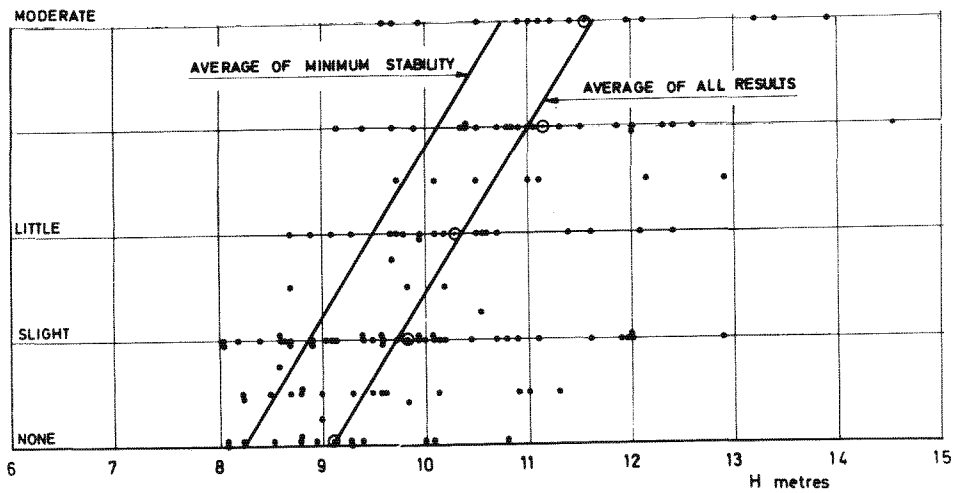


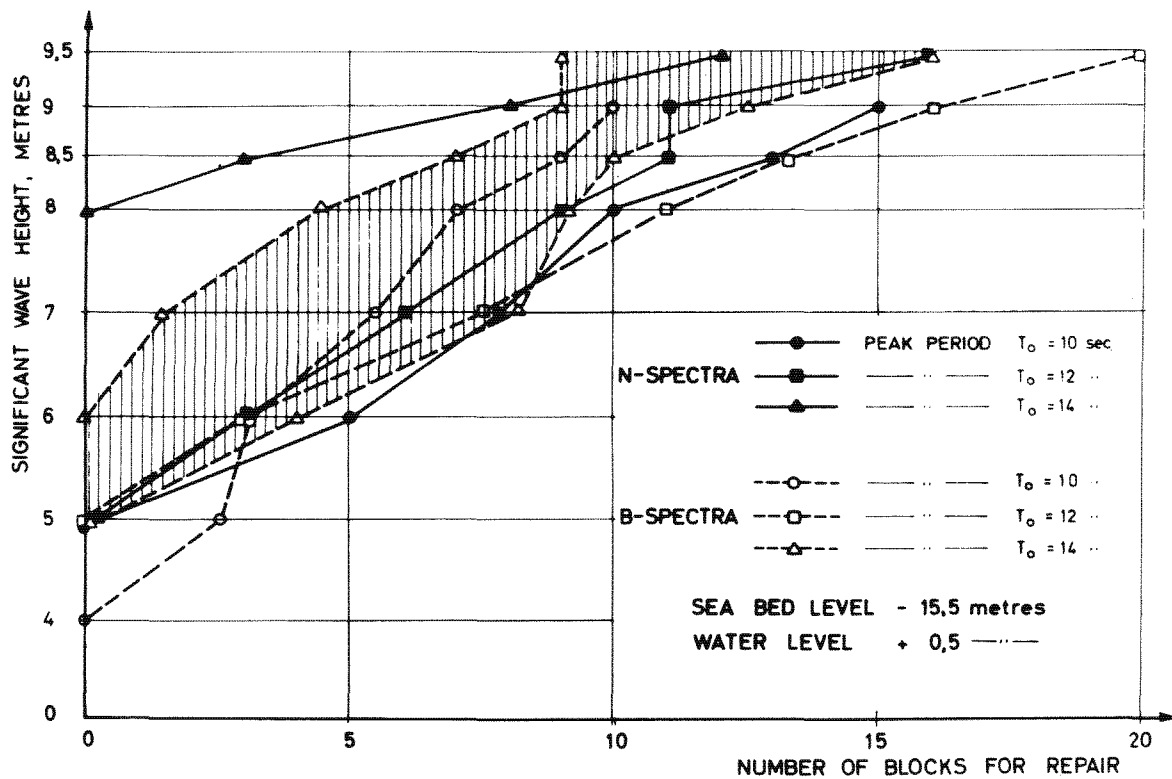
Fig. 18. Test results. Irregular waves.



* Fig. 19. Results from tests in regular waves evaluated for conditions $G = 43 \text{ t}$, $\gamma_D = 2,65 \text{ t/m}^3$.



* Fig. 20. Results in irregular waves.
All test data.

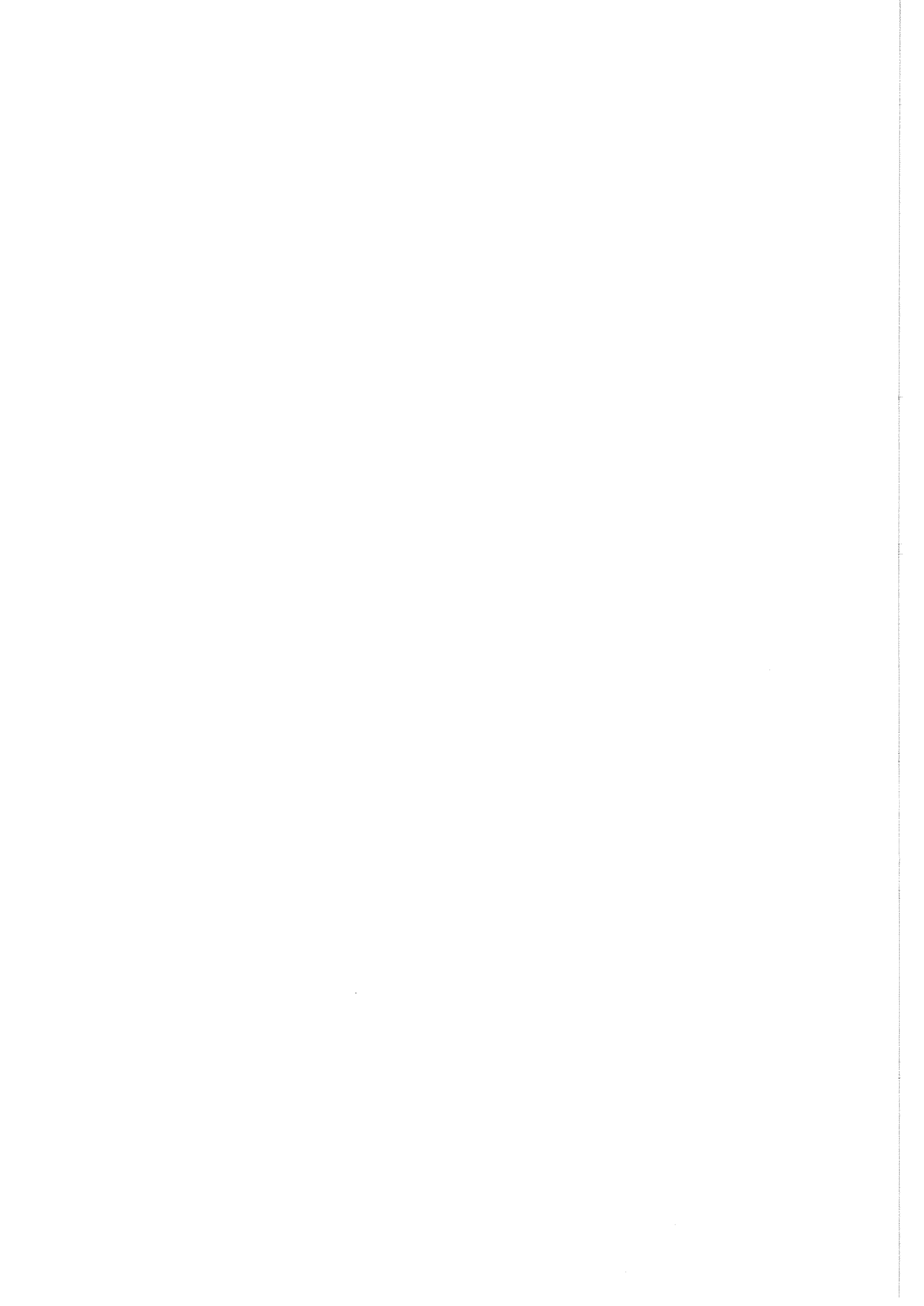


** Fig. 21. Test results. Irregular waves. RHL.

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1. Svee, R., Trøttestad, A. and Tørum, A.
"The Stability Properties of the Svee-block".
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2. Carstens, T., Trøttestad, A. and Tørum, A.
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DISCUSSION ON PAPER 10

J. H. van OORSCHOT

Delft Hydraulics Laboratory, The Netherlands

Berge and Treatteberg conclude, after having reviewed the results of experiments performed at the R.H.L.* and the D.H.L.** regarding the Europoort breakwaters, that "the factors which influence the stability of a breakwater are many and complex and vary within wide ranges from project to project. The best basis for breakwater design is still model testing, preferably with irregular waves".

Additional tests performed at the D.H.L. with a somewhat different cross-section have once again positively underlined the significance of this conclusion. Moreover, some results of these additional experiments are so interesting that a short discussion seems justified.

The cross-sections tested by R.H.L. and D.H.L. can be realized at those places where the original depth is about N.A.P. - 15 to - 20 m. However, parts of the Southern breakwater have to be constructed on a shallower bottom. After the construction is finished, the contraction of the tidal currents in combination with dredging work at the harbour entrance will cause a scouring in front of the bed protection of the breakwater up to N.A.P. - 20 m or even N.A.P. - 25 m. The influence of this bottom configuration on the stability of the breakwater has been extensively tested by the D.H.L. Though the differences in the cross-sections compared with those described by Berge and Treatteberg are minor, the stability appeared to be entirely different. With a proposed length of the apron of 20 m, serious damage occurred under design-wave conditions, whereas, considering the former experiments, only slight damage was expected. Moreover, damage increased rapidly with increasing wave height. After having considered this result, it was decided to vary the apron length in the model. Four lengths were taken: 3, 12.5, 20 and 50 m.

* River and Harbour Research Laboratory, Trondheim.

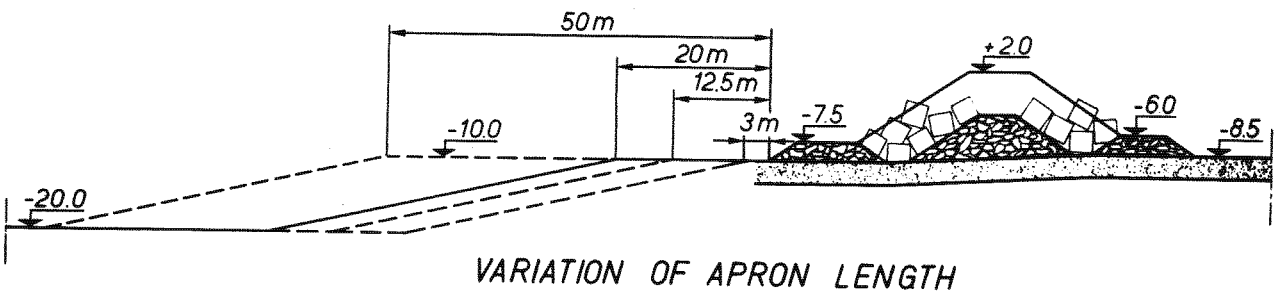
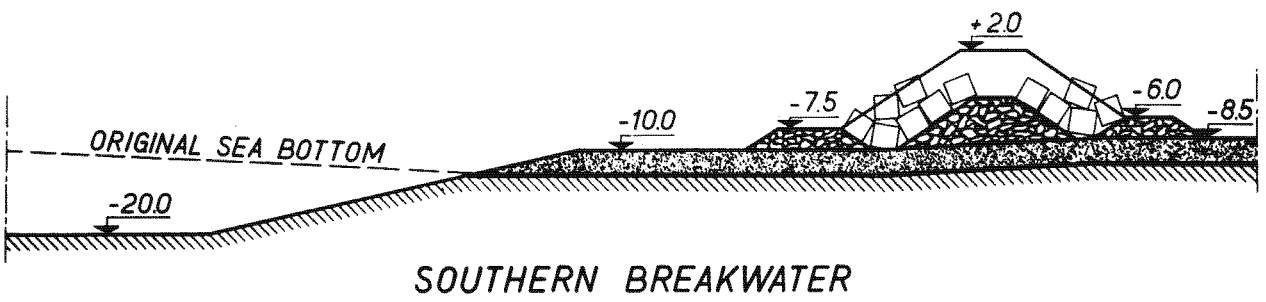
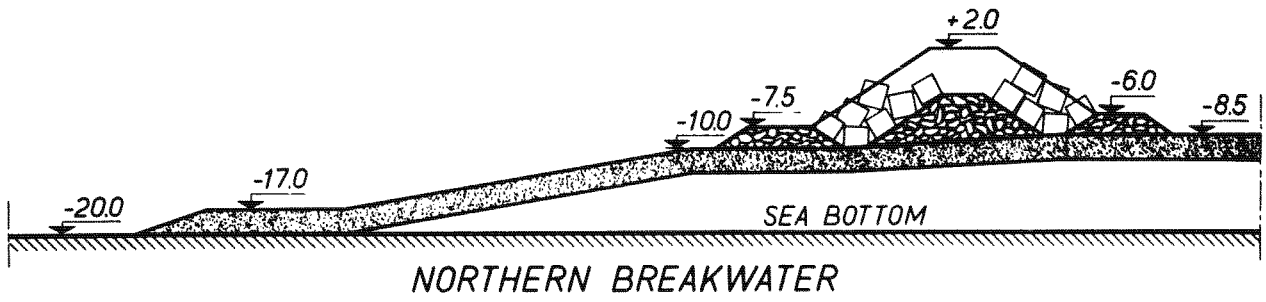
** Delft Hydraulics Laboratory.

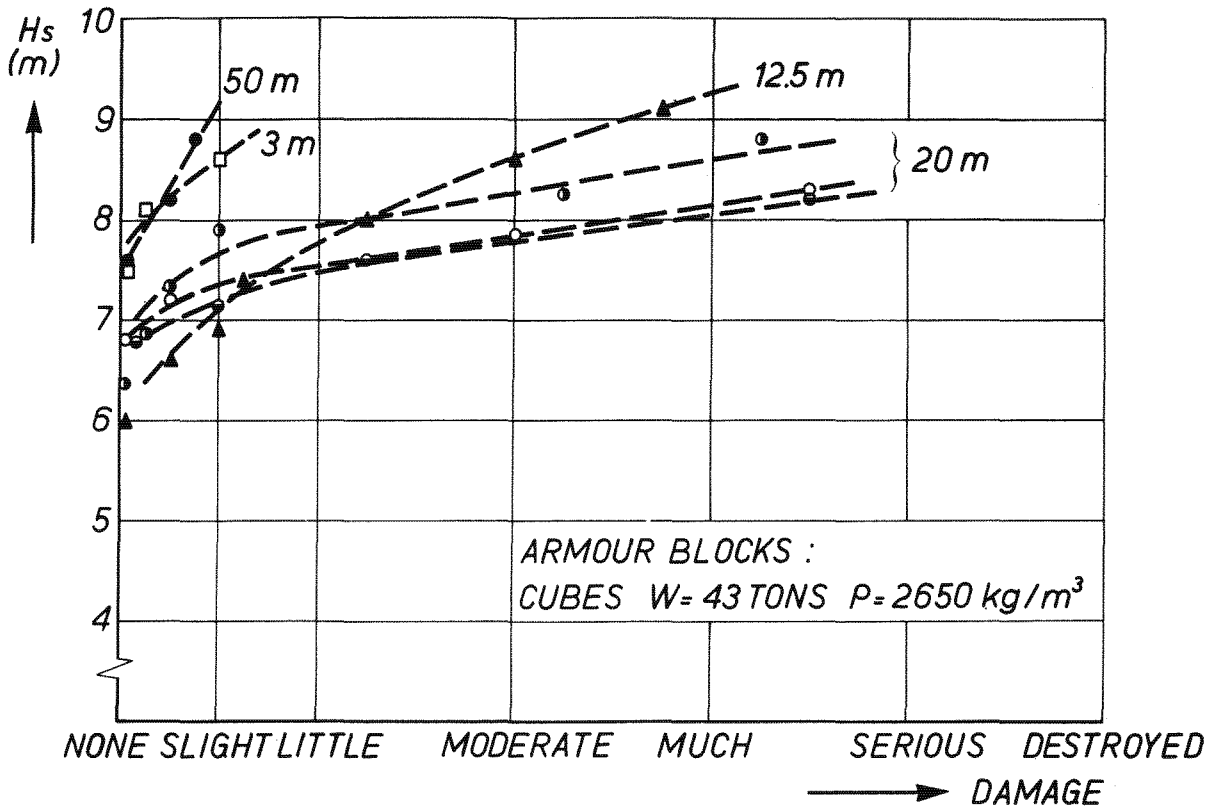
The results show the smallest damage for the 3 and 50 m aprons and a maximum damage for the originally selected length of 20 m.

Visual observations of the model have created the impression that the character of the breaking wave is one of the causes of this phenomenon. Wave attack on the armour blocks got more the character of wave impact with increasing apron length up to 20 m, whereas at a length of 50 m a substantial amount of energy has already been dissipated by wave breaking in front of the breakwater.

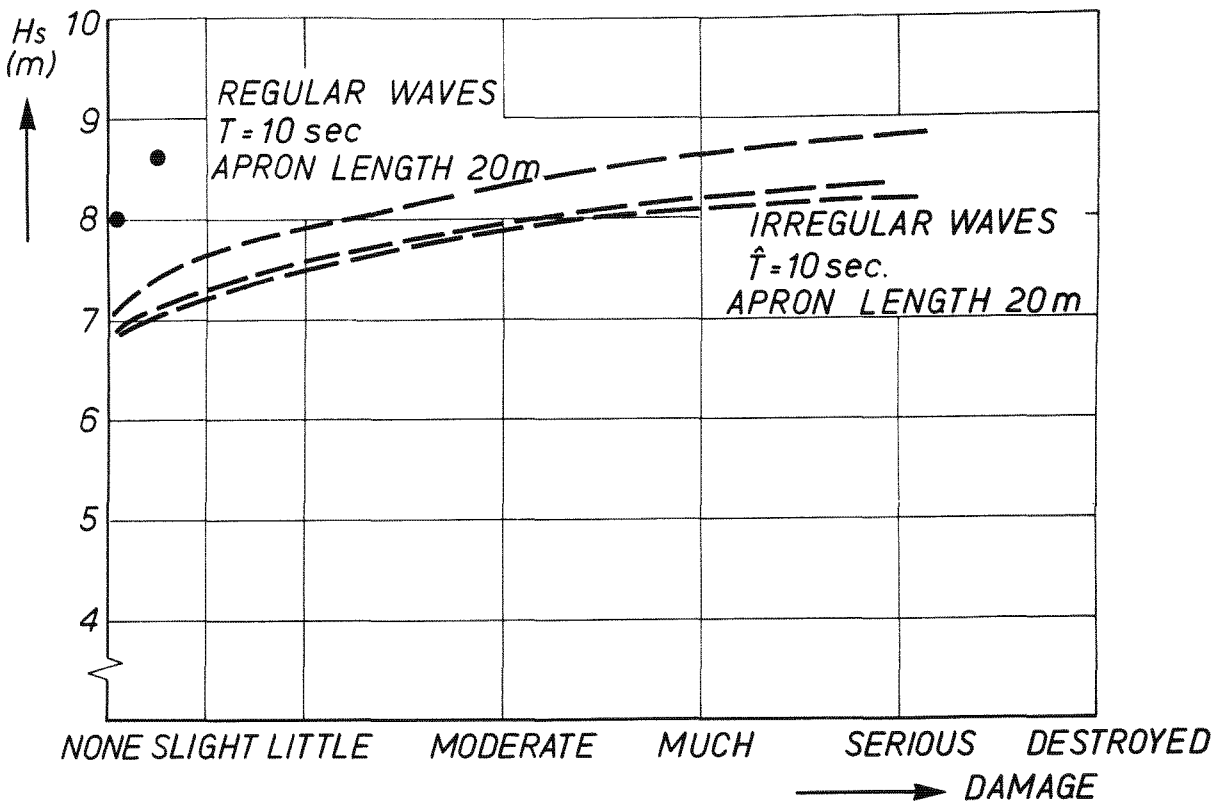
One reference test applying regular waves showed hardly any damage at the design wave height of 8.5 m and an apron length of 20 m, whereas in the case of irregular waves the structure was seriously damaged at the corresponding significant wave height and period. Though for some specific problems regular-wave experiments may yield useful information, this result once more focusses attention on the risk of this method.

The amount of influence of the apron length was not predicted, and was discovered only thanks to the fact that extensive model studies had been performed for the Europoort project. As apparently minor factors have an important influence on model results, the extrapolation of data to other problems, which may look similar in a first approximation, has to be applied with the utmost care. It is evident that the same holds good even more for the use of stability formulae.





DAMAGE ARMOUR-LAYER AS A FUNCTION OF WAVE HEIGHT AND APRON LENGTH.



COMPARISON OF DAMAGE CAUSED BY REGULAR AND IRREGULAR WAVES.