PORT AND OCEAN ENGINEERING UNDER ARCTIC CONDITIONS
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STABILITY TESTS ON A RUBBLE MOUND BREAKWATER HEAD IN REGULAR AND IRREGULAR WAVES. SØRVÆR FISHING PORT, NORWAY.

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1. INTRODUCTION.

The stability of rubble mound breakwaters has been studied by several investigators through the years. Different formulas for calculating the necessary armour block weights have been proposed. The formulas have been based on different philosophies on the mechanics of the dislocation of the armour blocks and the damage of the breakwater.

All the proposed formulas have in common that they include coefficients with values determined by model tests. The tests have mostly been carried out with uniform waves and the value of the coefficients have been based on some percentage of damage.

The results from tests on rubble mound breakwaters in regular waves have been used for prototype irregular sea. However, when using the results from tests on rubble mound breakwaters in regular waves it is a question what waveheight the damage wave height would correspond to in an irregular sea. This question was open until modern testing equipment was developed. In 1966 tests were carried out at the River and Harbour Laboratory (RHL) at the Technical University of Norway where the effect on non-overtopping regular and irregular waves on the trunk of a rubble mound breakwater was investigated [1]. The number of test conditions were limited compared to the number of conditions to be handled in practice. However, the following conclusions were drawn from these tests:

"The tests have indicated a relation between stability and run up rather than apparent wave heights, the run up being a function of the spectrum. The spectrum with the highest run up was observed to have the lowest number of whitecaps in front of the breakwater. As one could expect, there does not seem to exist one single relation which describes the effect of regular waves compared with irregular waves valid for all shapes of wave spectra. The substitutions in model tests on breakwater stability of a confused sea with a wave train of significant waves is not a safe procedure for all wave spectra".

As a rule of thumb for feasibility study designs the test results showed that when applying, say the Svee formulae [2], for evaluating the necessary weight of the armour stones, one should use the significant waveheight $H_{1/3}$ in the formulae.

This conclusion was also reached by Rogan [3] in a paper to the Symposium on Wave Research in Delft 1969.

Results from a two-dimensional case study carried out at RHL, regarding the stability of the new Europort breakwater, showed that irregular waves seemed to represent a more severe wave attack than regular waves with heights equal to the significant wave heights of the irregular wave. It was further concluded that the factors that 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. This conclusion was also positively underlined by van Oorschot in a discussion paper to the Delft Symposium on Wave Research in 1969, [4] and [5].

2. THE BREAKWATER AT SØRVÆR FISHING PORT.

In view of the above conclusions a case study has been carried out at RHL regarding the stability of the head of the rubble mound breakwater under construction at the fishing port of Sørvær in the northernmost county Finmark of Norway. The sponsor of the tests was the Norwegian State Harbour Works. The study involved evaluation of the design wave height from meteorological data as well as model testing. The evaluation of the design wave height followed more or less standard procedures, the presentation of which is not included in this paper. The tests involved intensive investigations of the breakwater stability in regular waves and some check tests and comparative tests in irregular waves. It is the latter tests that are considered to be of some general interest.

Fig. 1 is a map showing the location of Sørvær. Fig. 2 shows the harbour plan with the new breakwater, and Fig. 3 shows the detail of the bottom configuration at the breakwater head. The breakwater head is to be located on an underwater reef with a depth of approximately - 5 m. The reef has rather steep slopes down to a depth of approximately - 20 m.

The breakwater was first extensively tested on a scale 1:40 in a basin with regular waves only. Fig. 3 shows also the boundaries of the model basin during these tests. The design significant wave was evaluated to be 5,5 m at a location approximately corresponding to the location of the wavegenerator flap.

After extensive testing in regular waves in close cooperation with the sponsor, a breakwater head design as shown in Fig. 4 was adopted. As a final check this design was then tested in irregular waves on scale 1:40. Some tests with regular waves were also included in this test series for further comparison of the effect of regular and irregular waves on the breakwater.

The weights of the armour blocks in the tests corresponded to 16 - 20 metric tons. At the crest of the breakwater head is a heavy concrete cap, and a berm is to be constructed at the foot of the breakwater. The size of the berm material was not exactly known at the time of testing, but was judged from the visible surface material of the large gravel stones to have equivalent diameters 11 - 25 cm. In case the natural gravel stone material is smaller than assumed the berm should be covered with heavier quarry stones.

The sieve curve of the anticipated core material is shown in Fig. 5.

2.1. Model test set up.

The comparative tests in irregular and regular waves were carried out in scale 1:40 in a wave flume shown in Fig. 6. Fig. 7 shows the model test set-up in this flume. The breakwater head was built on a flat bottom without exact reproduction of the bottom configuration. The underwater reef was hence also omitted and the cross section of the breakwater modified as shown in Fig. 7.

The water depth at the plateau where the model was built was approximately 40 cm. From this plateau the depth increased to about

1 m through slopes of 1:30 and 1:20 to the horizontal bottom of the flume.

The wave generator consists in principle of a wave paddle operated by two hydraulic pistons, the movement of which are controlled by an electric signal from a sine wave generator (regular waves) or a magnetic tape (irregular waves).

The model breakwater was built with water in the flume. In the prototype cranes were expected to be used and this was simulated in the model. The armour blocks below low water was dropped from the watertable, while the blocks above the water line was placed directly on the slope by simulating the use of a crane. The engineer in charge of construction was present in the laboratory during most of the tests.

The tests K8 - K10 were only concerned with the stability of the foot berm. During these tests the rubble mound blocks were placed to form a very stable cover layer.

2.2. Waves.

The tests were run with both regular and irregular waves. The wave period for the regular waves were kept constant equal to 9 sec. This period corresponded to the period of the peak in the wave spectrum used for the irregular waves. The normalized form of the applied wave power spectrum is shown in Fig. 8. In the same figure is also shown different theoretical spectra and a spectrum from wave measurements at the fishing port Berlevåg in Finmark, Norway. As can be seen the spectrum used during the test is narrow compared to the theoretical spectra, but fairly close to the Berlevåg spectrum. No effort was made to use another spectrum than the one that was available in the magnetic tape library.

During the tests the wave height was increased in steps until failure in the armour layer occurred. The stroke amplitude of the wave paddle, and hence the wave height, is controlled by the voltage reference signal from the magnetic tape and is easily varied by varying the amplification of the reference signal. The wave period distribution is determined by the taped program and the speed of the magnetic recorder,which is fixed.

The energy in the model power spectra will thus increase for all frequencies, as indicated in Fig. 9 instead of an increase of energy with decreasing frequencies as will occur within a wave generating area in the prototype.

During the tests the waves were measured both in the deep part of the flume and in front of the breakwater. Fig. 10 shows wave height distributions in the deep part of the flume. Since the waves in a point just in front of the breakwater head is affected by reflections in different directions, the wave heights measured in the deep part of the flume is used for a general comparision of the effect on the breakwater head from regular and irregular waves.

3. TEST AND TEST RESULTS.

It turned out that failure of the rubble mound layer never occurred due to erosion of the foot berm. We have therefore delt with the stability of the armour layer and the foot berm separately.

3.1. Stability of the rubble mound layer.

Nine relevant test series were carried out: five series with regular waves and four with irregular waves.

Each test series was carried out with a waterlevel first corresponding to + 3.00 m (± 0 = mean low water). The wave heights were increased in steps of about 1 m, starting with the wave height usually about 3 m. Each step in the wave height was then usually run for 15 min until the design wave height was reached. If unacceptable damage had not occurred the water table was lowered to ± 0 and the procedure was repeated, starting from a wave height of about 3 m again, and increasing the wave height until unacceptable damage occurred.

During one test series with regular and one series with irregular waves each wave step was run for a longer time period to investigate the influence of a longer testing time. The long term tests were carried out with a waterlevel of + 1.5 m.

The behaviour of the structure was carefully watched during the tests. For the foot berm five profiles I - V, Fig. 7, were taken before the tests started and after the design wave had been run for waterlevel + 3 m and ± 0 m.

When the waterlevel was ± 0 the highest irregular waves were breaking before they reached the structure when the significant wave height was about 5 m or more.

The main test results regarding the stability of the rubble mound layer are shown in Fig. 11. The waveheights for the irregular waves are the significant waveheights.

Since the wave heights were increased in steps, the shaded areas in Fig. 11 indicate that the wave height for unacceptable damage is in the shaded range.

Fig. 12 shows damage curves for the long term tests.

In view of the unavoidable scatter in the test results that has to be expected in results of rubble mound breakwater stability tests, the results show that the stability of the armour block layer was approximately the same in regular and irregular waves when the unacceptable damage wave height for regular waves corresponded to the significant wave height in the train of irregular waves.

The long term tests indicate damage for a slightly lower wave height than in the short term tests. However, only one test has been run for each wavetype and no definite conclusion can be drawn regarding the effect of the time factor.

The damage curves for regular and irregular waves, long term test Fig. 12, are somewhat different. For the regular waves the dislocation of blocks occur mainly when the wave height reaches the damaging wave height. In irregular waves the dislocation of the blocks are by the highest waves that occur every now and then, giving the armour layer time to relocate and stabilize itself.

3.2. Stability of the foot berm.

Most of the tests were carried out with the foot berm at an elevation - 9.0. Two test series were carried out with coarser material on top of the 11 - 25 cm material in the foot berm. The coarser material corresponded to the core material and was placed on the horizontal part of the foot berm between elevation - 9.0 m and - 7.5 m.

The most damage on the foot berm occurred usually at profile III (Fig. 7). Fig. 13 shows some comparision of this profile before and after tests for different test series.

Fig. 14 shows some profiles before and after tests when the foot berm was covered with coarser material.

It is interesting to note in this case that while the tests with regular waves show practically no erosion the tests with irregular waves do show an erosion. A conclusive explanation of this is not possible from these limited number of tests. However, the explanation is tentatively the following:

In the tests showing differences in erosion when using regular waves and irregular waves, $H_{reg} = H_{1/3\ irreg}$, the height of the regular waves has not been large enough to really cause any erosion, just small motions of the sand grains. When the foot berm is exposed to irregular waves with a significant wave height equal to the height of the regular waves, some of the irregular waves are large enough to cause erosion.

If the wave height is increased further, the erosion will also start for the regular waves. The erosion by regular waves in a given time period is approximately the same as the erosion by the higher but fewer erosive waves in a train of irregular waves. (Fig. 13).

4. CONCLUSIONS.

The tests have been concerned with a specific breakwater project. The design has been governed by technical and economical factors. The tests have been limited in number, but the tests have revealed things that should be of general interest regarding the testing of a breakwater head in a model.

1. The stability of the armour layer on the specific breakwater head is approximately the same in regular and irregular waves, when the significant waveheight of the irregular wavetrain correspond to the height of the regular waves.

   The two longterm tests indicated a breakdown for slightly lower waves than for the short term tests. However, since only two such tests were run, no definite conclusions can be drawn on the effect of testing time.

3. Erosion on the foot berm was in some cases the same in regular and irregular waves. The tests indicated, however, that the erosion was larger with irregular waves than with regular waves when the regular wave height was just below the wave height when erosion started. At larger waveheights the erosion tended to be approximately the same for regular and irregular waves.

LITERATURE.

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2. Svee, R.: Formulas for design of rubble mound breakwaters. Proc. of the ASCE, Journal of the Waterways and Harbour Division, May 1962.

3. Rogan, A. J.: A comparison of regular and wind-generated wave actions on rubble mound breakwaters. Proc. Symposium "Research on Wave Action", Delft 1969.

4. Berge, H., and Trætteberg, A.: Stability tests of the Europoort breakwater. Proc. Symposium "Research on Wave Action", Delft 1969.

5. van Oorschot, J. H.: Discussion on [4], Proc. Symposium "Research on Wave Action", Delft 1969.

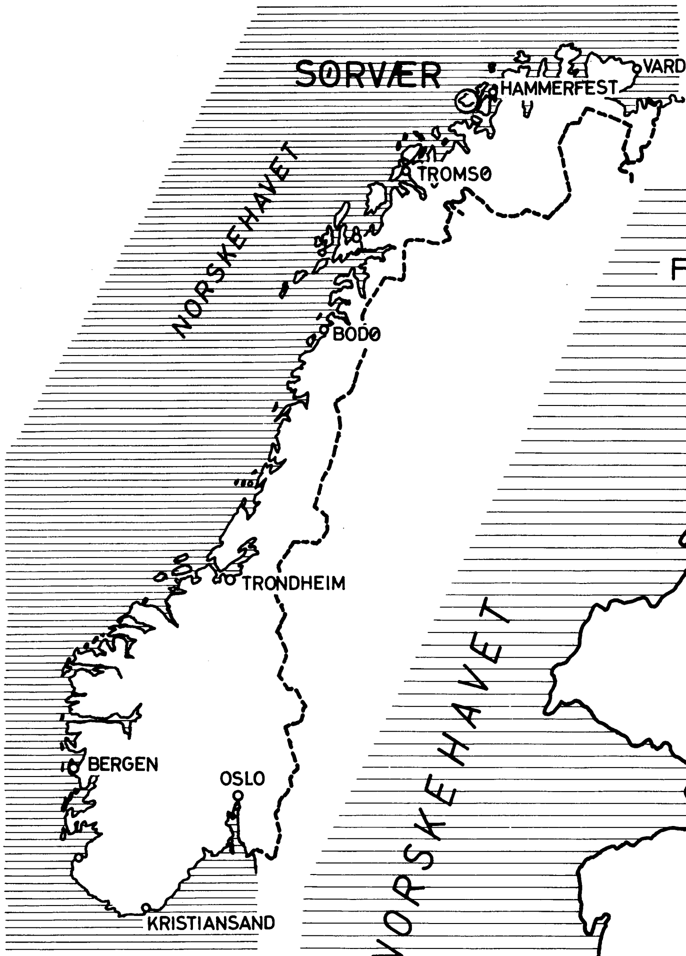


FIG.1 LOCATION MAP

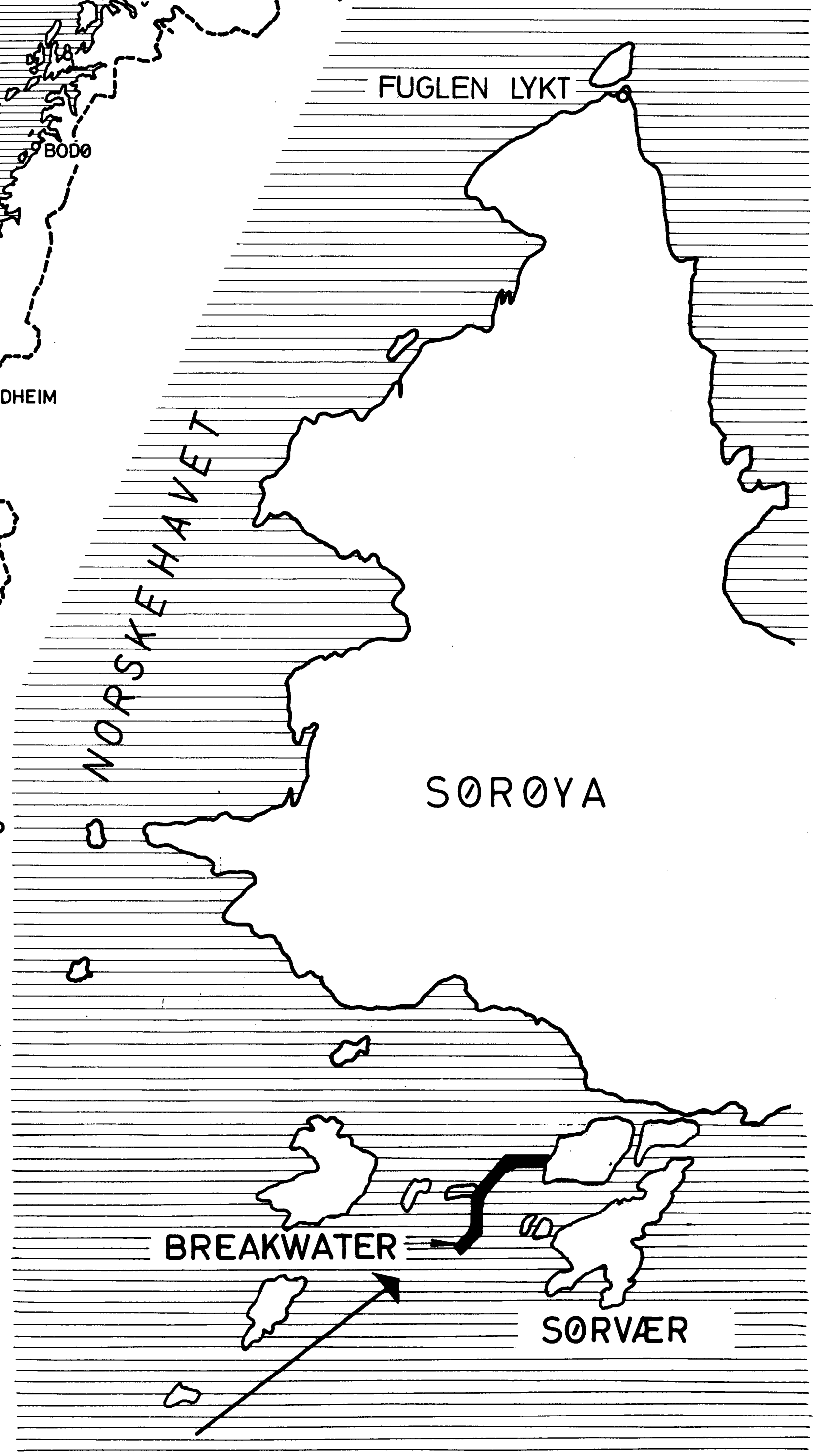


FIG.2 HARBOUR PLAN

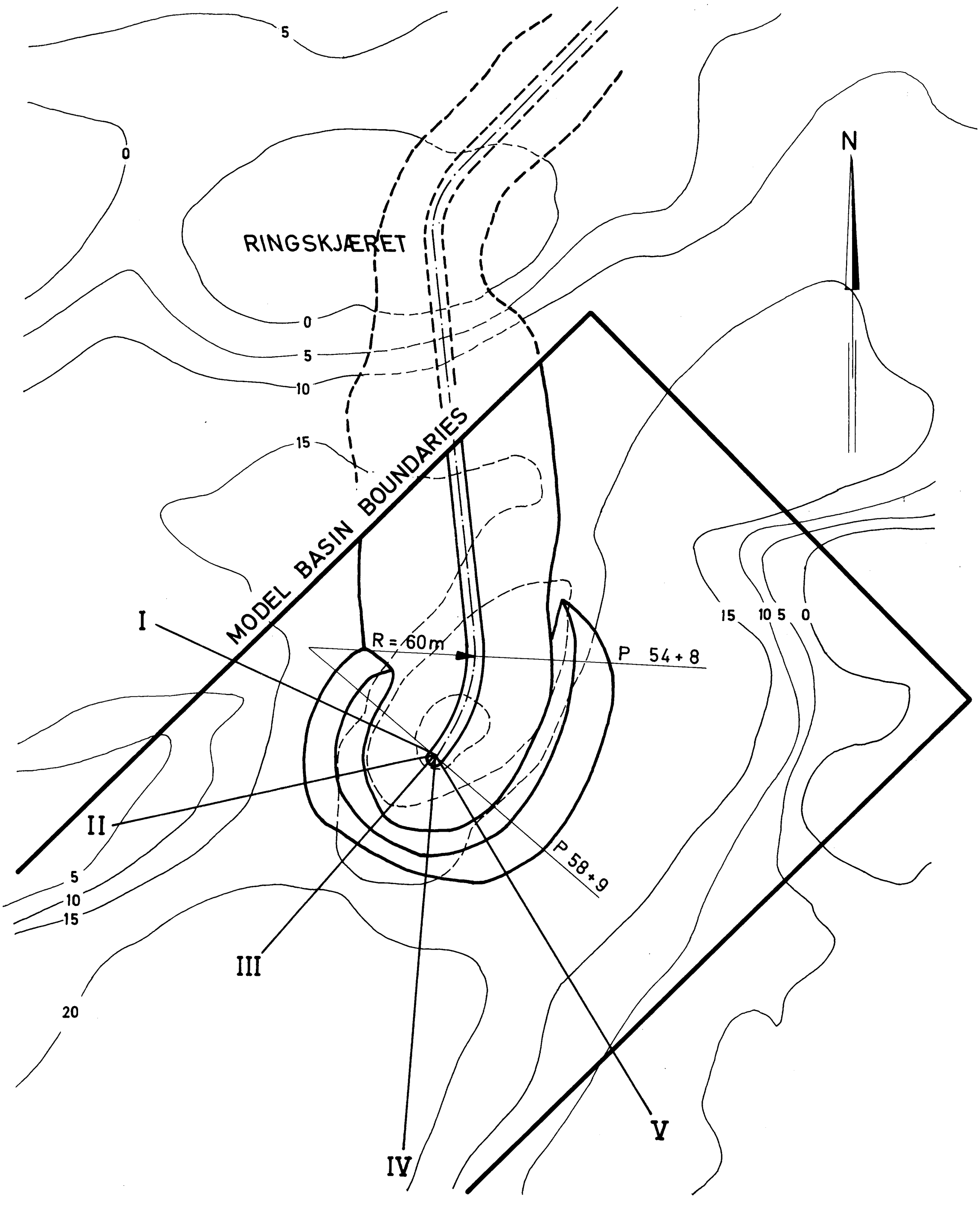


FIG. 3. DETAILS OF BOTTOM CONFIGURATION AND LOCATION OF BREAKWATER HEAD.

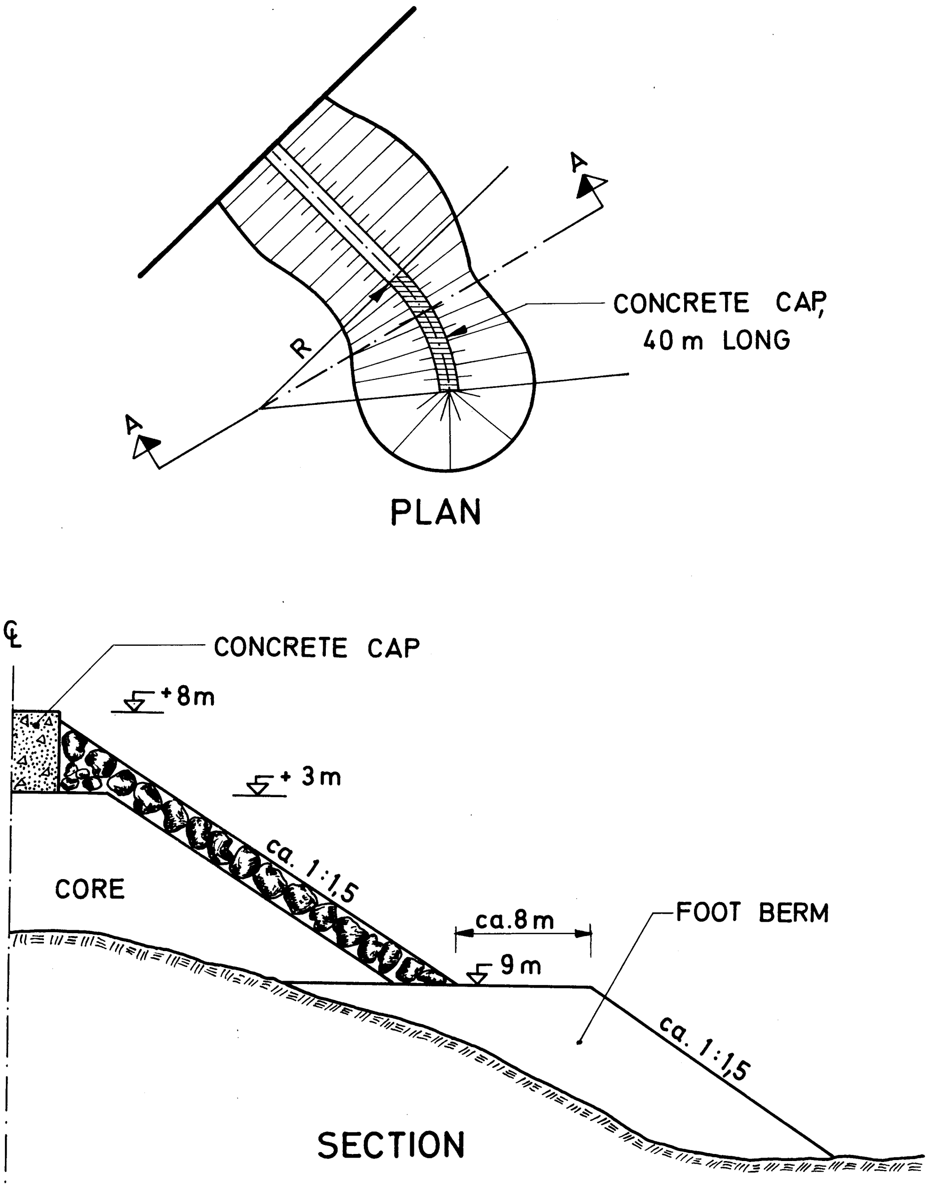


FIG. 4. SKETCH OF BREAKWATER HEAD

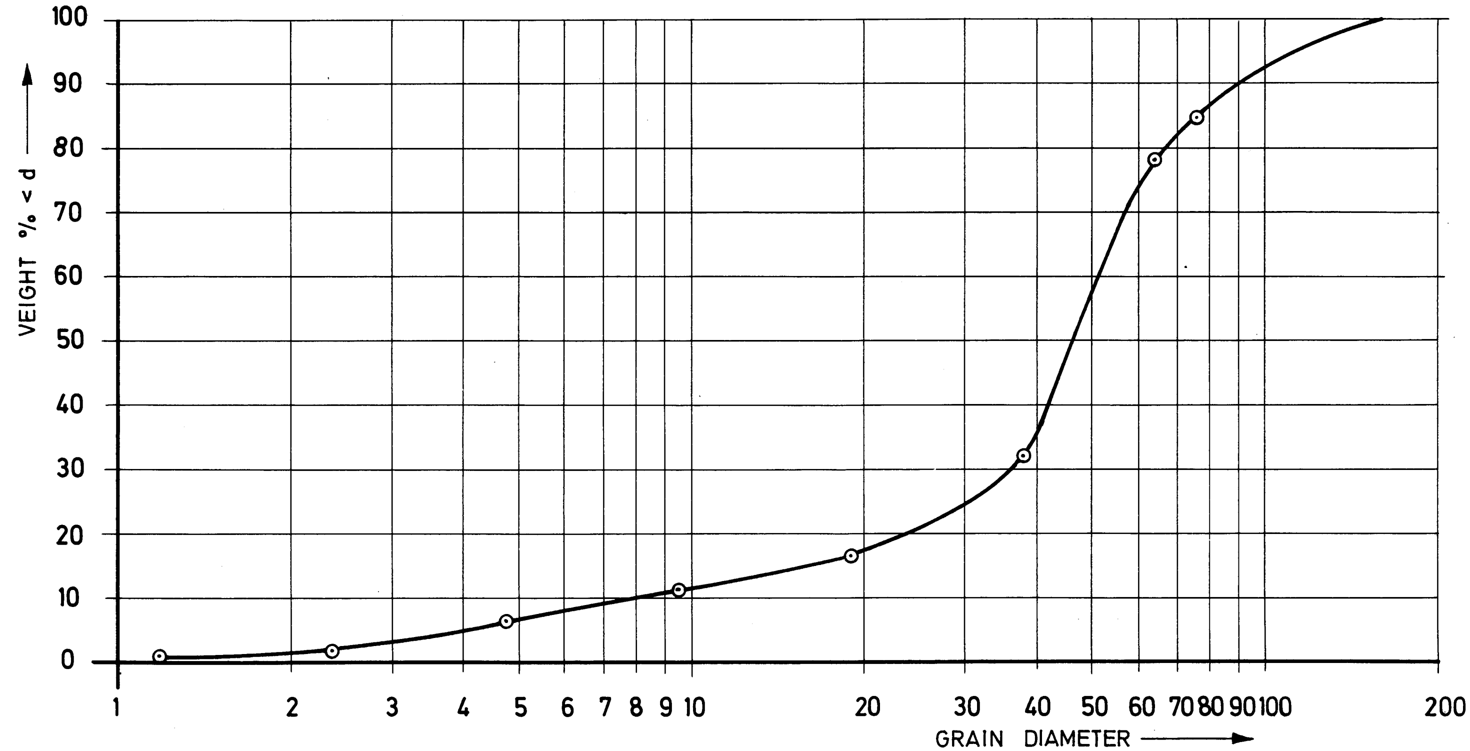


FIG. 5. ANTICIPATED GRAIN SIZE DISTRIBUTION OF CORE MATERIAL

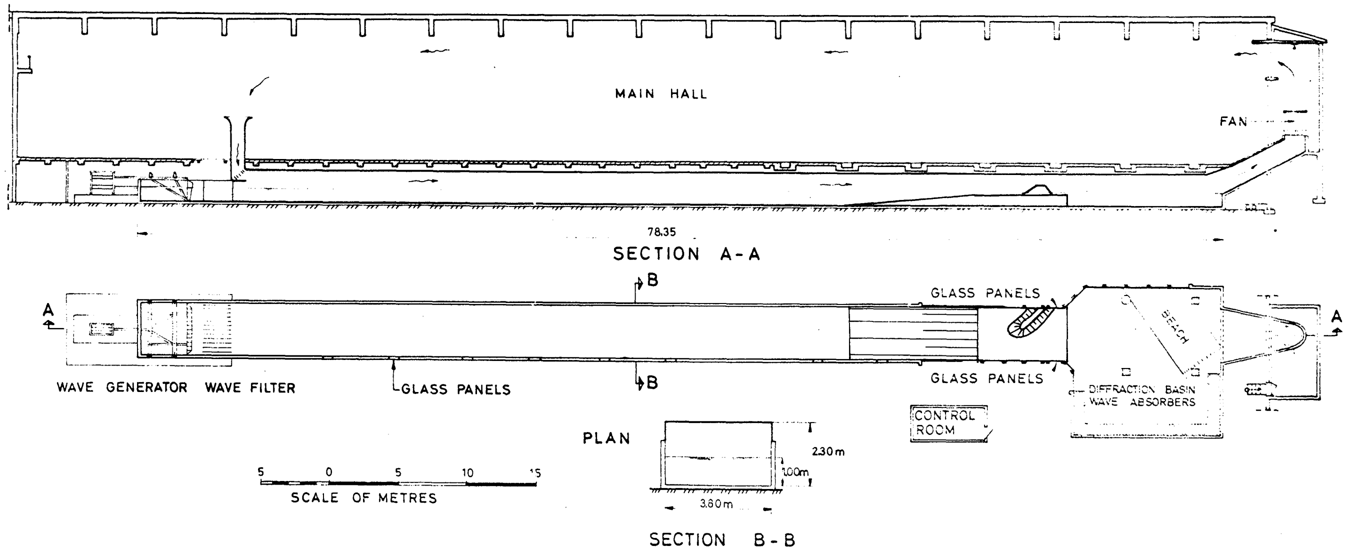


FIG. 6. WAVE FLUME

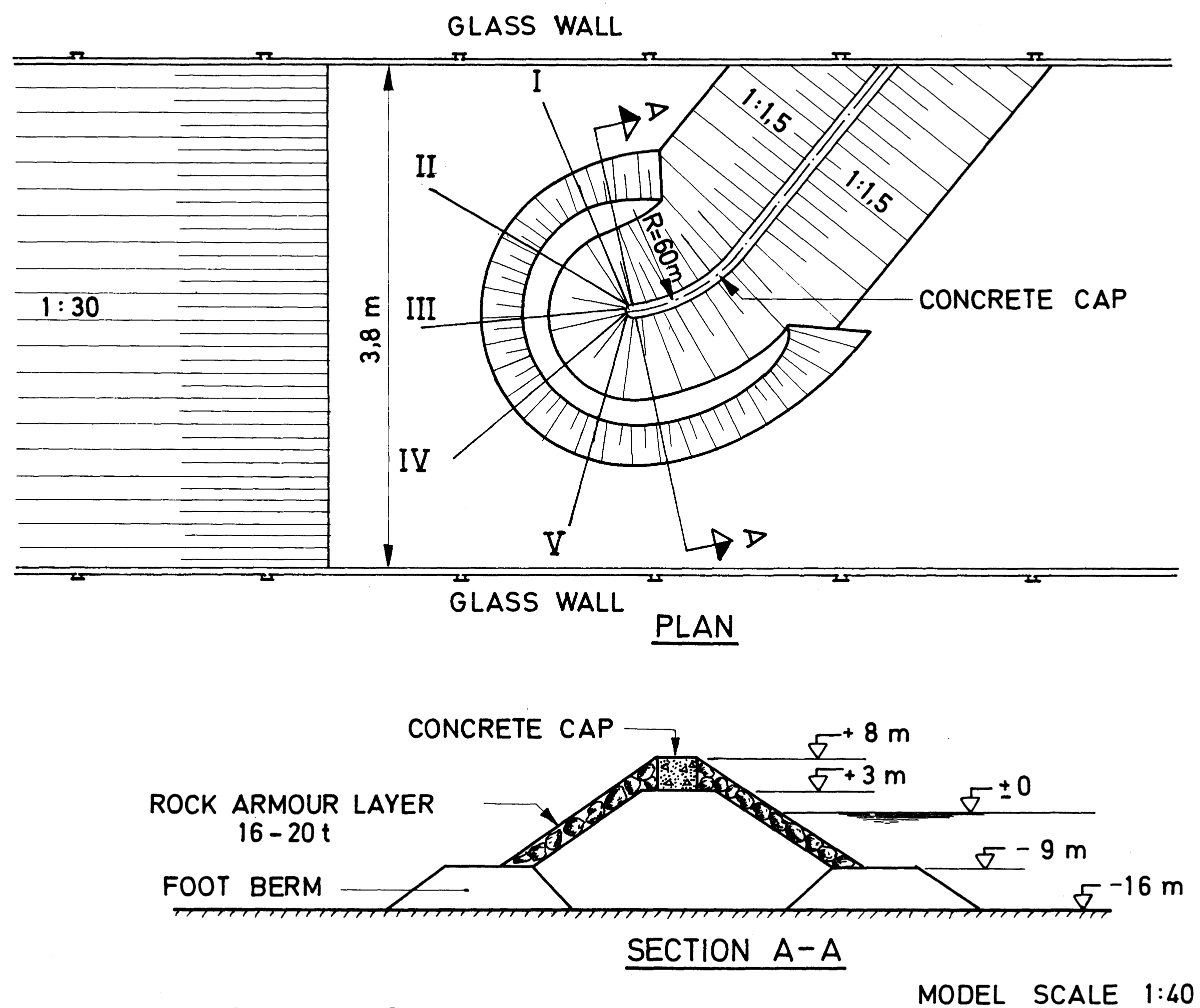
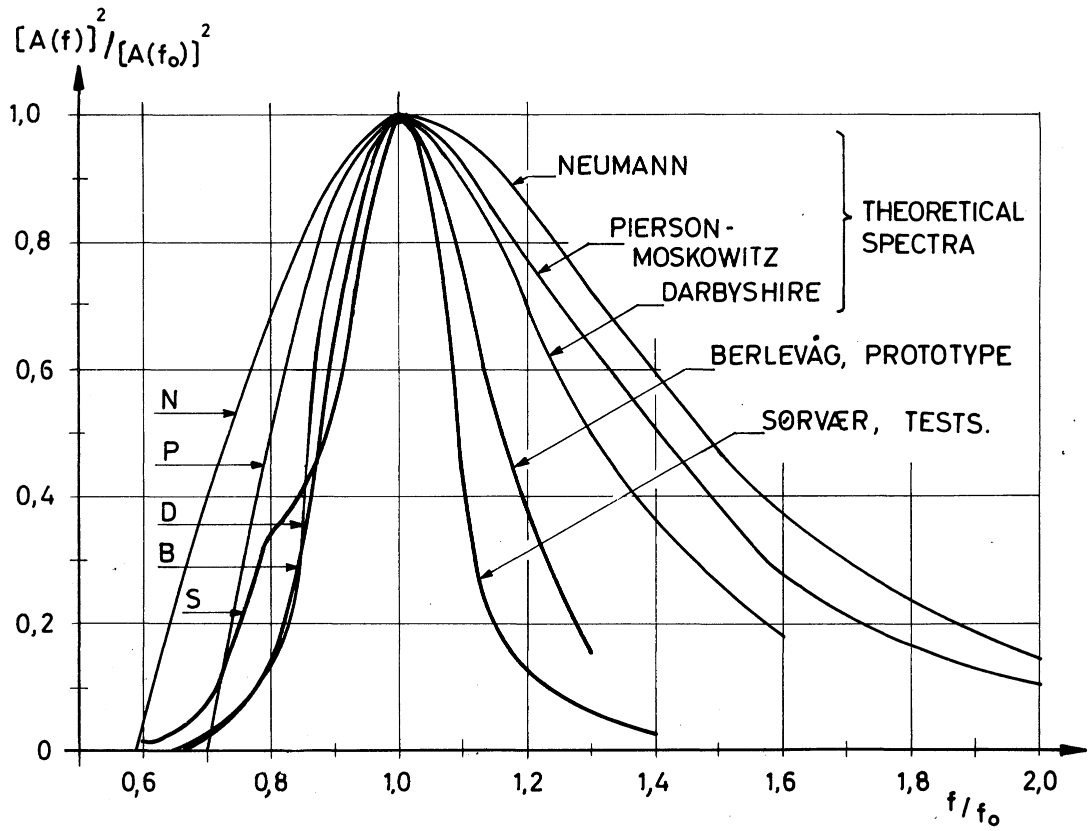


FIG. 7. MODEL TEST SET-UP.



$[A(f_o)]^2$ - WAVE ENERGY DENSITY AT THE PEAK ENERGY DENSITY.

f_o - WAVE FREQUENCY FOR PEAK ENERGY FREQUENCY.
$f_o = 0{,}7$ 1/sec. IN THE MODEL, CORRESPONDING TO $T_O = 9$ sec IN THE PROTOTYPE.

FIG. 8. NORMALIZED WAVE SPECTRA.

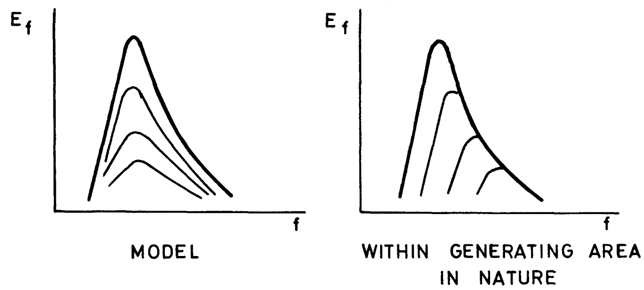


FIG. 9. INCREASE IN WAVE ENERGY IN PROTOTYPE AND MODEL.

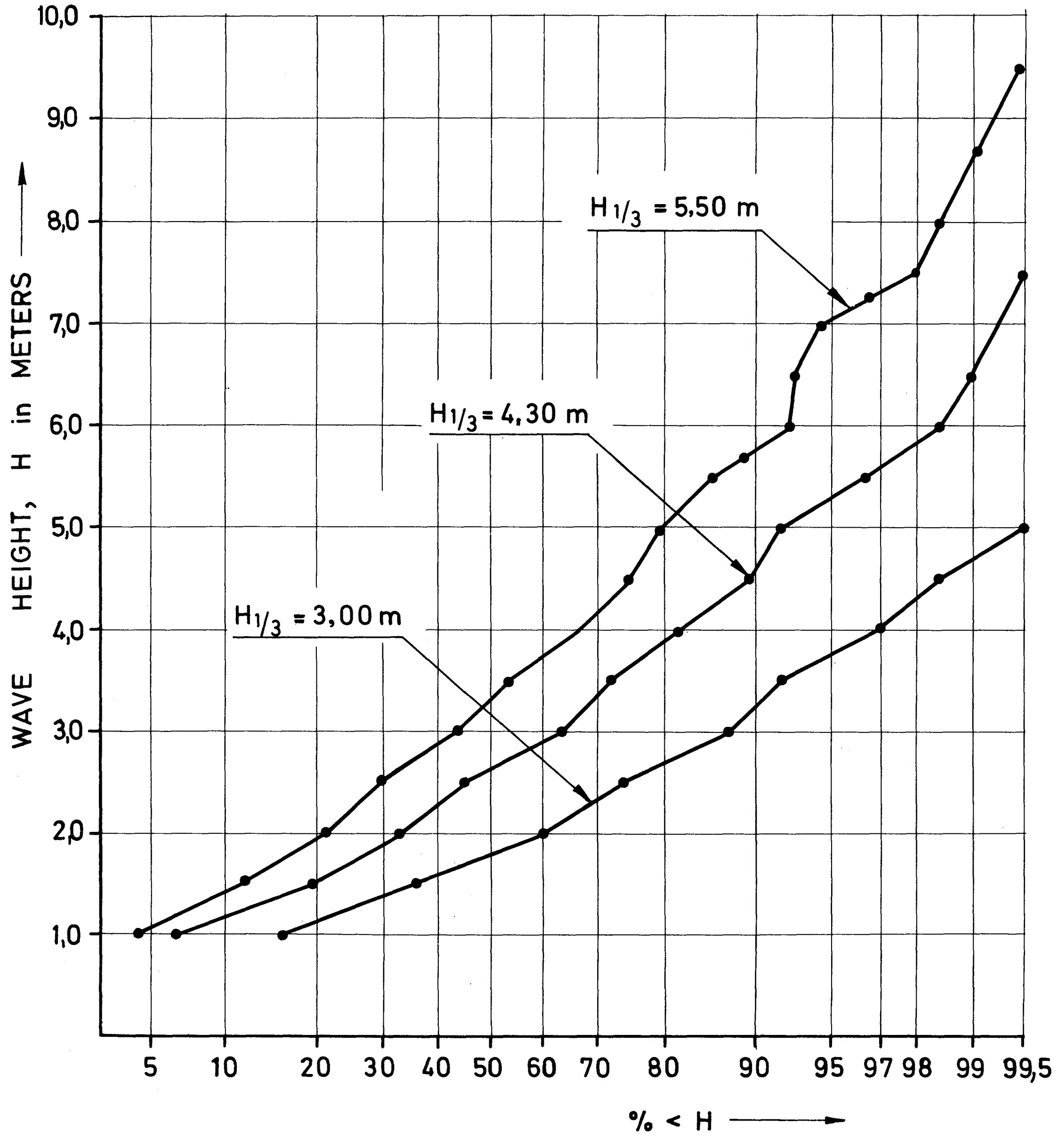
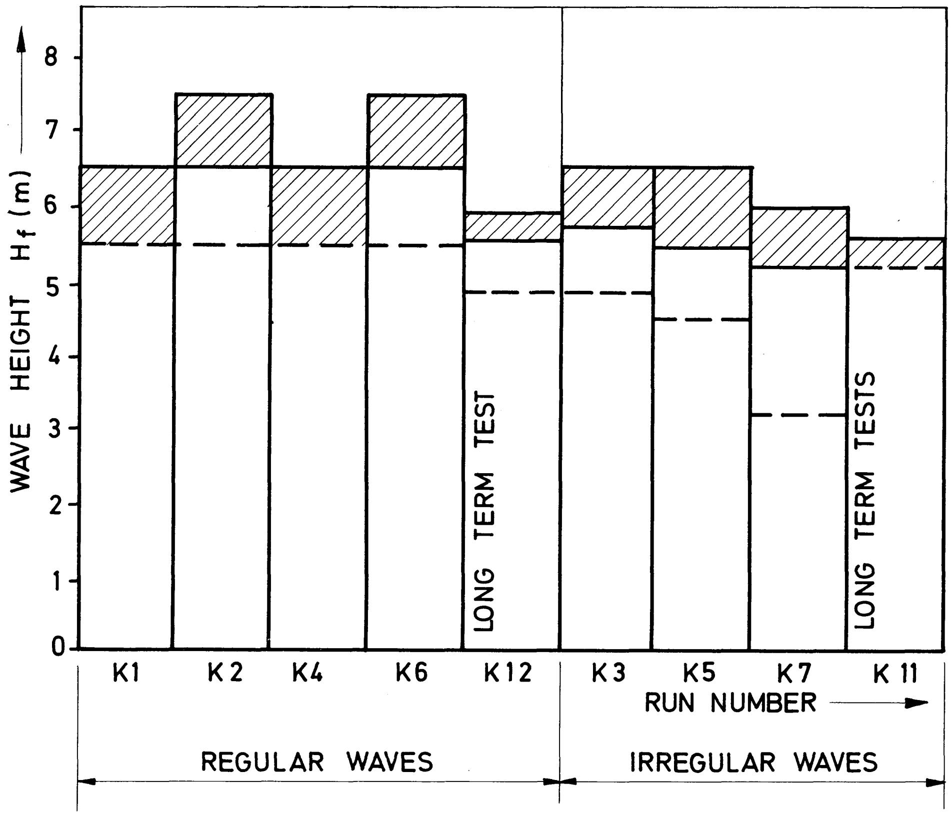


FIG. 10. WAVE HEIGHT DISTRIBUTIONS IN THE DEEP PART OF THE WAVE FLUME.



——— WAVE HEIGHT AT UNACCEPTABLE DAMAGE

– – – – – WAVE HEIGHT WHEN THE FIRST ARMOURBLOCK WAS REMOVED.

FOR IRREGULAR WAVES $H_f = H_{1/3}$ (SIGNIFICANT WAVE HEIGHT.)

FIG. 11. TEST RESULTS

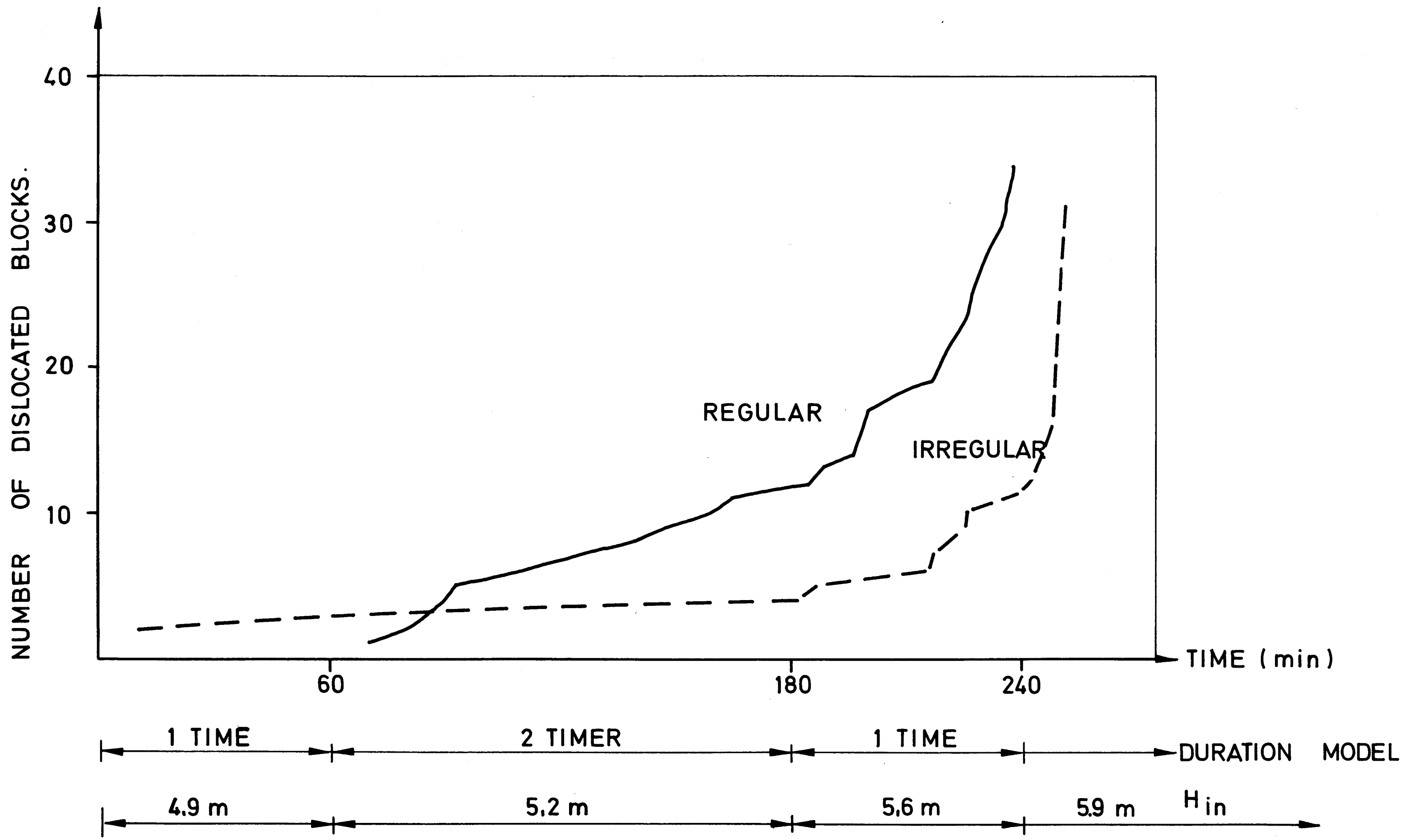


FIG. 12. DAMAGE CURVES, LONG TERM TESTS

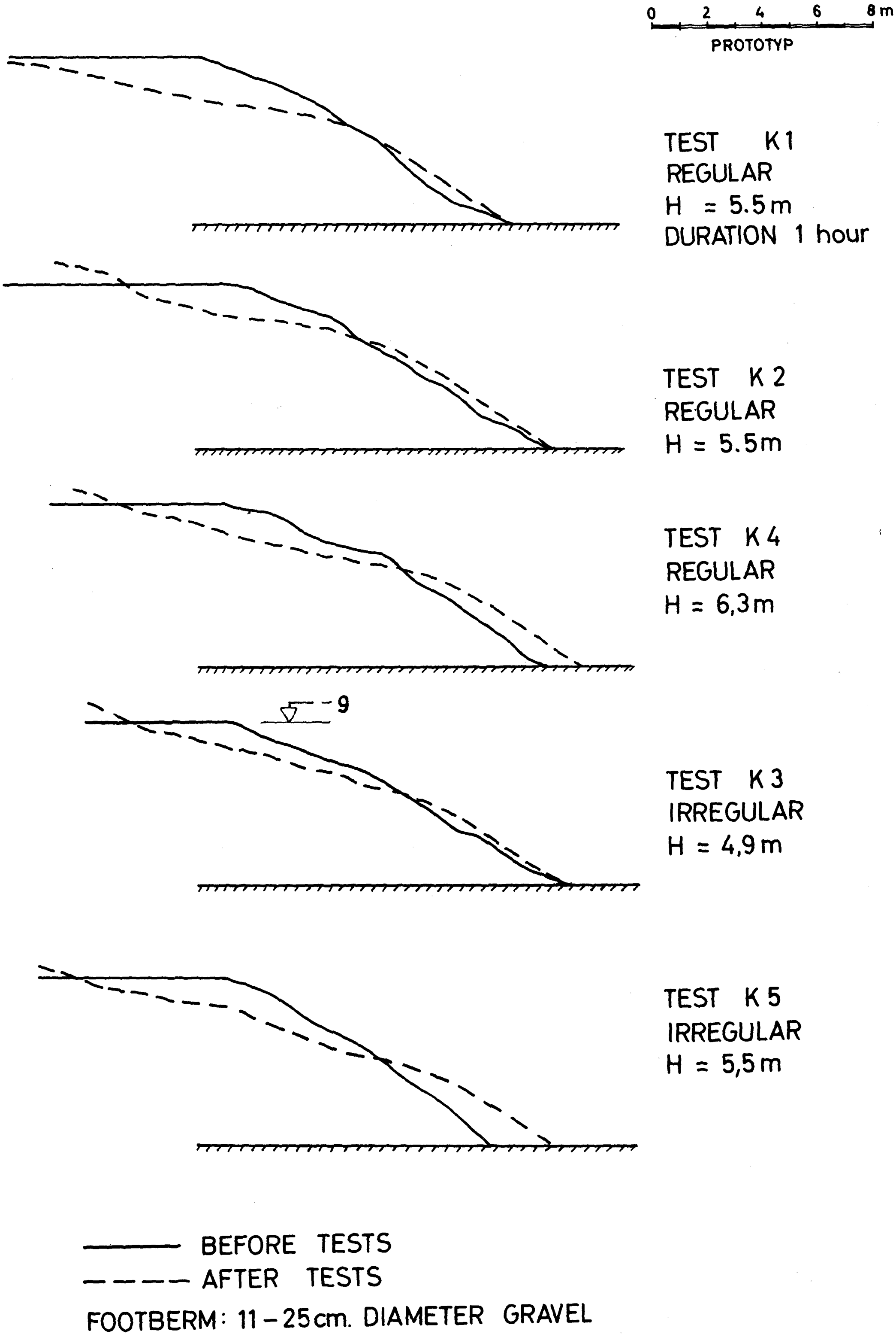
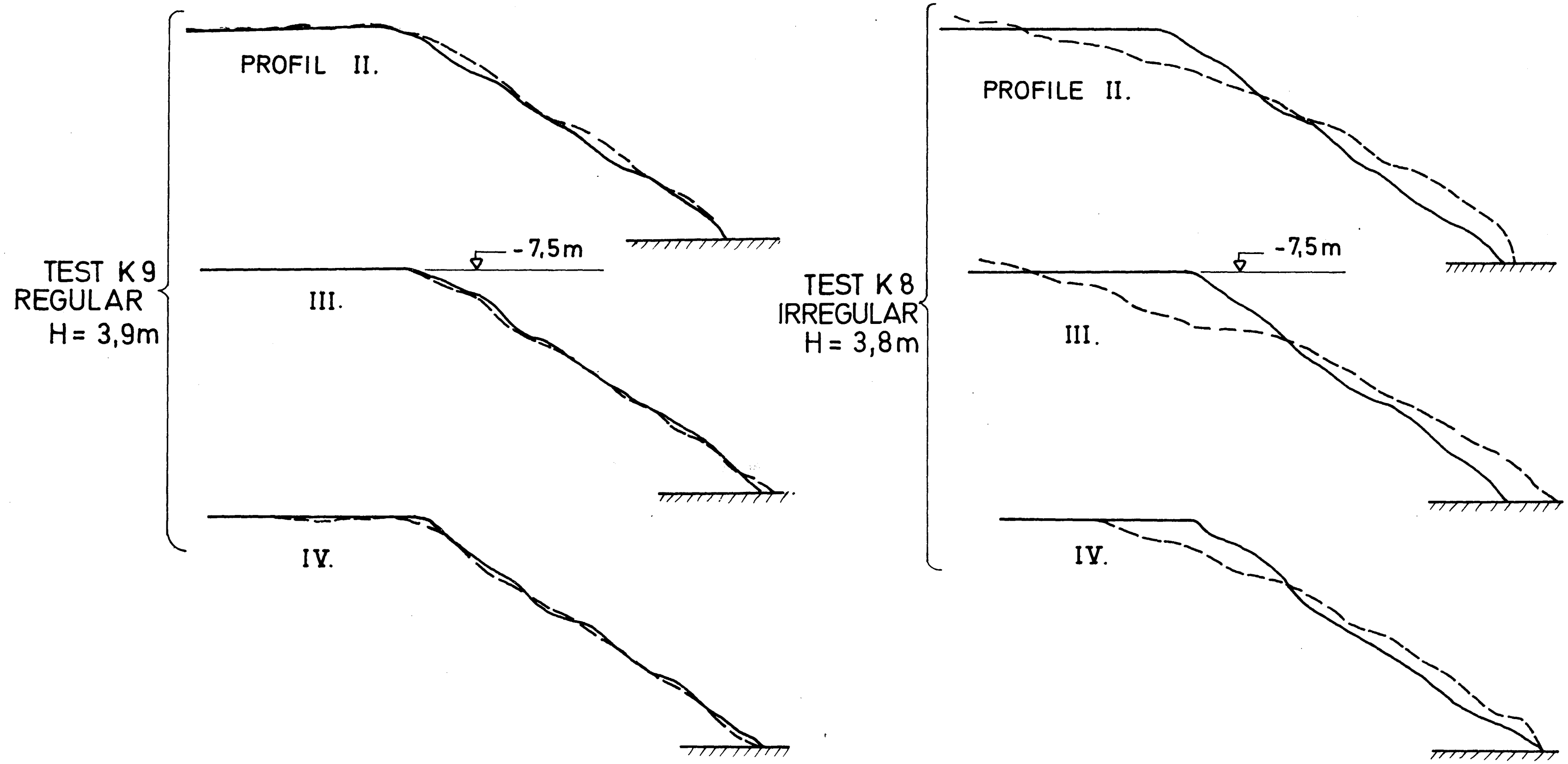


FIG. 13. PROFILE III OF FOOTBERM



——— BEFORE TESTS.
– – – – AFTER TESTS.

FOOTBERM : 11 – 12 cm. DIAMETER GRAVEL
FROM BOTTOM TO -9,0m
CORE MATERIAL FROM -9,0 TO -7,5m

FIG. 14. PROFILES OF FOOTBERM