# CHAPTER 135

## Influence of the core configuration on the stability of berm breakwaters

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Abstract

An experimental study has been carried out to investigate the concept of extending the core into the berm of a berm breakwater. Five different core configurations were tested. The reshaped profiles were not significantly influenced by the core configuration. The core configuration should be such that the core material is not directly exposed to the waves during the reshaping.

The concept of extending the core into the berm will lead to cheaper berm breakwater structures since the core material is cheaper than the armour stones.

## Introduction

Berm breakwaters was introduced as a economical breakwater concept in the beginning of the 1980'ties. Baird and Hall (1987) discussed the main advantage of the concept. The main advantage of the berm breakwater concept is that smaller stone weights can be used in a berm breakwater than in a conventional rubble mound breakwater. Hence general contracting equipment can be used instead of more costly speciality equipment.

Several investigations have been carried out on the stability of berm breakwaters. van der Meer (1988) have carried out the most comprehensive study

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on the stability of static and dynamic stable breakwaters, including the berm breakwater. Burchart et al (1988) carried out tests on berm breakwaters in which they concentrated mainly on 3-dimensional effects, i e transport of stones along the structure exposed to oblique waves and deformation of roundheads.

Much research on berm breakwaters was carried out during the EU MAST I (1992-1994) and MAST II (1994-1996) program. The present study was initiated by the second author when he participated in MAST I. Up to that time most of the systematic laboratory work had been carried on berm breakwaters as shown in Figure 1A. A cross section as shown in Figure 1B could be more economical since more of the (cheaper) core material can be used. The experimental study on the influence of the core configuration on the berm breakwater stability was reported in detail by Lissev (1993). This paper is a summary of the report.

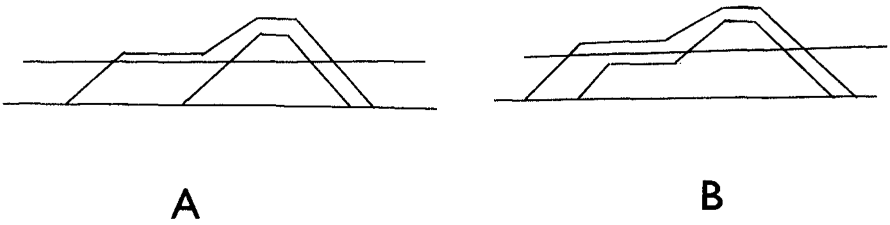


Figure 1 Berm breakwater cross-sections

Test set-up

The model testing was carried out in a wave flume as shown in Figure 2. The width of the flume is 1.0 m.

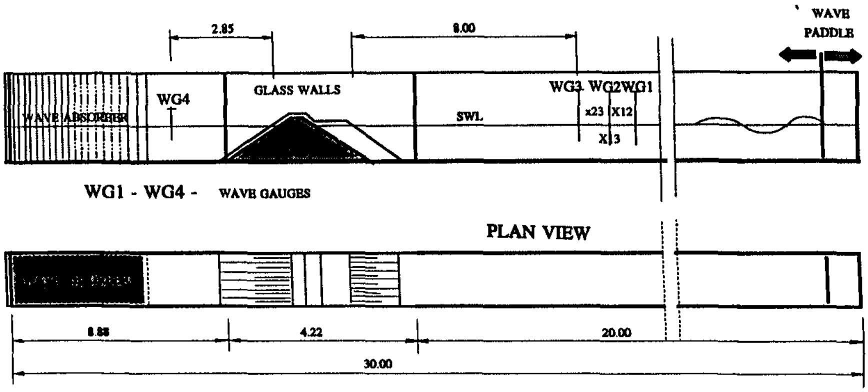


Figure 2 Wave flume with berm breakwater model

Five different cross sections as shown in Figure 3 were tested.

The gradation curves for the berm stone material and the core material are shown in Figure 4. The nominal diameter is defined as $D = \sqrt[3]{W / \rho_s}$ , where $W$ = mass of the stone, $\rho_s$ = specific mass of the stone.

Wave measurement

The surface elevations were measured with conductive type wave gauges. In order to be able to sort out the incoming and reflected waves the three gauges were placed with individual distances as recommended by Mansard and Funke (1980).

The group of three gauges were placed 8 m in front of the breakwater. This is more than one wave length, as recommended by Goda (1985).

To decompose the wave field into incoming and reflected waves a method based on linear wave theory and developed by Zelt and Skjelbreia (1992) was used.

Figure 5 shows reflection coefficients as obtained for profiles 1, 2 and 3 compared with coefficients obtained by Andersen et al (1992).

Test program

As mentioned five initial profiles with the same outer profile configuration were used, but with different configuration of the core, Figure 3. Two types of model tests were conducted for profiles 1, 2 and 3: Stability tests and tests for measuring some of the hydraulic responses of the developed profile of the breakwater after finishing the stability tests. For the initial profiles 4 and 5 only stability tests were conducted.

The tests were carried out with irregular waves with increasing peak period $T_p$ with increasing significant wave height such that the wave steepness $s_{02} = 2\pi \quad H_s / gT_{02}^2$ was kept approximately constant, $s_{02}$ = 0.045. The lowest peak period was $T_p$ = 1.32 s and the highest peak period $T_p$ = 2.8 s. $H_s$ = significant wave height, $T_{02} = 2\pi\sqrt{m_o / m_2}$ , $m_0$ = area under the wave spectrum, $m_2$ = second area moment of the wave spectrum.

The time duration of the different significant wave height steps are shown in Figure 6. Table 1 shows significant parameters for the tests.

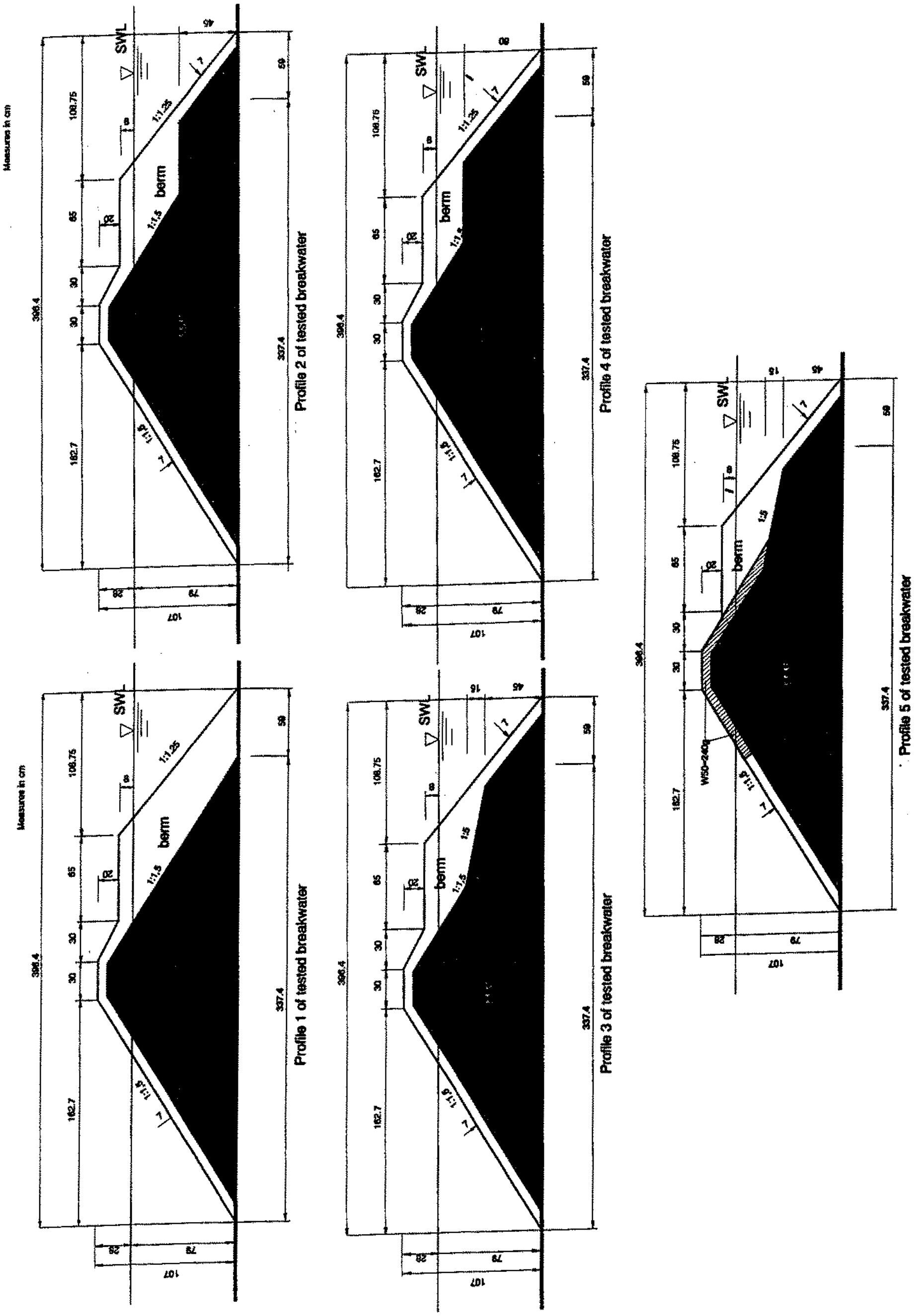


Figure 3 The five tested cross sections

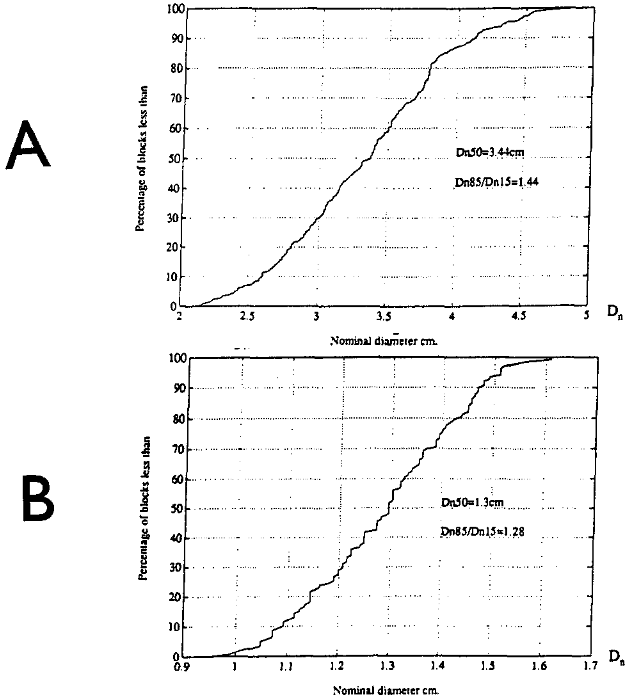


Figure 4 Gradation curves A) Berm stones B) Core stones

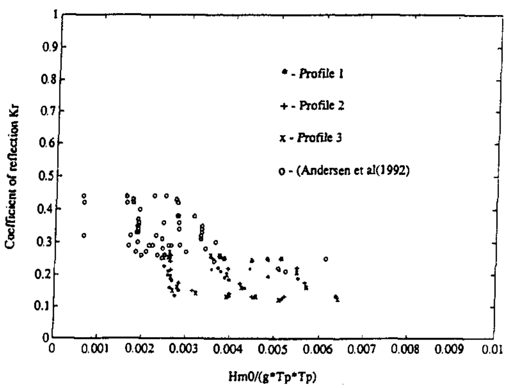


Fig 5 Reflection coefficients

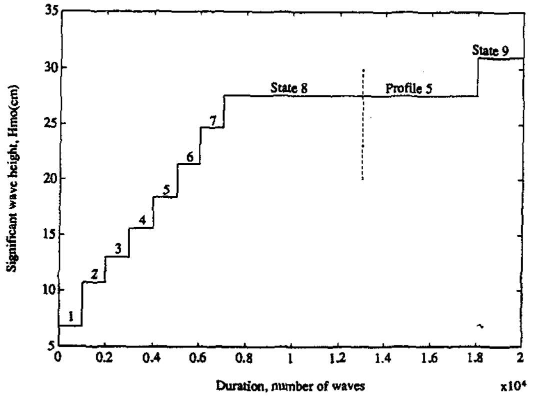


Figure 6 Time duration of the significant wave height

Table 1 Tested sea states

| State no. | Tp (sec) | $T_{02}$ (sec) | $H_{m0}$ (cm) | $S_{02}$ | $H_{m0}/D_{n50}\cdot\Delta$ |
|---|---|---|---|---|---|
| 1 | 1.32 | 1.05 | 6.8 | 0.040 | 1.18 |
| 2 | 1.55 | 1.24 | 10.7 | 0.044 | 1.86 |
| 3 | 1.75 | 1.36 | 13.0 | 0.046 | 2.26 |
| 4 | 2.00 | 1.51 | 15.6 | 0.044 | 2.72 |
| 5 | 2.20 | 1.65 | 18.4 | 0.044 | 3.20 |
| 6 | 2.40 | 1.74 | 21.4 | 0.046 | 3.72 |
| 7 | 2.60 | 1.87 | 24.7 | 0.046 | 4.30 |
| 8 | 2.80 | 2.00 | 27.6 | 0.046 | 4.80 |

## Test results

Reshaped profiles after attacks of 1000 waves for each wave state between state 4 and state 8 and for every one of the five tested initial profiles are shown in Figure 7. The most significant changes for every wave state took place after the attack of the first two-three hundred waves for every wave state. For wave states 4, 5 and 6 only very small changes of the profile were observed after this time.

In Figure 8 are shown the reshaped breakwater profiles for all five initial profiles measured after finishing the tests with 1000 waves for every wave state 1-8. In Figure 9 are shown the final profiles after completing the tests with additional 5000 waves for wave state 8 for every of the five initial profiles.

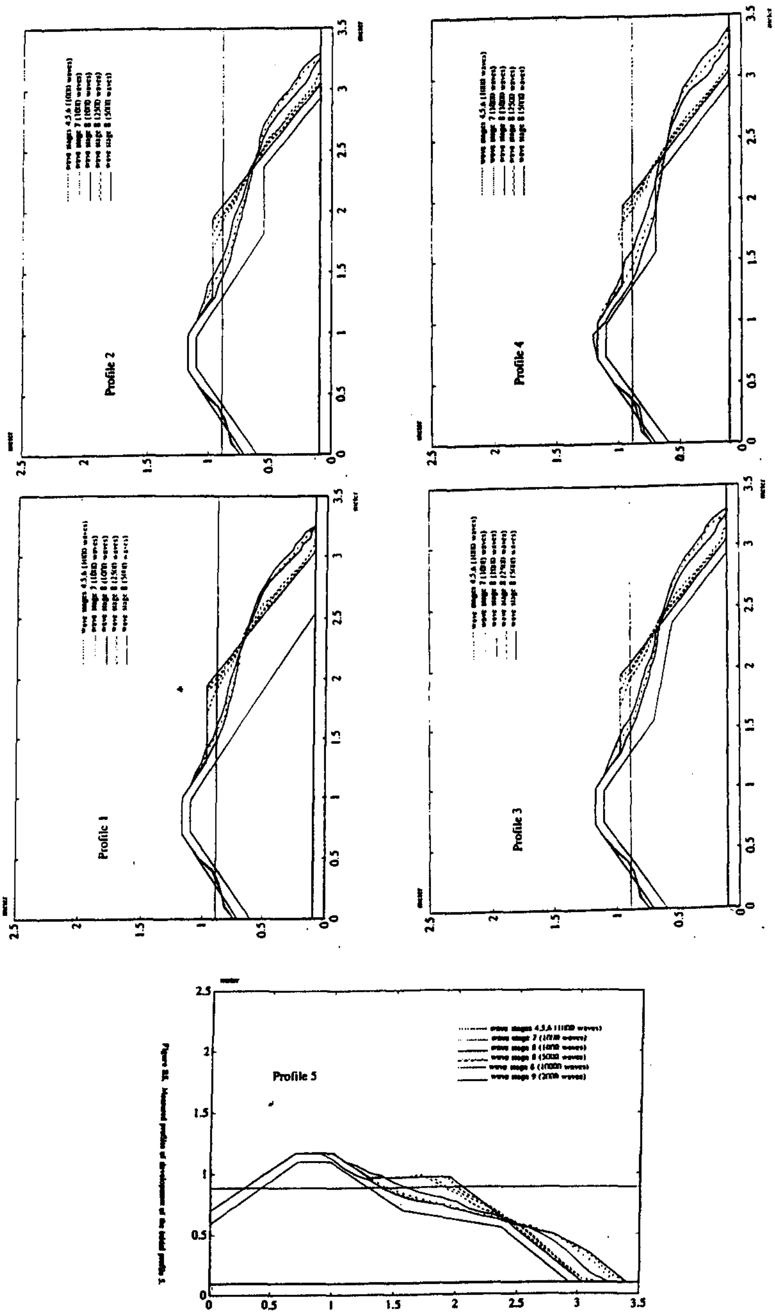


Figure 7 Reshaped profiles for the different initial profiles

Figure 8 shows that there are no significant differencies for all five tested initial profiles after completing the sea wave states 1-8 with 1000 waves in each state. The profiles are almost identical with small deviations at the upper and lower berm slope. In this case the width of the berm was reduced from 0.65 m to 0.10 - 0.13 m. This means that for no one of the tested profiles seaward damage conditions are reached. The damage to the seaward side of the berm breakwater is defined to occur when the entire width of the top berm is eroded.

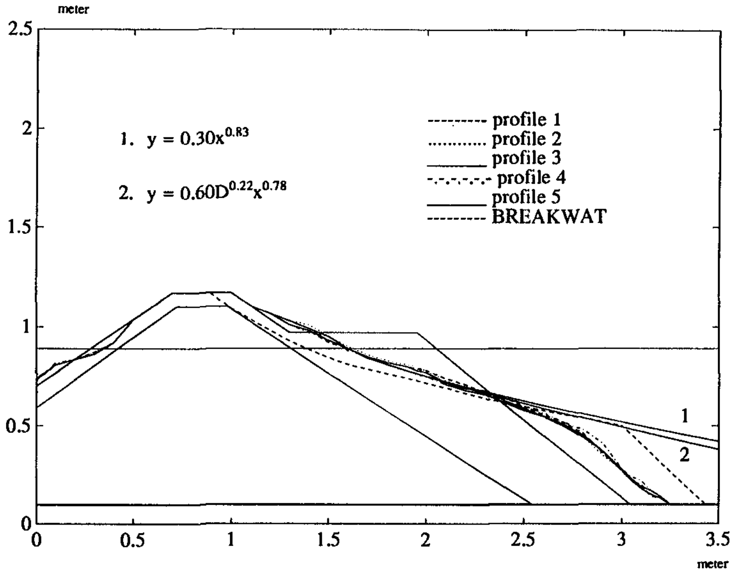


Figure 8 Reshaped profiles for every initial profiles 1-5 after completing every wave state 1-8 with 1000 waves for each state.

For profile 4 there was another critical point close to the dynamic stable profile. The berm stones were removed at this point and the core became directly exposed to the waves after completing the test with sea states 1-8 with 1000 waves in each sea state. Even though a critical condition was reached for profile 4 then it was decided to continue the experiments with the 5000 waves for wave state 8 for this profile to see if a complete failure would develope.

The experiments with additional 5000 waves for wave state 8 show almost the same reshaping for the initial profiles 1, 2 and 3. At the end of the tests for all these three profiles the width of the horizontal berm was reduced to zero . The results from the tests with the initial profiles 1, 2 and 3 gave the idea to use somewhat larger stones over the breakwater crest, Profile 5, Figure 3. Profile 5 is a modified version of profile 3. The medium weight of the stones over the crest is $W_{50} = 0.24$ kgs, which is approximately twice the weight of the stones in the berm.

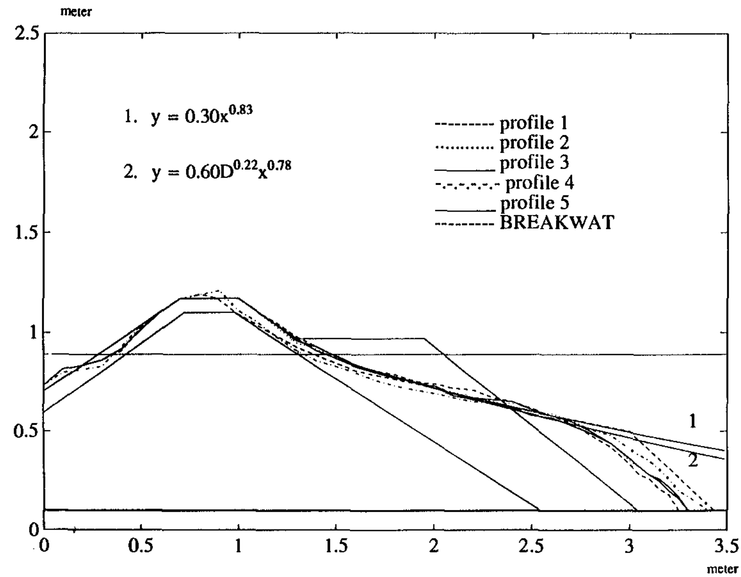


Figure 9 Reshaped profiles for every initial profiles 1-5 after completing every wave state 1-8 with 1000 waves for each state plus additional 5000 wave for wave state 8.

The development of Profile 5 up to 5000 waves is similar to the profiles 1, 2 and 3. For Profile 5 an additional 5000 waves of wave state 8 were run. During these 5000 waves there were only small changes in the upper and lower part of the profile. This means that the profile is dynamically stable for this wave state. Only some few stones were moved from the additional armour layer to the toe of the breakwater.

After completing the tests with wave state 8 for Profile 5, the wave height was increased to wave state 9, $H_s = 0.31$ m and $T_p = 2.8$ s. After 2000 waves with this wave state the armour layer was almost completely destroyed and the test was stopped. In this wave state the dynamic stable profile in the middle part was the same. The main development was in the upper and lower part of the berm slope. Bigger stones from the upper part of the profile were moved to the toe of the breakwater.

For Profile 4 the "failure" which started during the first 1000 waves with wave state 8 increased with the number of waves. Part of the core which was directly exposed to the wave action migrated gradually towards the crest of the breakwater. It is important to point out that this process developed gradually with time. Even without armour stones on the flat part, the profile was relatively stable because stone prism formed in the front part of the profile supporting finer core material. This is a good example that processes of destruction of the berm breakwaters are relatively slow in comparison with conventional mound breakwaters where failure is very fast after some critical stage has been reached.

The curves marked BREAKWAT in Figure 8 are results obtained with the computer program BREAKWAT developed by Delft Hydraulics (1990) based on the extensive tests by van der Meer (1988).

The curves in Figure 8 represented by the equations

$$y = a_4 x^{0.83}$$

$$y = p \cdot D^{0.22} x^{0.78}$$

comes from van der Meer (1988) and a modified version of Vellinga's (1986) equations for dune erosion during storm surges.

There is fair agreement between the results obtained in this study and previous studies.

## Overtopping

Tests to determine irregular wave overtopping were carried out on the reshaped breakwater after the stability tests had been completed for Profiles 1 and 3. Figure 10 shows the test set-up to measure overtopping.

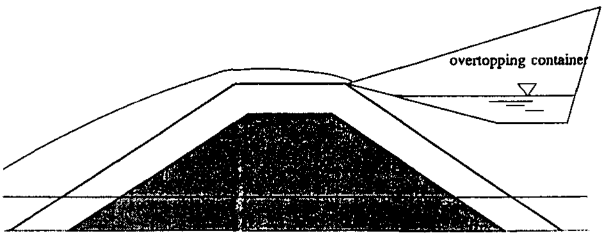


Figure 10 Scheme of measuring overtopping

We used the dimensionless freeboard parameter $F'$, Ahrens (1986):

$$F' = \frac{F}{\left(H_s^2 L_p\right)^{1/3}}$$

where $F$ = freeboard, i e the vertical difference between the crest height of the berm breakwater and the still water line, SWL. $L_p$ is the Airy wave length based on the peak period $T_p$ and the water depth in front of the breakwater.

The overtopping rate $Q$ is defined as the volume of water overtopping the breakwater per unit length of breakwater per unit time.

In Figure 11, $Q$ is plotted versus $F'$. The curve shown in Figure 11 represents an equation of the general form due to Owen (1982):

$$Q = Q_o \exp\left(C_1 F'\right)$$

The curve shown in Figure 11 is an eyefitted curve. The coefficients $Q_o$ and $C_1$ leave in this case the values $Q_o$ = 4600 $cm^3/cm$ s and $C_1$ = -21.

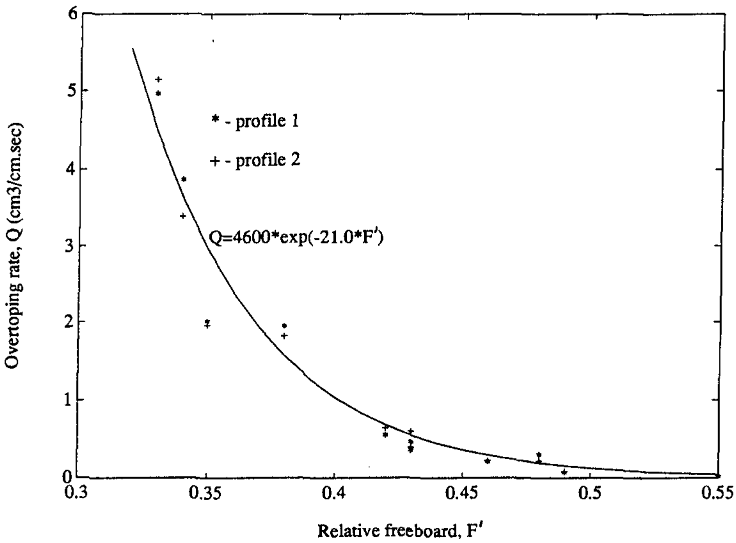


Figure 11 Overtopping rate versus relative freeboard

## Rear side damage

No special tests for rear side stability were included in this study. We only registered the beginning of the damage processes on the rear side. Some comparisons were made with Andersen et al (1992) on rear side damage. There was a fair agreement between the two test series.

## Conclusion

Based on the results obtained in this study it can be concluded that the core can be extended into the berm of a berm breakwater. Since the core material is generally cheaper than the armour stones, the concept of extending the core material into the berm will give a cheaper berm breakwater structure.

## Acknowledgement

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