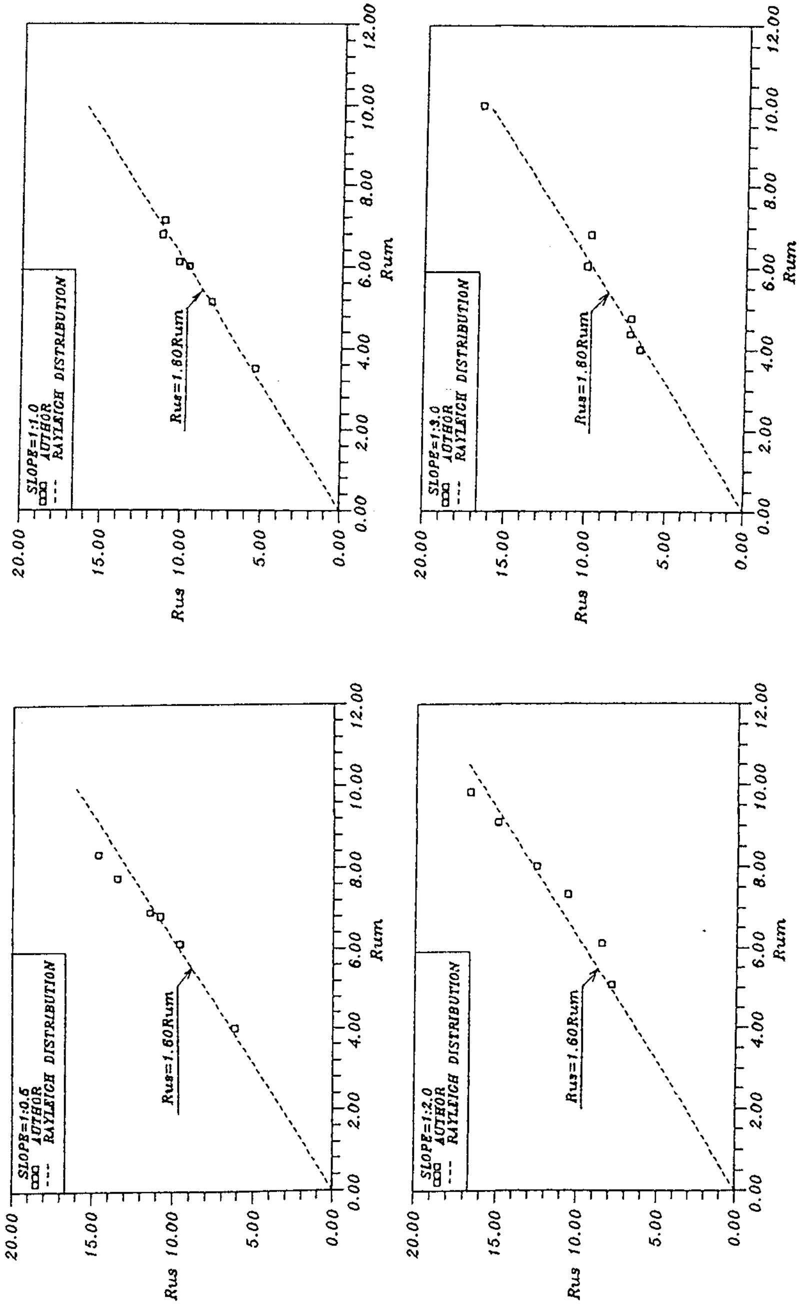

Fig.6 Relationship between significant runup height and mean runup height in smooth dike

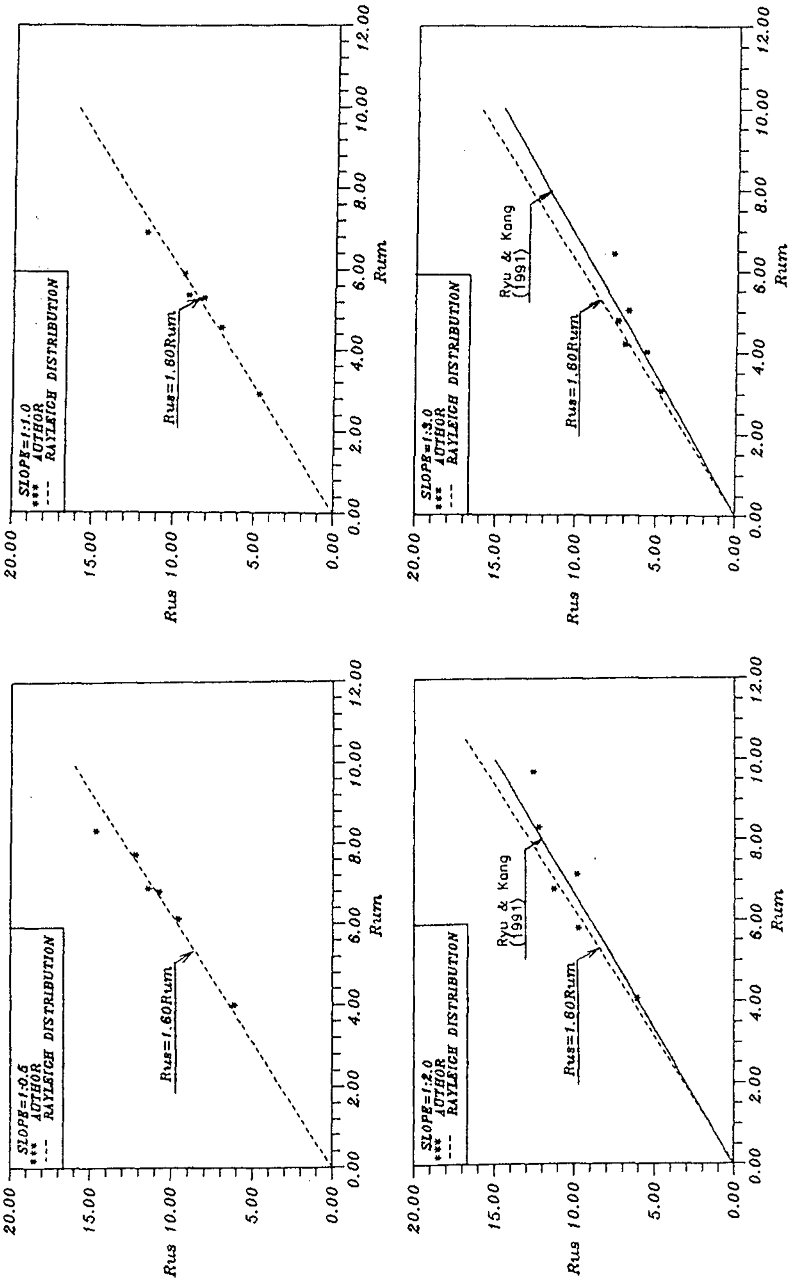

Fig.7 Relationship between significant runup height and mean runup height in rough dike

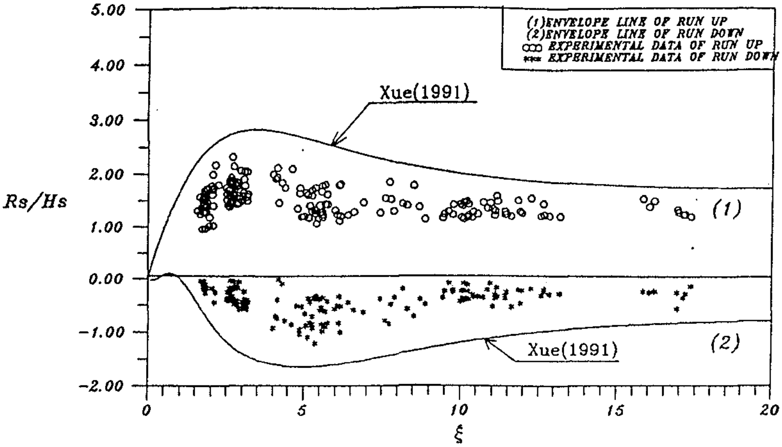

Fig.8 Relationship between run-up-down height and surf parameter

ther, we can find the rougher the surface of the dyke slope, the lower the relative runup height decreased as shown in Fig.10. When the roughness coefficient approach to 0.1212, the corretation curve between the relative runup and the surf parameter was very similar with those results which was presented by Ryu in 1990 in rubble mound experiment.

At last, the normalized relationship between the relative run-up and rundown height and the surf parameter was shown in Fig.11 and 12 respectively. The meaning of normalized stand for the ratio of relative run-up(down) height in rough dyke compared with those in smooth dyke. From the figures, we can see that when the surf parameter becomes bigger, that means the dyke slope approach to vertical, the influence of roughness to wave run-up-down height will vanished.

CONCLUSIONS

1. The distribution of the irregular wave runup heights provides a good approximation to the Rayleigh distribution on the smooth surface dyke. But it will be overestimated when the dyke slope milder than 1 to 2 in rough surface test.
2. The effect of roughness to the relative runup height was certainly and the rougher the surface of the dyke slope, the lower the relative runup height will decreased.
3. When the roughness coefficient approach to 0.1212, the relation

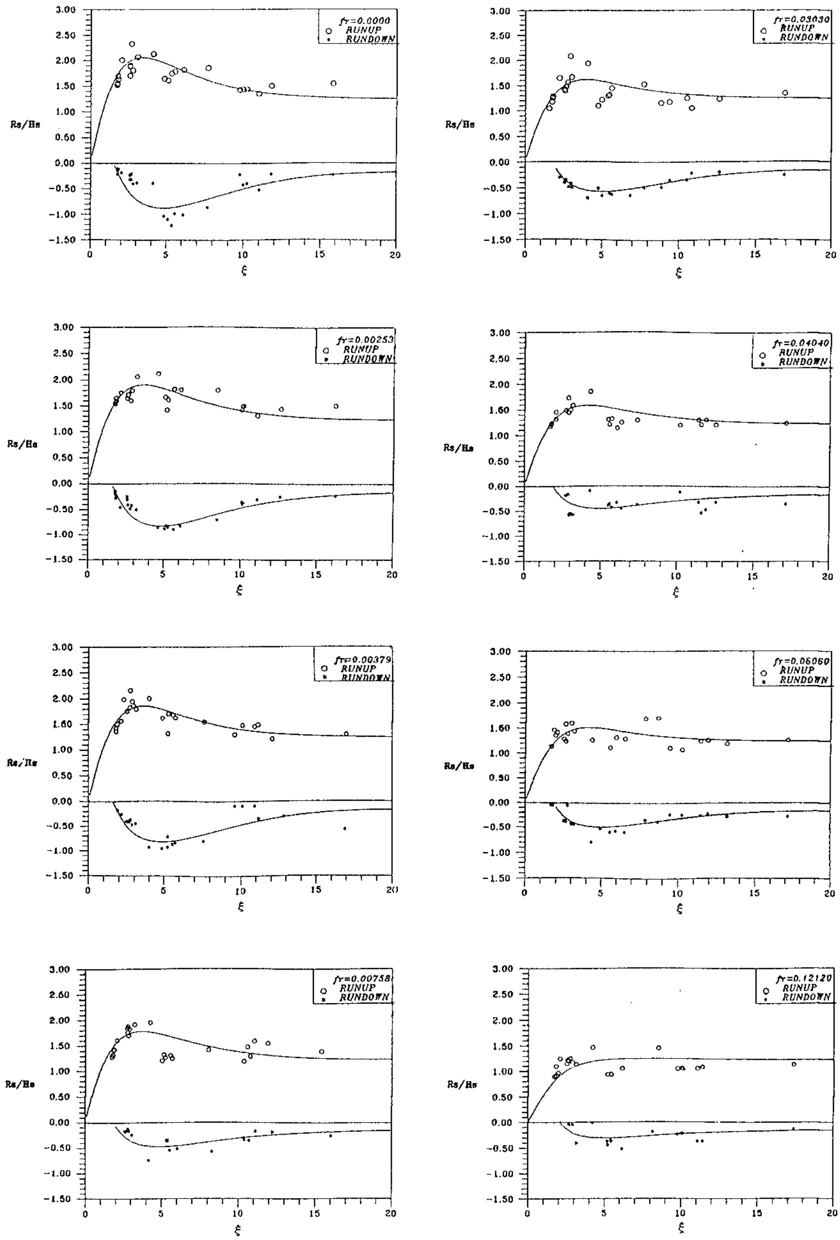

Fig.9 Relationship between significant relative runup height and the surf parameter

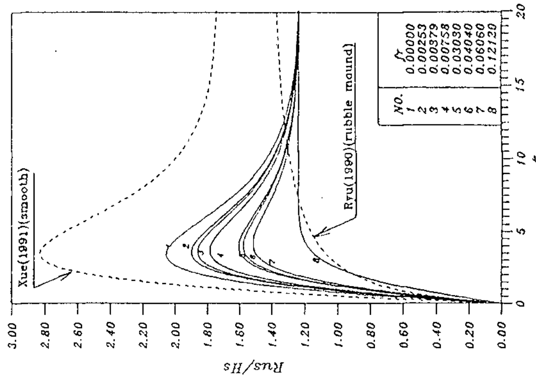

Fig.10 Comparison of relative runup in various rough surface

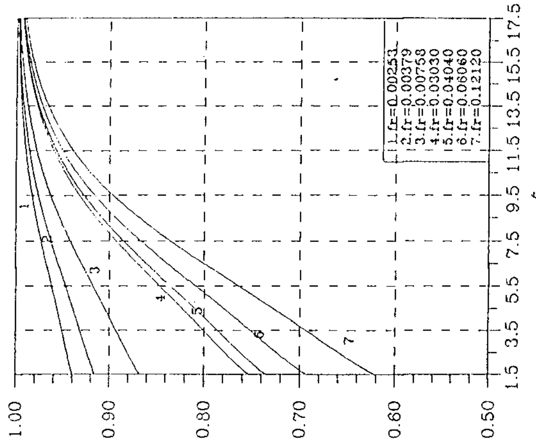

Fig.11 Normalized correlation of runup height to surf parameter

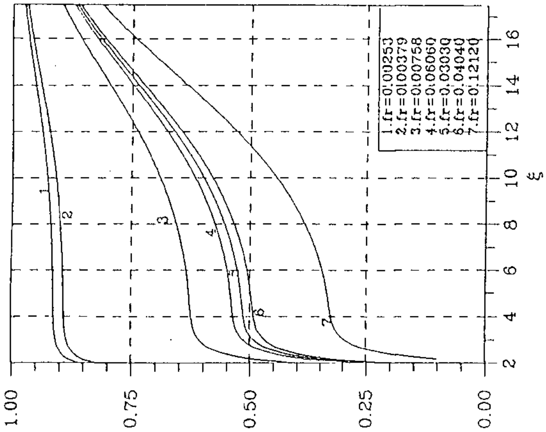

Fig.12 Normalized correlation of rundown height to surf parameter

curve between the relative runup height and the surf parameter was quite similar with those results in rubble mound experiment.

4. When the surf parameter becomes bigger, that's the dyke slope approach to vertical, the influence of roughness to wave run-up-down height will vanished.

ACKNOWLEDGEMENTS

This study was sponsored by the National Science Council of ROC under Grant NSC 81-0209-E124-01. The author would like to thank Mr. J. M. Chou, C. C. Chien and Miss E. J. Chien for their assistance in experiment and preparing the manuscript.

REFERENCES

Cross, R.H. and C.K.Sollitt(1972): Wave transmission by overtopping, Journal of Waterway, Port and Coastal Engineering, Vol.98, No. WW3, PP.295-309.

Goda, Y.(1985): Random seas and design of maritime structures, University of Tokyo Press, PP.1-323.

Juang, J.T.(1992): Effect on wind speed to wave runup, Coastal Engineering, ASCE, Vol.2, PP.1245-1257.

Kobayashi, N., D. T. Cox and A. Wurjanto (1990): Irregular wave reflection and runup on rough impermeable slopes, Journal of Waterway, Port, Coastal and Ocean Engineering, Vol.116, No.6, PP.708-726.

Mase, H.(1989):Random wave runup height on gentle slope, Journal of Waterway, Port, Coastal and Ocean Engineering, Vol.115, No.5, PP. 649-661.

Meer, J.W. and C.M.Stam(1992): Wave runup on smooth and rock slopes of coastal structures, Journal of Waterway, Port, Coastal and Ocean Engineering, Vol.118, No.5, PP.534-550.

Ryu, C.R. and H.Y. Kang(1990): A prediction model of irregular wave runup height on coastal structures, Proc. Coastal engineering, ASCE, Vol.1, PP.371-383.

Xue, H.C., D. Guo, S.H. Pan and H.S.Zhong(1991): Wave runup-rundown amplitude on slopes, China Ocean Engineering, Vol.5, No.1, PP.39-49.

CHAPTER 86

Wave Overtopping of Breakwaters under Oblique Waves

Jørgen JUHL[1] and Peter SLOTH[1]

Abstract

A series of hydraulic model tests has been carried out in a wave basin with the aim of studying the effect of oblique waves on the wave overtopping of traditional rubble mound breakwaters without superstructure. The model tests concentrated on measuring the mean overtopping discharge for wave attacks varying from 0° (perpendicular to the structure) up to 50°. Analyses of the overtopping results were made with respect to the significant wave height, wave steepness, crest free board, crest width and angle of wave attack. The paper describes the influence of these parameters on the mean overtopping discharge for a traditional rubble mound breakwaters with an armour layer slope of 1:2.0.

Introduction

Wave overtopping of coastal structures is influenced by a large number of parameters related to breakwater geometry, construction materials, and hydrographic data. Some of the main parameters are listed below:

Geometrical parameters:
free board, crest configuration and width, slope of armour layer (irregular slope), and water depth

Construction material parameters:
porosity, stone shape and diameter (artificial blocks)

Hydrographic parameters:
wave height, wave period, angle of wave attack, wave steepness, spreading, wave sequences, wind conditions, and water level

[1] Danish Hydraulic Institute, Agern Allé 5, DK-2970 Hørsholm, Denmark

Wave overtopping is normally studied under perpendicular wave attack in wave flumes. Jensen and Juhl (1987) have shown overtopping data from model testing of rubble mound breakwaters and sea dikes. The paper mainly concentrates on mean overtopping discharges, but also includes a description of the horizontal distribution of the overtopping behind a breakwater, individual wave overtoppings and the influence of wind on wave overtopping, and a comparison to prototype measurements.

Franco et al (1994) have established a formula for the mean overtopping discharge for vertical breakwaters exposed to perpendicular wave attack. The influence of various geometrical types of breakwaters is taken into account using influence factors in connection with the general formula for a fully vertical breakwater. Further, a prediction formula for the probability distribution of individual overtopping volumes is presented. The effect of overtopping volumes on persons and cars behind a crown wall of a vertical breakwater was assessed by Franco (1993), and a set of critical overtopping discharges were proposed (safety criteria).

Only a little research has been made to study the influence of the angle of wave attack on the amount of overtopping water. De Wall and Van der Meer (1992) have carried out tests on the influence on wave run-up and overtopping on smooth slopes. The angle of wave attack, β, was varied from 0° up to 80°, and tests were performed with both long-crested and short-crested waves. For long-crested waves, a few tests showed larger run-up for angles between 10° and 30° than for perpendicular waves, but on average no increase was found. This also applies to the average measured overtopping discharges. For perpendicular wave attack, no difference in wave overtopping was measured between tests with long-crested and short-crested waves, whereas for oblique waves the influence of the angle of wave attack was less for short-crested waves. A reduction in the mean overtopping discharge of about 40 per cent was found for long-crested waves with an angle of 50° and of about 15 per cent for short-crested waves.

Galland (1994) has measured the number of waves overtopping rubble mound breakwaters exposed to oblique wave attack. The model tests were made with four different types of armour units, ie quarry stones, accropodes, antifer cubes and tetrapodes. In the case with quarry stones, the test results for long-crested waves showed a significant decrease in the percentage of waves overtopping the crest by increasing the angle of wave attack. For a dimensionless free board, R_c/H_s, higher than 2.0, no overtopping waves were measured, and for R_c/H_s = 1.0 the percentage of overtopping waves was about ten per cent for perpendicular waves which was reduced to no overtopping waves for an angle of 75°.

Model Set-up and Test Programme

Physical model tests have been carried out in a wave basin at the Danish Hydraulic Institute with the aim of measuring mean overtopping discharges defined as the volumes of wave overtopping per unit length of the breakwater per unit time. The basin was equipped with a movable wave generator in order to study the effect for different angles of wave attack, see **Fig 1**. The tests were carried out using long-crested irregular waves generated on basis of a Pierson-Moskowitz spectrum.

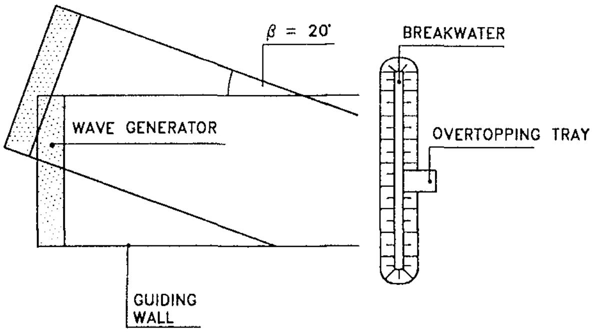

Fig. 1 Model plan for the basin tests showing the set-up for perpendicular and 20° wave attack.

The modelled structure had a total length of about seven metres and was constructed as a traditional rubble mound breakwater with a core, filter layer and armour layer of quarry stones, see **Fig. 2**. The size of the armour stones was selected not to allow for significant damage during testing, ie a nominal diameter, $D_{n,50}$, of about 0.04 m. The model consisted of a horizontal seabed which together with the use of a fixed breakwater height of 0.45 m means that variations in the crest free board were obtained by changes in the water level.

The wave conditions in the model were measured by seven resistance type wave gauges located in front of the breakwater. For perpendicular wave attack, the incident wave conditions and the reflection coefficients have been calculated using a multi-gauge technique. The significant wave height was calculated as $4 \times \sqrt{mo}$, where mo is the zeroth moment of the spectral energy density function.

The overtopping water was collected in a 0.6 m wide tray located immediately behind the breakwater in a level corresponding to the crest elevation of the breakwater. This means that the recorded wave overtopping refer to water passing the rear edge of the breakwater crest. By measuring the total amount of overtopping water after each test with a duration of 600 to 1800 seconds, the mean overtopping discharge, q, was calculated.

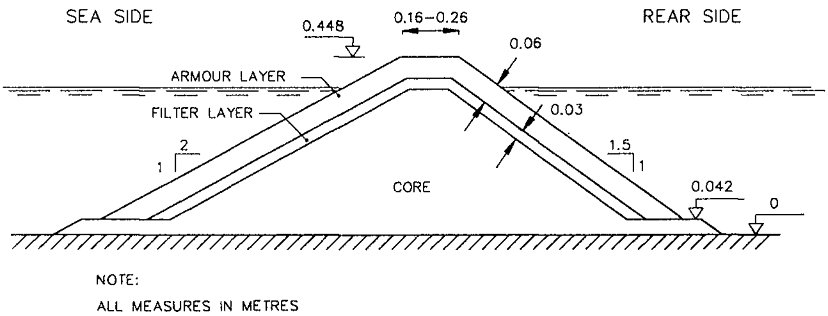

Fig. 2 Typical cross-section of breakwater used in the overtopping tests.

The following ranges of parameters were tested in the model study (all measures are in model measures):

- Significant wave height, H_s: 0.05 to 0.11 m
- Peak wave period, T_p: 0.8 to 2.0 s
- Wave steepness, s_{0p}: 0,018, 0,025, 0,030 and 0,045
- Crest free board, R_C: 0.050, 0.075 and 0.100 m
- Width of crest, B: 0.16 ($4 \cdot D_{n50}$), 0.21 and 0.26 m
- Slope angle, $\cot\alpha$: 2.0
- Angle of wave attack, β: 0° to 50° in steps of 10°

The wave steepness is given by the ratio between the significant wave height and the deep water wave length calculated on basis of the peak wave period:

$$s_{0p} = H_s/L_{0p} = 2\pi/g \cdot H_s/T_p^2$$

A parameter often used in the research on coastal structures is the surf similarity parameter given as:

$$\xi_{0p} = \tan\alpha/\mathrm{sqrt}(s_{0p})$$

The model tests were run in test series with fixed wave steepness, ie a fixed ratio between the significant wave height and the deep water wave length. Thus all tests were made with a surf similarity parameter larger than 2 (two), which means that the wave conditions can be characterised as non-breaking waves.

The dimensionless free board, defined as R_c/H_s, varied between 0.5 and 2.0, which means that the tests covered both low and high crested breakwaters.