

CHAPTER 57

WAVE OVERTOPPING ON RUBBLE MOUND BREAKWATERS

Pierluigi Aminti *

Leopoldo Franco **

ABSTRACT

The paper gives the results of an extensive series of hydraulic model tests carried out in a random wave flume, in order to study the effects on wave overtopping of the main geometric parameters of a typical rubble mound breakwater with crown wall. The results have been compared with those from other studies and analyzed with different methods. Generalized design diagrams and formulae for the prediction of overtopping discharges are finally given for a large number of popular breakwater configurations.

1. INTRODUCTION AND BACKGROUND

Wave overtopping is one of the most important hydraulic processes affecting the design of a breakwater, especially when a crown wall protects a quay or a reclamation. However, most research work on breakwaters just deals with the hydraulic stability and structural response of the primary armour. Still too limited information is available to the engineers for predicting the overtopping discharges and then checking against well established admissible values. In particular no reliable methods allow the fundamental design selection of the breakwater crest elevation and configuration to be effective against wave overtopping.

Model tests have often been carried out with regular waves, with consequent underestimation of the overtopping rates, which are mainly governed by the highest waves in the train.

Even the current prediction method given by the Shore Protection Manual (SPM, 1984) is based on early tests with monochromatic waves. In this method the overtopping discharge is given by a formula

* Associate Professor of Marine Civil Engineering, University of Florence, Via S.Marta 3, Florence, Italy

** Associate Professor of Coastal Engineering, Polytechnic of Milan, Piazza Leonardo da Vinci 32, Milan, Italy

related to the potential runup on a corresponding infinite slope. Then many problems arise as pointed out by Ahrens and Heimbaugh (1986). Moreover it should be observed that the runup computations themselves may be wrong. In fact in the SPM the runup on a rough slope is linearly related, by a simple reduction factor, to the runup on the equivalent smooth slope, while extensive random wave model tests reported by Allsop, Franco and Hawkes (1985) have shown a quite different behaviour of the two types of slopes for variable Iribarren numbers.

Some simplified theoretical approaches for the evaluation of overtopping discharge have been proposed by various Japanese researchers and recently by Jensen and Juhl (1987). These Authors also report on the very few existing field measurements carried out in Denmark and in Japan, which seem to agree with model test results when the experiments are conducted with irregular waves.

The Japanese prototype measurements reported by Fukuda, Uno and Irie (1974) were also used to provide the only available rough and conservative guidelines for acceptable overtopping discharges or intensities. In general, according to them, inconvenience for persons or vehicles (passing 3 m behind the crown wall) arise when the mean overtopping discharge per unit length of breakwater (Q) reaches about $10^{-6} \text{ m}^3/\text{s m}$, while danger occurs if Q exceeds $2 \cdot 10^{-5} \text{ m}^3/\text{s m}$. These apparently low figures account for the fact that the danger levels are actually determined by the single largest overtopping waves which, due to the high irregularity of the physical phenomenon, can produce peak intensities a few hundreds times greater than the average intensity.

Most information is still obtained from laboratory work, which however often concerns seawalls in shallow water, such as the random wave investigations performed by Owen (1980) and by Ahrens and Heimbaugh (1986).

With reference to rubble mound breakwaters in relatively deep water, Jensen (1984) collected results from irregular wave model tests carried out at DHI for several different projects (some including the wind effect too); the set of graphs, later reanalyzed by Jensen and Juhl (1987), is useful but limited to the specific geometries and conditions of the tested breakwaters.

The lack of reliable and complete design guidelines then stimulated the Authors to begin in 1986 a systematic experimental research program: the preliminary results given by Aminti and Franco (1987) are now supplemented by numerous additional tests with new configurations, whose results are presented here in a more effective and general form.

At the final preparation of this paper it was possible to examine the results of a similar basic research program conducted at HR, Wallingford by Bradbury and Allsop (1988). They studied the effect on overtopping discharge of various crown wall and armour crest configurations with smooth and rock armoured slopes at a fixed angle (1:2). The satisfactory comparison of the hydraulic performance of the similar structures then suggested a consistent and general form of analysis and presentation of the experimental data, as shown in par. 3.

2. TEST EQUIPMENT AND PROCEDURE

The model tests were conducted in the 48 m long, 0.80 m wide and 0.80 m deep wave flume of the Florence University's hydraulic laboratory. The flume is equipped with an oleodynamically actuated piston-type wavemaker, with a random wave input signal obtained by filtering a white noise (Aminti, Liberatore and Petti, 1984).

A Jonswap type spectrum was reproduced with a significant wave height $H = H_{s1/3,u} = 0.136$ m and a mean period $T = T_u = 1.33$ s, measured by capacitive gages at the model structure location before its construction, having an absorbing beach at the end of the flume.

The water depth at the model section was 0.40 m (with a constant foreshore gradient of 1/50) to ensure non-breaking wave conditions before the structure. The duration of each test, twice repeated, was 10 minutes (one hour prototype). A 1:36 Froude undistorted scale was in fact used.

Only one random wave condition was considered in order to reduce the number of tests to the manageable figure of 270 and focus the attention to the influence of breakwater geometry and construction.

The typical model test section is shown in fig.1. The usual composition of the relatively impermeable core and of the filter layer were not changed, while the other structural parameters affecting the overtopping performance were varied as follows, in order to simulate the most popular prototype configurations (for symbols see fig.1):

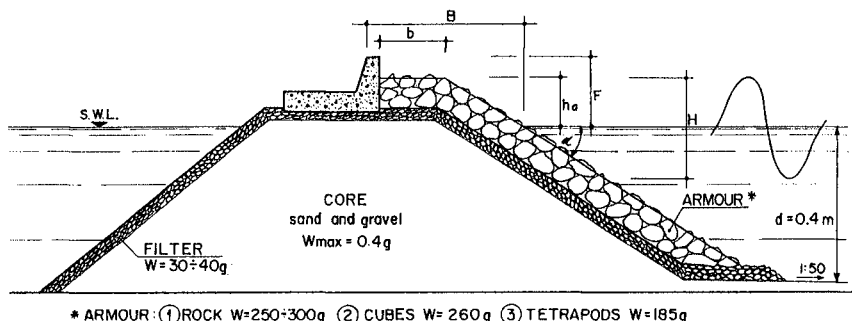


FIG.1: Typical model test section with notation of geometric parameters

- 1 - Relative crown wall freeboard (to S.W.L.): $F/H = 0.60 \div 2.00$
- 2 - Relative armour crest height (to S.W.L.): $h_a/H = 0.60, 0.75, 1.05$
- 3 - Relative armour crest berm width: $b/H = 1.10, 1.85, 2.60$
- 4 - Seaward slope angle: $\cot \alpha = 1.33, 1.50, 2.00$
- 5 - Type of armour unit: rock, cube, tetrapod

It should be noted that:

- F and h_a are not totally independent, being $F/h_a \geq 1.0$ in all tests;
- the three ratios b/H approximately correspond to horizontal armour crest berms of 3-5-7 units respectively;
- the armour units had random placement with the typical recommended

densities and their weights determined as to guarantee stability under the test conditions;

- the parameter B/H_s , which represents the breakwater width, is dependent on b , h_s^a and $\cotg\alpha$ and varies between 1.9 and 4.7;
- due to the marginal influence of the slope angle the analysis was then restricted to the extreme values of $\cotg\alpha = 1.33$ and 2.00 (slopes 3/4 and 1/2).

In each tests the following measurements were taken:

- a) number of overtopping waves (visually counted)
- b) average overtopping discharge (collected in a graduated cylinder through a 0.1 m wide prismatic pipe placed behind the crown wall)
- c) jet falling distance (visualized by the water quantities contained in five consecutive trays).

However, the latter measurements were not considered accurate enough to derive reliable quantitative results, also bearing in mind the neglected wind effect and the incorrectly scaled simulation of the air-drop interaction. It was just possible to observe qualitatively the same exponential decay found by Jensen and Juhl (1987).

In consideration of the practical design needs the attention was concentrated on the model measured overtopping discharges Q ($m^3/m\ s$) averaged over the time interval for one metre of breakwater length.

3. PRESENTATION OF MODEL TEST RESULTS

An open question is the selection of the proper parameters for the presentation of the experimental data. The above mentioned Authors use different dimensionless frameworks (and often different symbols) to generalize the relationship between the overtopping discharge Q and the crown wall freeboard F , which is the most relevant geometric factor.

A significant summary of this variety of coefficients is given in tab.1. In Jensen and Juhl (1987) the parameter Q was not even made dimensionless, because of the difficulty² in deriving a unique universal factor (the previously used factor QT_m/B^2 is no more regarded correct).

Even the proposed empirical relationships are not consistent and generally have one of these two different forms:

$$Q^* = A \exp(-BF^*) \quad (1)$$

$$Q^* = A (F^*)^{-B} \quad (2)$$

where Q^* and F^* are the dimensionless discharge and freeboard, A and B best fit coefficients.

The data obtained from the present tests was therefore analyzed using a variety of methods. In general a higher correlation was obtained with equation (2), as shown by the linear regressions of the logarithms of Q^* and F^* (in agreement with Bradbury and Allsop, 1988).

The dimensionless parameters which are regarded most effective and of simplest practical use are Q^* ($=Q/g T_m H_s$) and F/H_s . However, due to the unique wave test condition, all graphs have been drawn with double scales/parameters for both the discharge (Q and Q^*) and the freeboard F/H_s and $F^* = (F/T_m \sqrt{g H_s}) \cdot (F/H_s)$ in order to make an easier comparison with the recent data of Bradbury and Allsop (1988) and to

AUTHOR	DIMENSIONLESS DISCHARGE Q^*	DIMENSIONLESS FREEBOARD F^*
Owen (1980)	$\frac{\bar{Q}}{\bar{T} g H_s}$	$R^* = \frac{R_c}{\bar{T} \sqrt{g H_s}}$
Jensen (1984)	$\frac{Q T_z}{B^2}$	$\frac{H_s}{\Delta h}$
Ahrens and Heimbaugh (1986)	$\frac{Q}{\sqrt{g H_{mo}^3}}$	$F' = \frac{F}{(H_{mo}^2 L_p)^{1/3}}$
Bradbury and Allsop (1988)	$\frac{\bar{Q}}{T_m g H_s}$	$F^* = \frac{R_c}{T_m \sqrt{g H_s}} \cdot \frac{R_c}{H_s}$
Sawaragi, Deguchi Park (ICCE, 1988)	$\frac{\bar{Q}}{\sqrt{g L H^2}}$	$\frac{H_c}{H}$
Note: $R_c = H_c = \Delta h = F =$ crown wall freeboard		

TAB.1: Summary of representative overtopping parameters
proposed by various Authors

facilitate their general design use.

The large number of tested variables allows the plotting of several diagrams outlining the influence of each variable with different configurations. Due to space limitations it was decided to plot all data within six graphs for fixed armour type and slope angle, thus emphasizing the most relevant effects of variable freeboards and armour berm widths (figs. 2 A-B, 3 A-B, 4 A-B).

Then, the marginal influences of variable armour type or slope angle (for fixed berm widths) can be both shown by just one representative typical diagram (fig. 5 A-B). For a better analysis of the effect of the armour type, the configuration with minimum path to be run by the overtopping wave was selected ($b/H_s = 1.1$, slope $3/4$). Similarly the influence of the slope could be better detected for the configuration with shortest berm and most controlled armour units (tetrapods).

It should be noted that the data scatter is partly due to the different values of h_a tested for the same F . However all regression lines have very high correlation coefficients, with a minimum of 0.89.

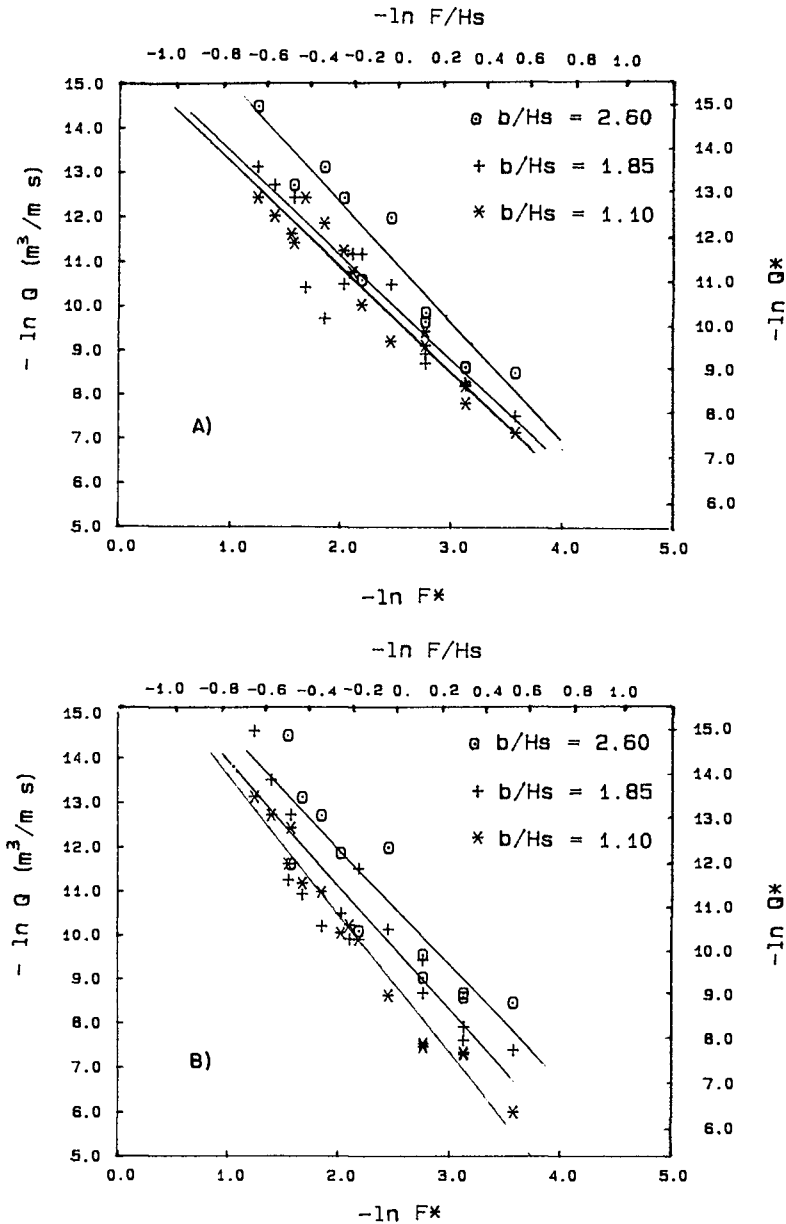


FIG.2: Effect of freeboard and armour crest berm width on overtopping discharge. Test results for rock armor: A)slope 1/2,B)slope 3/4

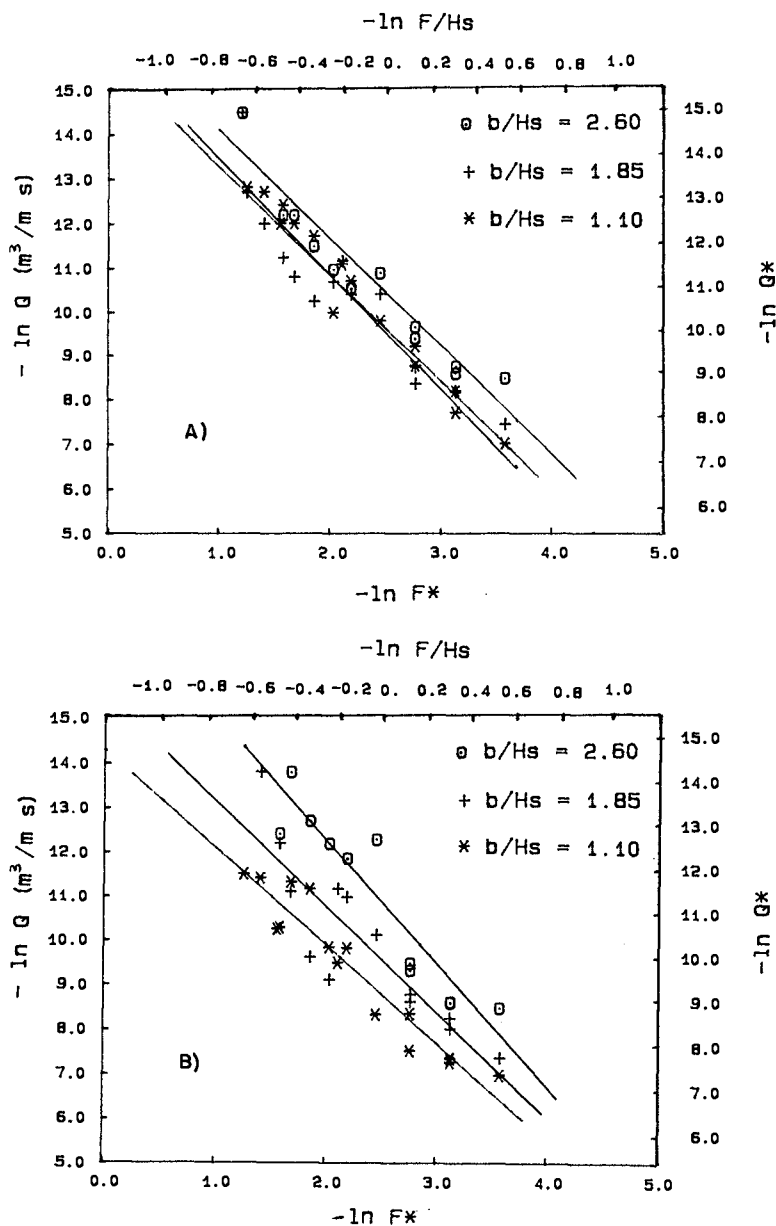


FIG.3: Effect of freeboard and armour crest berm width on overtopping discharge. Test results for cubes: A) slope 1/2, B) slope 3/4

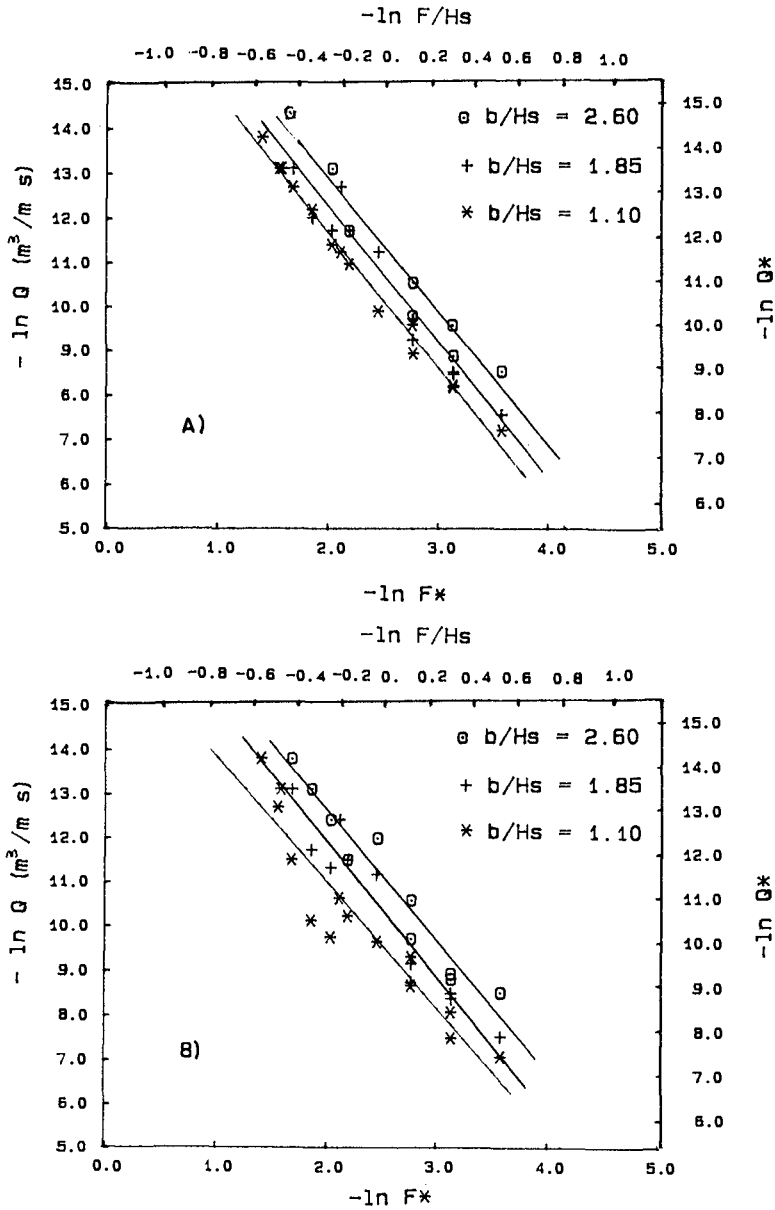


FIG.4: Effect of freeboard and armour crest berm width on overtopping discharge. Test results for tetrapods: A)slope 1/2, B)slope 3/4

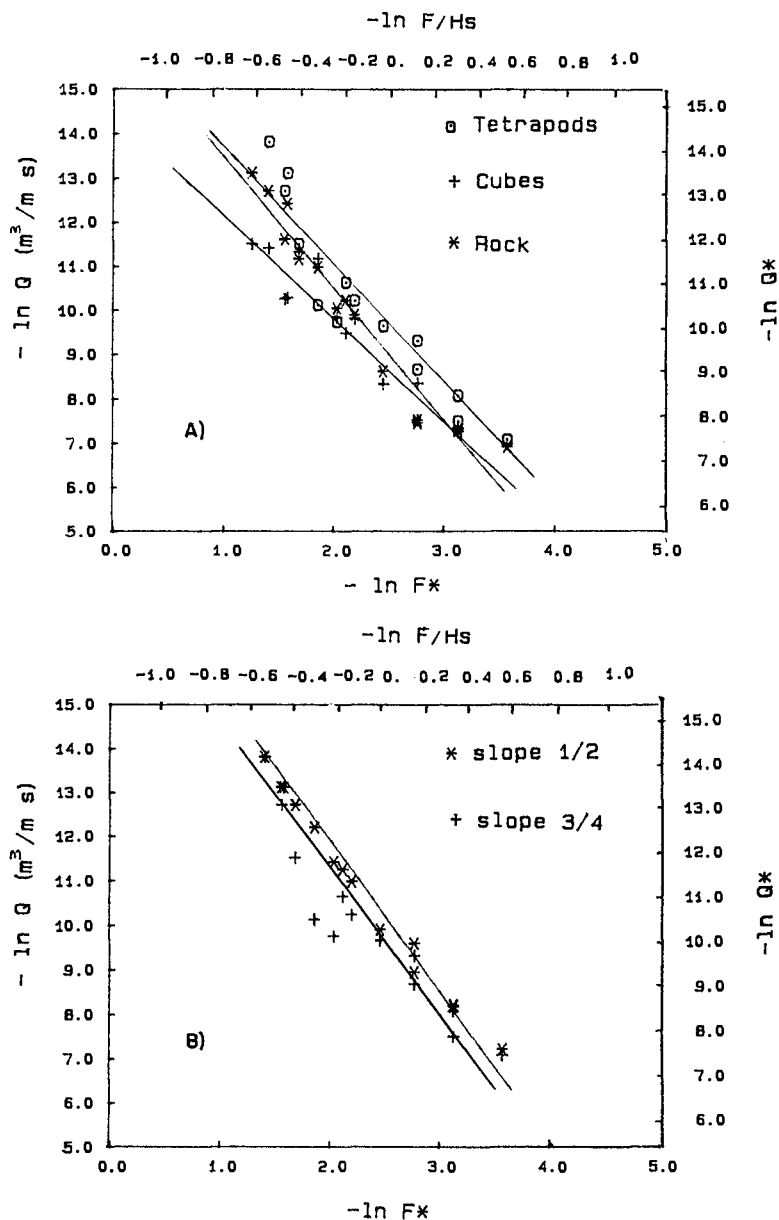


FIG.5: A) Effect of freeboard and armour type on overtopping discharge. Test results for slope 3/4 and $b/H_s = 1.1$

B) Effect of freeboard and slope angle on overtopping discharge. Test results for tetrapod armour and $b/H_s = 1.1$

In tab.2 the empirical coefficients A and B obtained for the various test section are listed.

ARMOUR	SLOPE	b/H _s	A	B	R ²
ROCK	1/2	1.10	1.67 10 ⁻⁸	2.41	0.97
		1.85	1.85 10 ⁻⁷	2.30	0.92
		2.60	2.27 10 ⁻⁸	2.68	0.95
ROCK	3/4	1.10	5.05 10 ⁻⁸	3.10	0.98
		1.85	6.83 10 ⁻⁸	2.65	0.92
		2.60	3.07 10 ⁻⁸	2.69	0.95
CUBES	1/2	1.10	8.33 10 ⁻⁸	2.64	0.98
		1.85	1.52 10 ⁻⁷	2.43	0.94
		2.60	8.35 10 ⁻⁷	2.38	0.96
CUBES	3/4	1.10	6.16 10 ⁻⁷	2.20	0.95
		1.85	1.68 10 ⁻⁷	2.42	0.89
		2.60	1.86 10 ⁻⁸	2.82	0.93
TETRAPODS	1/2	1.10	1.88 10 ⁻⁸	3.08	0.99
		1.85	1.13 10 ⁻⁸	3.80	0.97
		2.60	1.07 10 ⁻⁸	2.86	0.96
TETRAPODS	3/4	1.10	5.59 10 ⁻⁸	2.81	0.94
		1.85	1.68 10 ⁻⁸	3.02	0.96
		2.60	9.23 10 ⁻⁹	2.98	0.97

TAB.2: Summary of empirical coefficients

The analysis of the plotted results leads to the following main observations:

- 1) Increasing the freeboard of the vertical wall (F) has the greatest effect in reducing the overtopping discharge (according to a power law).
- 2) Increasing the armour crest berm width (b) also produces a reduction of discharge, which is more evident and consistent with steeper slopes and with tetrapod armour units.
- 3) Shallower slopes (1:2) can also reduce overtopping, probably due to the longer rough path; this slope-effect is confirmed by the results from Jensen and Juhl (1987) and is more evident with narrow armour berms.
- 4) Tetrapods have a slightly better hydraulic performance than rock and cubes; the rock armour seems to give less overtopping discharges than cubes except for extreme events.
- 5) Increasing the armour crest elevation (h_a) often results in a reduction of discharge, but the trend is not clearly defined and no plot is therefore given.

It can be observed that in order to satisfy the existing safety guidelines proposed by Fukuda, Uno and Irie (1974), within the test

conditions, the relative crown wall freeboard F/H generally ranges between 1.0 and 1.4. The conservative nature of these^s guidelines may be compensated by the neglected effects of onshore winds, oblique wave attack and by scale effects.

It was possible to check just a few test results against data points for similar conditions reported by Jensen and Juhl (1987) in their figures 12-13: the measured discharges compare quite well.

A fair agreement can also be found with the results obtained by Bradbury and Allsop (1988), although the overtopping prediction seems to give slightly larger values than those reported for their most similar configurations n.8 and 10. Moreover their assumption of slope $1/2$ being the worst case is contradicted by the present test results.

Finally another regression analysis was carried out with all data, in order to relate the overtopping discharge Q to the percentage of overtopping waves P . Again the best fit was given by an equation of the form $Q = C P^D$ (C , D empirical coefficients): the discharge increases rapidly as the number of overtopping waves increases. However the data scatter and the limits of tested conditions don't allow the derivation of a useful relationship of general validity.

4. CONCLUSIONS

The results of several random wave model tests for variable configurations of breakwater superstructure have shown the influence of various geometric parameters on the overtopping discharge and allowed the presentation of a set of graphs useful for preliminary design purposes.

The prediction model providing the best description of the overtopping performance is based on an equation of the form $Q^* = A (F^*)^{-B}$, where the dimensionless discharge Q^* rapidly increases with decreasing relative crown wall freeboard F^* .

However, representative universal dimensionless factors are still to be defined. Other geometric parameters have shown a significant effect on the overtopping, especially the width of the armour crest berm. The change of seaward slope angle also has some influence, while different armour types have a quite similar behaviour (within the test conditions). The inclusion of these effects into a more complex comprehensive factor might be possible when more tests will be conducted with different wave conditions (wave periods in particular), and by combining the data from various Authors.

It is then hoped that a normalization of definitions and symbols of the typical breakwater geometric parameters will be agreed internationally, as recently done by IAHR-PIANC for the wave parameters.

Further work should also be addressed to the definition of more accurate, detailed and universal design criteria, i.e. admissible overtopping discharges for different degrees and destinations of the protection. More measurements of individual overtopping discharges should be carried out both in model and prototype conditions, together with the assessment of their consequences on variable targets.

REFERENCES

- AHRENS J.P. and HEIMBAUGH M.S. (1986): Irregular wave overtopping of seawalls, Proc. Conf. IEEE Oceans '86, Washington
- ALLSOP N.W.H., FRANCO L., HAWKES P.J. (1985): Probability distribution and levels of wave runup on armoured rubble slopes, Int. Conference on Numerical and Hydraulic Modelling of Ports and Harbours, Birmingham
- AMINTI P., LIBERATORE G., PETTI M. (1984): Generatore di moto ondoso irregolare: l'esperienza del laboratorio idraulico del dipartimento di ingegneria civile di Firenze, XIX Convegno di Idraulica, Pavia
- AMINTI P., FRANCO L. (1987): Indagine sperimentale sulla tracimazione di onde irregolari su dighe a scogliera, Congresso AIPCN sez. Italiana, Ravenna
- BRADBURY A.P. and ALLSOP N.W.H. (1988): Hydraulic effects of breakwater crown walls, BREAKWATERS '88 Conference, ICE, Eastbourne
- C.E.R.C. (1984): Shore Protection Manual, Washington
- FUKUDA N., UNO T., IRIE I. (1974): Field observations of wave overtopping of wave absorbing revetment, Coastal Eng. in Japan, 17
- JENSEN O.J. (1984): A monograph on rubble mound breakwaters, Book published by Danish Hydraulic Institute, Horsholm
- JENSEN O.J., JUHL J. (1987): Wave overtopping on breakwaters and sea dikes. 2° Conf. Coastal and Port Eng. in Developing Countries, Beijing
- OWEN M.W. (1980): Design of seawalls allowing for wave overtopping, Report EX924, Hydraulics Research, Wallingford