CHAPTER 121

STABILITY OF RUBBLE MOUND BREAKWATER

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ABSTRACT

The stability of a rubble mound breakwater section, with 3 in 2 armour slope, was tested under random waves attack. Tests analysis shows that the equivalent wave height characterizing the spectrum to be used in a stability formula elaborated with regular waves (for instance the Hudson's formula) is the upper twentieth height of the distribution for a storm duration of 6 hours. An analytical expression of the damage evolution as function of time modulates this choice according to the storm duration. The same rubble mound breakwater was also tested under the action of regular breaking waves. The damage was expressed in terms of the four following parameters :

H_o : wave height

T : wave period

D_p : water depth at the toe of the structure

D_b : breaker depth without the breakwater

For a given wave height, the most important damage occur when :

$$\frac{D_p}{D_b} = 1$$

In this case the design wave height must be increased by about 30 % when using a stability formula elaborated for non breaking waves.

1 - INTRODUCTION

The use of breakwater stability formula (e.g. Hudson's formula) [1] to determine the weight of armour units of rubble mound breakwater raises two questions :

What wave height do we have to choose to represent the action of a random sea and is this dependant upon the duration of the design storm ?

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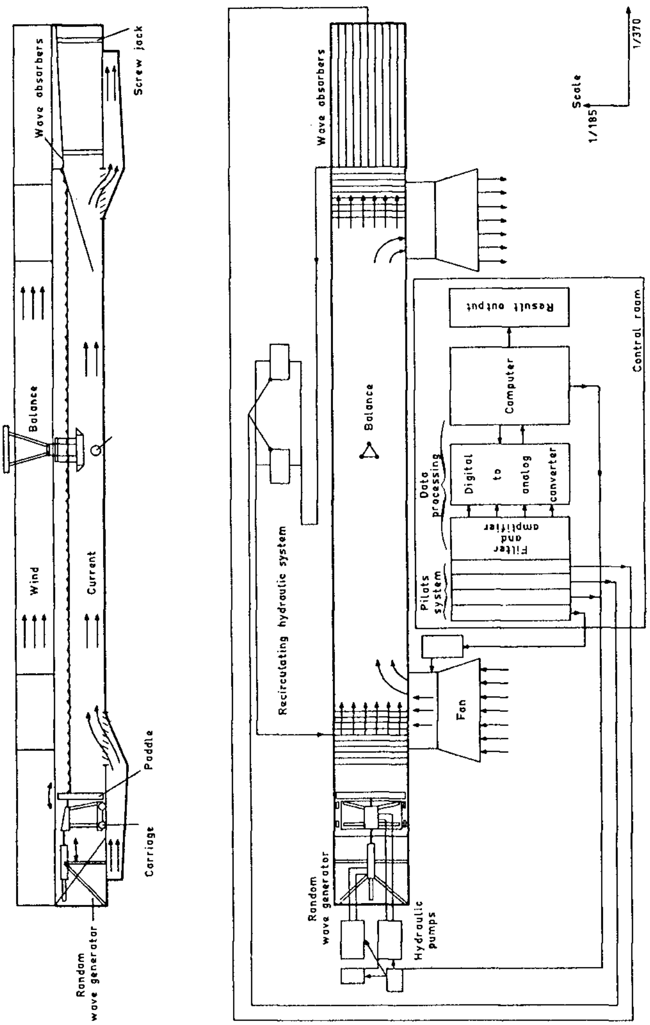


Figure 1

SCHEMATIC OF THE EXPERIMENTAL INSTALLATION

How can we take into account the effect of breaking waves ?

It has been generally accepted that the destructive effect of random waves is equivalent to that of regular waves having a height equal to the significant wave height characterizing the real sea state and the armour blocks weight is often calculated with this criterium. Is this approach correct ? The paper presents the results of three series of tests of breakwater stability on a reduced scale model with random non breaking waves, regular non breaking waves and regular breaking waves.

II - DAMAGE EVOLUTION IN RANDOM WAVES AS COMPARED WITH REGULAR WAVES

II.1 - Test procedure

. wave flume : A sketch of the wave tank layout is shown in figure 1. The flume is sixty meters (185 ft) long, five meters wide (15,4 ft) and 1,6 meters (4,9 ft) deep. The wave generator consists of a carriage rolling on rails laid on the flume bottom and of a plane paddle, hinged on this carriage, swinging around an axle located near the flume bottom. Carriage and paddle are operated by hydraulic pistons fed with two variable output pumps. The two movements, translation and rotation are independant. This wave generator is able to simulate random or monochromatic waves. In random waves the control signal is provided directly by a digital computer. Wave heights and periods were measured with resistance type wave gages. A fast measurement central unit ensures the analog switching and the analog digital convertion of wave data. These last one are analysed by the computer which gives harmonic, spectral, and statistical analysis of the data.

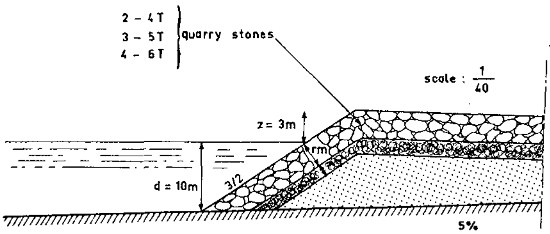


Figure 2

BREAKWATER MODEL FOR RANDOM WAVES

. Breakwater model : A simplified sketch of the cross section of the model studied is shown in figure 2. The scale was 1/40. The armour layer was composed of blocks with a weight in the range 2-4 T (mean 2.6 T) and a specific weight of 2.6 T/m^3. Its slope was 3 in 2. The water depth at the toe of the breakwater was 10 m (30,8 ft). The bottom slope in front of the breakwater was 1 in 10.

. Test conditions : In order to make comparisons possible, the same model was submitted to the attack of regular waves and irregular waves. The storm duration was lower than 20 hours (30 mn at model scale).

The incident wave was measured using 3 wave gages, according to the method described by GODA [7].

II.2 Test results and analysis

II.2.1 - Results in regular waves

Figure 3 shows that the results obtained in regular waves compare well with Hudson's results. The stability number N_S is defined as :

$$N_s = \frac{H}{\left(\frac{w}{\gamma_r}\right)^{1/3} (S_r-1)} \quad \text{with} \quad S_r = \frac{\gamma_r}{\gamma_w}$$

The damage D is defined as the ratio of the number of removed blocks to the total number of armour layer blocks.

II.2.2 - Results in irregular waves for a storm duration of 6 hours.

In order to define a design wave height in random waves for a storm duration of 6 hours, we have compared the values of N_s obtained by Hudson in regular waves to the values obtained in random waves for different typical wave heights :

$$\left(\overline{H_Z,\frac{1}{3}},\ \overline{H_Z,\frac{1}{10}},\ \overline{H_Z,\frac{1}{20}},\ \overline{H_Z,\frac{1}{50}}\right)$$

The best agreement between N_s values calculated in regular waves and random waves is obtained for the wave height $\overline{H_Z,1/20}$ (figure 4), which characterizes the action of random waves for a storm duration of 6 hours. This wave height will be noted H_D. But this result has to be modulated because the storm duration has a great effect upon the rubble mound breakwater stability.

III.2.3 - Effect of the storm duration in random waves

The purpose of this study was to compare the damage evolution in regular and irregular waves, considering random waves as a successsion of independant waves, and to obtain a literal formula expressing the damage evolution under random waves action as function of a characteristic height and of the storm duration. Two different methods have been explored. In the first one we determine in regular waves a law describing evolution as a function of the wave height and the time, and then deduced by integration a law in irregular waves.

This method leads to the following expression :

$$D = \frac{1}{T_{moy}} \int_o^t \int_o^\infty DPV.p(h)\ dh\ d\tau. \qquad (1)$$

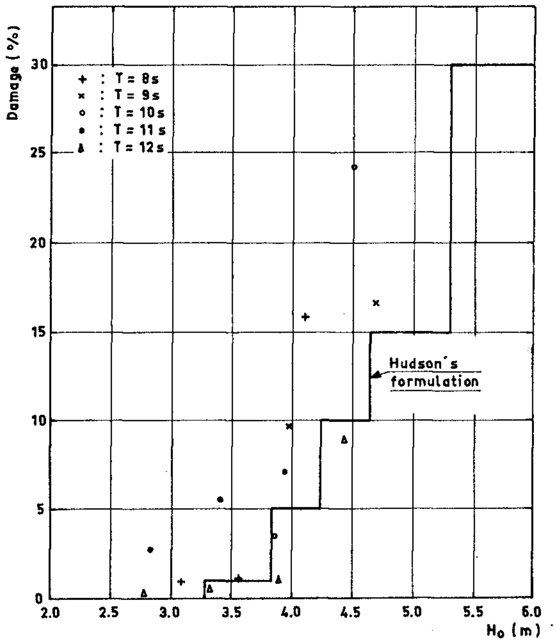


Figure 3

COMPARISON OF EXPERIMENTAL RESULTS WITH HUDSON'S FORMULA IN REGULAR WAVES.

with $p(h)$: probabilité law of wave height ;

T_{moy} : mean period ;

D : damage ;

t : storm duration ;

DPV : function expressing damage provoqued by one wave of height h attacking the breakwater at time τ.

The values given by this formula compare well with experimental results (figures 5 and 6).

In the second method we have obtained directly a literal formulation of damage evolution in random waves by a statistical adjustment of the experimental results with a least squares fitting through all the experimental points.

By this method we have obtained the following expression [5] :

$$D = \frac{2.78}{T_p} \overline{H_{Z,\frac{1}{10}}}^{3.9} t^{0.37} \qquad (2)$$

with D : damage expressed as number of removed blocks ;

t : storm duration in hours ;

T_p : peak period ;

$\overline{H_{Z,\frac{1}{10}}}$: the upper tenth mean height of the distribution.

This statistical adjustment has been obtained with a good correlation coefficient ($\rho \simeq 0,96$).

The results obtained by this method are represented in dashed lines on the figures 5 and 6.

II.2.4 - Application to the determination of the design wave height H_D

The results obtained in [4], and presented in II.2.2, lead for a storm duration of 6 hours to the relation :

$$H_D = \overline{H_{Z,\frac{1}{10}}} \# 1.1 \overline{H_{Z,\frac{1}{10}}} (t=6) \qquad (3)$$

expressing the equivalence (for the damage) between a regular wave of height H_D acting till the stabilization of the breakwater and a random wave of height $\overline{H_{Z,\frac{1}{20}}}$ acting during 6 hours.

From equation (2) can be derived the relation :

$$\overline{H_{Z,\frac{1}{10}}} = kt^{-0,095} \qquad (4)$$

Which expresses that for given blocks and damage (e.g. k = constant)

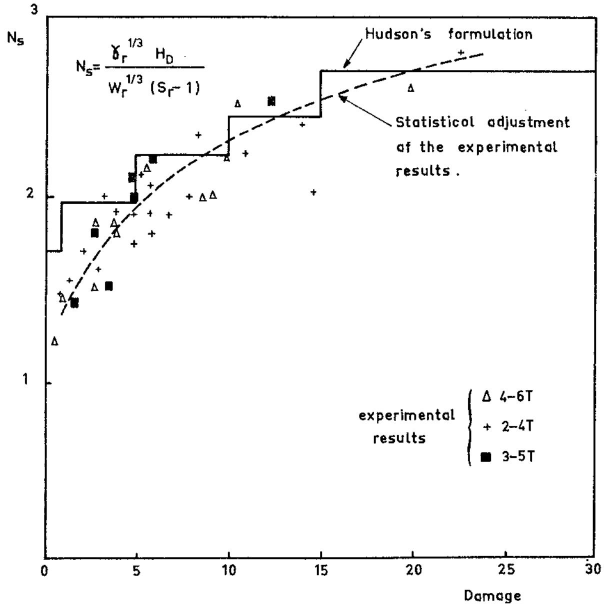


Figure 4

COMPARISON OF EXPERIMENTAL RESULTS
WITH HUDSON'S FORMULA

N_S estimated with $\overline{H_Z \cdot \frac{1}{20}}$ for a storm duration of 6 hours

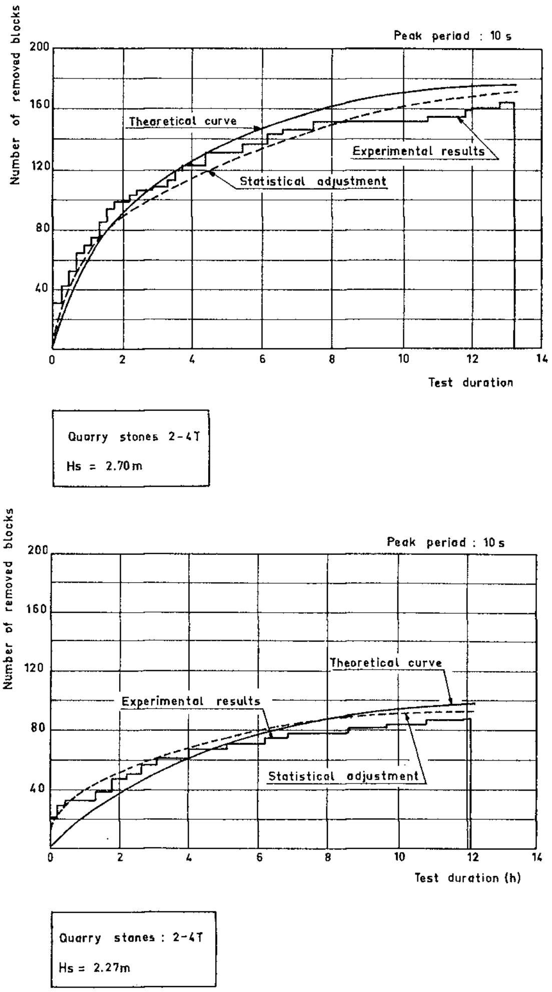


Figure 5

DAMAGE EVOLUTION IN RANDOM WAVES

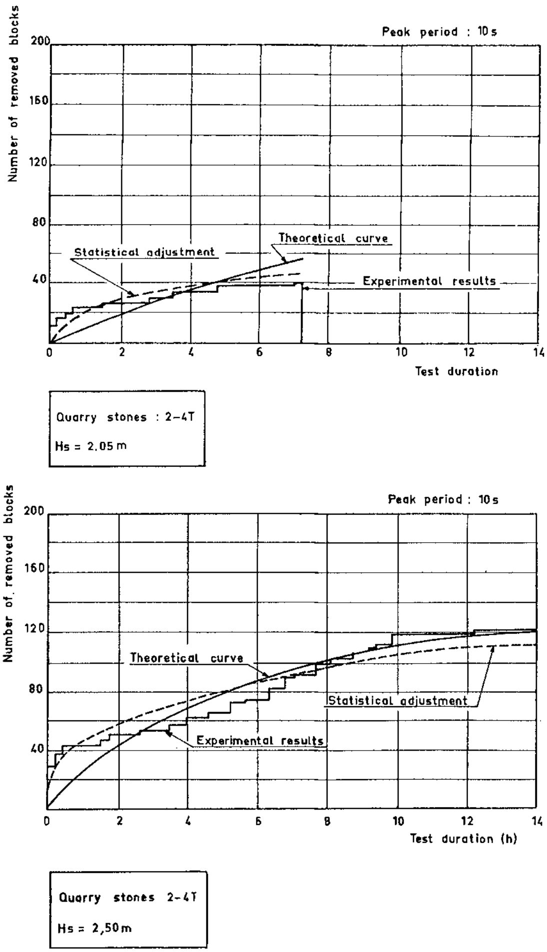


Figure 6

DAMAGE EVOLUTION IN RANDOM WAVES

the wave height $\overline{H_{Z,\frac{1}{10}}}$ at which the rubble mound breakwater resists, decreases when the storm duration increases.

The relation (4) allows us to write (figure 7) :

$$\overline{H_{Z,\frac{1}{10}}}\ (t=6)\ 6^{0,095} = \overline{H_{Z,\frac{1}{10}}}\ (t)\ t^{0,095}$$

and, taking into account the relation (3) :

$$\boxed{\begin{array}{l} H_D = f(t)\ \overline{H_{Z,\frac{1}{10}}} \\ \text{with } f(t) = 0,93\ t^{0,095} \end{array}}$$

This last formula expresses the design wave height H_D to be used in a stability formula established in regular waves as a function of storm duration and wave height $H_{Z,\frac{1}{10}}$ characterizing the random sea spectrum.

The variations of the design wave height with the storm duration (for a duration greater than 3 hours) are relatively small (figure 8).

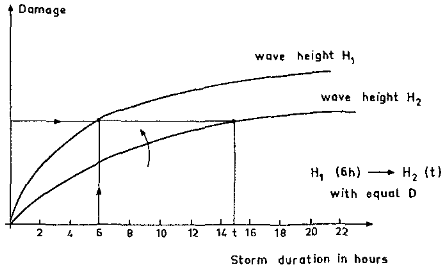


Figure 7 : Method used for the determination of H_d for a storm duration t.

But the choice of $\overline{H_{Z},1/3}$ as design wave height usually adopted seems to underestimate the effect of random waves, whatever the storm duration may be. $\overline{H_Z,\ 1/20}$ is approximatively 40 % higher than the significant wave height and this leads to an important difference in armour units weight, this last one being generally proportionnal to the height to the cube (figure 8).

III - EFFECTS OF BREAKING WAVES

These tests were conducted only in regular waves, in a smaller flume which is 50 m long and 0,60 m wide. The rubble mound breakwater model was the same as the one describes above but the median armour stone weight was 5,37 T.

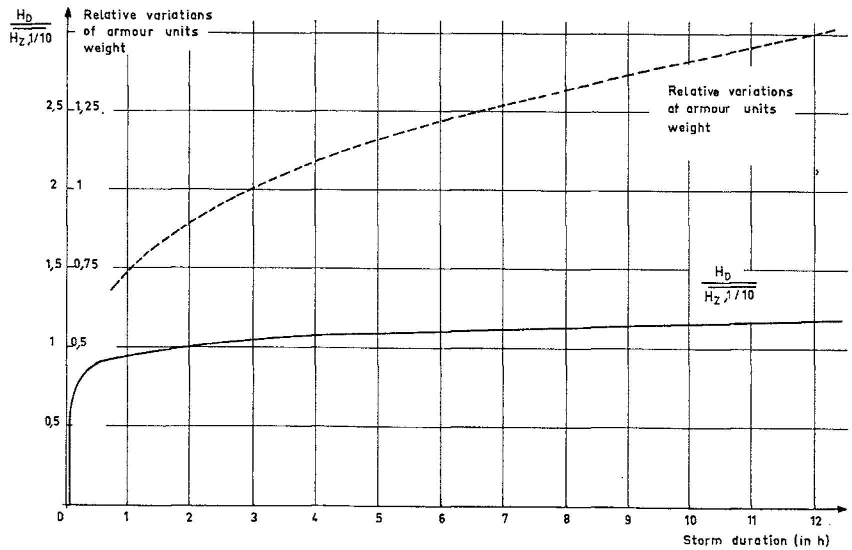


Figure 8

DETERMINATION OF THE DESIGN WAVE HEIGHT AS A FUNCTION OF THE STORM DURATION

The water depth at the toe of the structure varied from 3 m to 10 m and the wave height from 3 m to 5 m. Three periods were tested : 8, 10, 12 s.

Damage was expressed in terms of the four following parameters :

- wave height H_o ;
- wave period T ;
- water depth at the toe of the structure D_p ;
- breaker depth D_b without the rubble mound breakwater.

III.1 - Multicomponents analysis

The statistical study of all the results has pointed out that the stability is strongly influenced by two parameters : the height H_o as usually expressed by the stability formula and the ratio D_p/D_b characterizing the position of the breakwater in the surf zone. This result is illustrated by the figure 10.

Interprepation of the figure 10 :

- the correlation coefficient between two variables may be characterized by the distance séparating these variables ;
- each axis may be interpreted as a variable to be defined ;
- the correlation coefficient between an axis and a variable is given by the projection of the variable on the axis.

So the observation of the figure 10 allows to establish that :

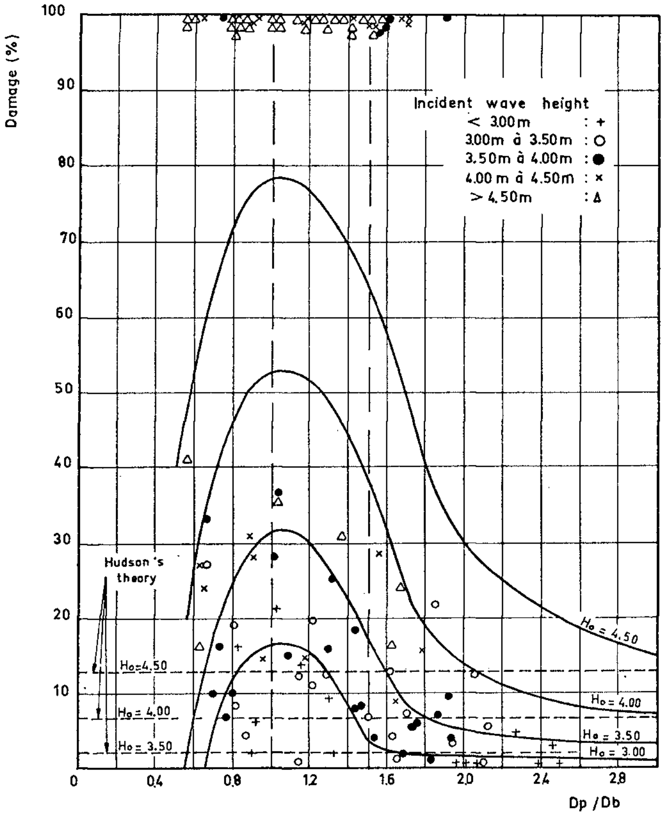
- the damage D and the wave height H_D are strongly correlated ;
- Oy axis may be interpreted as the ratio $\frac{D_p}{D_b}$;
- the correlation coefficient between the damage D and the ratio D_p/D_b is important.

III.2 - Expression of the damage D as a function of $\frac{D_p}{D_b}$ an H_o

Although there is a great scatter of the tests results, a statistical adjustment has been made (figure 9). For a given wave height, the most important damage occurs when $\frac{D_p}{D_b} = 1$.

So a rubble mound breakwater is submitted to the worst attack of waves when it is located just in the breaker zone corresponding to design storm conditions.

When the ratio $\frac{D_p}{D_b}$ is great, that is when in non breaking conditions, the curves presented on the figure 9 are in good agreement with Hudson's results.



Dp : depth at toe of breakwater

Db : breaker depth

Period 8 to 12 seconds

Figure 9

DAMAGE EVOLUTION WITH THE BREAKWATER POSITION AND THE WAVES HEIGHT

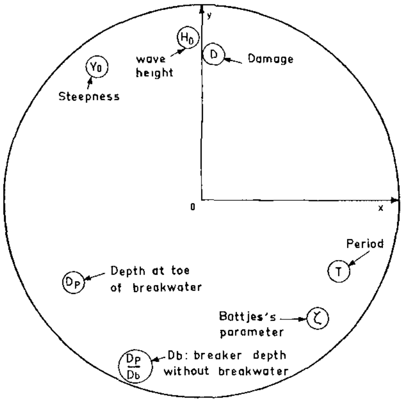


Figure 10 : Détermination of the most influencing parameters (multicomponents analysis)

III.3 - Application to the determination of the design wave height

This statistical adjustment has permitted to express the design wave height as a function of $\frac{Dp}{Db}$ and H_o. This function is represented graphically an the figure11.

This figure shows that in the most unfavourable case $\left(\frac{Dp}{Db} = 1\right)$, the design wave height must be increased by about 15 to 40 % when using a stability formula elaborated for non breaking waves. Again this leads to significantly increased armour units weights.

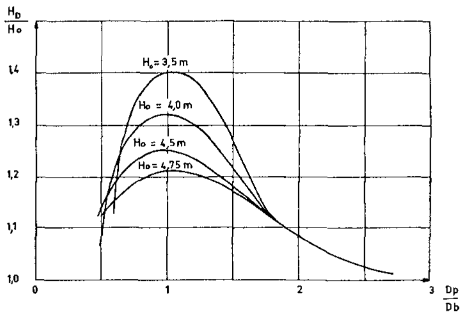


Figure 11

Variations of the ratio $\frac{H_d}{H_o}$ versus $\frac{D_p}{D_b}$ and H_o

IV - CONCLUSIONS

However these tests have been run for conditions which are limited in comparison with the range of conditions to be handled in practice, (e.g. artificial blocks eventually with interlocking effects), they have pointed out that the design wave height to use in a stability formula is higher than the significant wave height of the spectrum and depends of the storm duration.

For a storm duration of 6 hours the design wave height should be approximatively $H_{Z,\frac{1}{20}}$.

Furthermore this choice must be modulated according to the location of the rubble mound breakwater in the breakwater zone of the most severe storms.

REFERENCES

[1] HUDSON R.Y.
Laboratory investigation of rubble mound breakwater.
Journal of the waterways and Harbors division, September 1959.

[2] AHRENS P.
The influence of breaker type on riprap stability.
Proceedings of twelth conference on Coastal Engineering, 1970.

[3] THORTON B, CALHOUN J.
Spectral resolution of breakwater reflected waves.
Journal of the waterways, Harbors and Coastal Engineering division
Proc. ASCE November 1972.

[4] LEPETIT J.P., FEUILLET J.
Etude de la stabilité d'une digue en enrochement en houle aléatoire.
Comparaison avec la stabilité en houle régulière.
Rapport E.D.F. HE42/78.44.

[5] LEPETIT J.P., FEUILLET J.
Etude de la stabilité d'une digue en enrochement en houle aléatoire.
Quantification de l'effet de la durée d'action.
Rapport E.D.F. HE42/79.15.

[6] FEUILLET J.
Etude de la stabilité d'une digue en houle déferlante.
Rapport E.D.F. HE42/78.36.

[7] CODA SUZUKI
Estimation of incident and reflected waves in random wave experiments.
Conference Coastal Engineering, 1976.

[8] CARTERS T., TORUM A., TRAETTEBERC A.
The stability of rubble mound breakwater against irregular waves.
Conference Coastal Engineering, 1970.