CHAPTER 131

Breakwater Stability under Regular and Irregular Wave Attack

Thomas Jensen[*], Henning Andersen[*], John Grønbech[**]
Etienne P.D. Mansard[***] and Michael H. Davies[***]

Abstract

The main objective of the present study was to compare the damage to a rubble mound breakwater under regular and irregular wave attack, and thereby identify an irregular wave height parameter that corresponds to the wave height of a regular wave in terms of inducing a similar degree of damage to the structure. The 1984 edition of the Shore Protection Manual recommends this irregular wave height parameter to be $H_{1/10}$ (the average of the highest one-10th of the waves in a sea state), but other researchers recommend $H_{1/30}$ (e.g. Vidal et al., 1995). For the present study, $H_{1/20}$ is the irregular wave height parameter that yields the best correspondence between irregular and regular waves. However, this result is dependent on the length of the time series and on the number of times this time series is recycled to achieve a given damage level. The new wave height parameter H_n (the average of the n highest waves in a sea state) proposed by Vidal et al. (1995) takes into account the statistics of the large waves contained in the time series as well as the number of times this time series is recycled. The present study indicates that H_{250} (the average of the highest 250 waves in the sea state) is a suitable wave height parameter for characterizing breakwater stability under irregular waves.

1 Introduction

Over the years, several formulae have been proposed for predicting the hydraulic stability of armour layers; All of them are empirical and do not account for all of the parameters that affect the stability. One of these formulae is the well-known Hudson

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[*]   Danish Hydraulic Institute. Agern Allé 5, DK-2970 Hørsholm, Denmark.
[**]   Department of Civil Engineering. Aalborg University, Sohngaardsholmsvej 57, DK-9000 Aalborg, Denmark
[***] Canadian Hydraulics Centre, National Research Council of Canada, Ottawa, Ontario K1A 0R6, Canada.

formula, developed by R.Y. Hudson in 1958. Although this formula was developed on the basis of regular waves, it has been applied for irregular waves by replacing the regular wave height by the significant wave height H_s (i.e. $H_{1/3}$) of irregular waves.

The Shore Protection Manual (1984) recommends the use of $H_{1/10}$ instead of $H_{1/3}$. However, authors such as Vidal et al. (1995) claim that a wave height parameter such as $H_{1/10}$ or $H_{1/3}$ does not describe the large waves in a wave train sufficiently well. They argue that additional information is required on the length of the time series and the number of times it is recycled to achieve a given degree of damage. They also show that a new wave height concept H_n, based on the average of the n highest waves in a sea state, can account for the statistics of the large waves contained in the sea state as well as for the number of times it is recycled for achieving a certain degree of damage.

The main objective of the present study was to compare breakwater stability under regular and irregular waves in terms of measured damage. Whether or not H_n is a suitable wave height parameter for carrying out this comparison will also be established.

2 Experimental Set-up and Test Series

The experimental investigations were carried out in the Wave Research Flume at the Canadian Hydraulics Centre (CHC) of the National Research Council of Canada (NRC) at a Froude scale of 1:15 (Andersen et al., 1995). This flume is 97 m long, 2 m wide and 2.75 m deep. The wave generator in the flume uses active wave absorption, which allows the wave paddle to absorb reflected waves during wave generation thereby eliminating re-reflections (Davies et al., 1994a).

The tested structure was composed of a core, filter and an armour layer placed on a uniform slope of 1:1.8. The gradation of the armour stones was such that the ratio of $M_{max}/M_{min} \approx 2.5$. The median mass of the rocks, M_{50} was 2600 kg. A sketch of the tested structure is shown in Figure 1.

An array of five capacitance-type wave gauges was placed in front of the structure in order to separate the incident and reflected wave components. The applied reflection analysis was based on the least-squares technique described by Mansard et al. (1980). The incident sea state parameters obtained in the tests with regular and irregular waves are shown in Table 1 together with the number of repetitions of each sea state. For irregular waves the sea state was characterized by the incident significant wave height $(H_{s,i})$, derived from the incident wave spectrum and the spectral peak period, T_p. The incident sea state parameters for regular waves were also determined from reflection analysis.
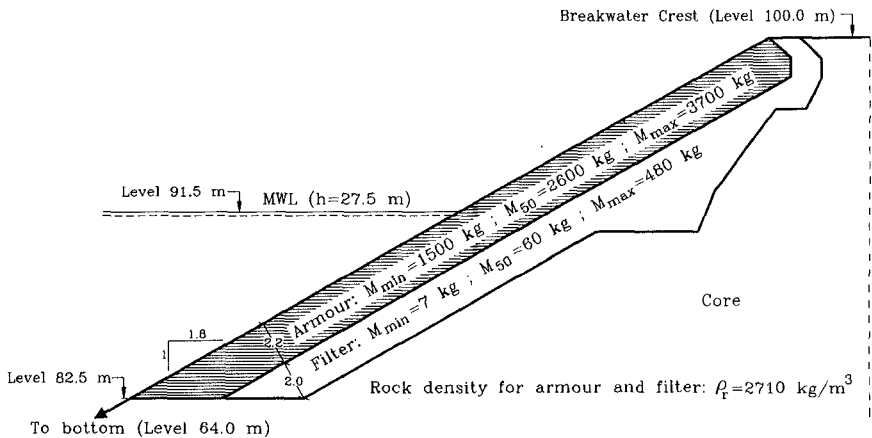
Figure 1 - Sketch of the tested breakwater cross-section (prototype units).

Irregular Waves			Regular waves		
$H_{s,i}$ (m)	T_p (s)	Repetitions	H_i (m)	T (s)	Repetitions
1.0	4.1	1	1.3	3.8	1
1.5	5.0	1	1.7	4.7	1
2.0	6.0	4	2.3	5.6	1
2.3	6.4	4	2.7	5.9	1
2.6	6.6	4	2.9	6.4	1
2.8	7.1	4	3.3	6.7	1
3.1	7.3	4	3.7	7.0	3
-	-	-	3.9	7.2	2
-	-	-	4.2	7.5	2
-	-	-	4.5	7.7	2

Table 1 - Incident sea state parameters for regular and irregular waves.

For each irregular wave condition a time series was synthesized using the random phase spectrum method, which combines the amplitude spectrum derived from the JONSWAP spectrum with a random phase spectrum (Mansard et al., 1994b). The prototype length of the time series for irregular waves was chosen to be relatively long (i.e. 2 hours) in order to minimize any of the potential variability in wave parameters often associated with shorter time series. Thus the time series contained 1200 to 2100 waves depending on the peak period of the sea state.

In order to ensure that the damage had stabilized under each wave condition, each time series was recycled four times resulting in approximately 5000 waves. Damage patterns generally stabilized after about 5000 waves. Since damage patterns develop much more quickly for regular waves, the length of the regular wave time series was

chosen to contain approximately 300 to 600 waves (depending on the period). For incident waves with $H_i > 3.3$ m it was necessary to repeat the time series 2 or 3 times in order for the damage to stabilize. This illustrates a fundamental difficulty with the concept of damage stabilization. The tendency for breakwater damage to stabilize and the time-frame within which this stabilization occurs are in fact directly related to the stability number, N_s.

Table 1 shows the wave parameters that were used in this study. These are the incident waves that were determined from reflection analysis. When generating the regular waves, an appropriate wave period had to be chosen for each wave height. Based on earlier works suggesting that either $H_{1/10}$ or $H_{1/20}$ are appropriate parameters when comparing regular and irregular waves, the wave periods for the regular waves in this study were chosen to be equal to the average of the wave periods corresponding to $H_{1/10}$ and $H_{1/20}$ of the corresponding irregular wave trains (i.e. the bivariate statistics were used).

3 Damage Measurements

After each test series the damage to the breakwater was estimated by computing the eroded area (A_e) using an electro-mechanical profiler. The performance of this profiler has been found to be very reliable and it has also been compared with the estimates of eroded area derived by counting the number of displaced armour stones (Davies et al., 1994b).
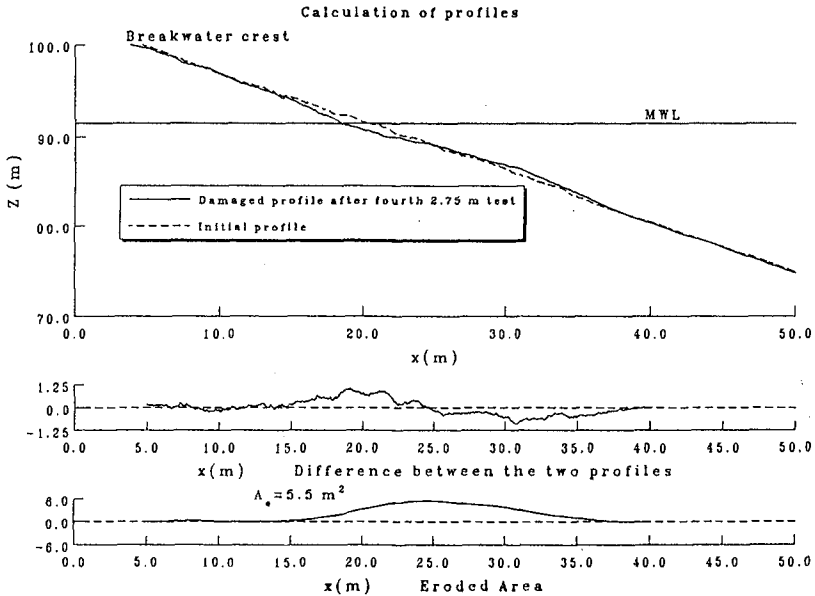
Figure 2 - Damaged breakwater profile after 4th irregular test with $H_{s,i} = 2.8$ m.

The damaged profile is determined as the average of nine evenly spaced profiles across the flume. The cross-sectional area of erosion is calculated by integrating the vertical difference between the damaged profile and the initial profile. Integrating from the crest of the breakwater, the eroded area is defined as the maximum value of this integral as a function of the distance x (see Figure 2). The eroded area is interpreted as the eroded volume/metre breakwater length.

When analyzing damage on rubble mound breakwaters, a dimensionless damage parameter S is often introduced:

$$S = \frac{A_e}{D_{n50}^2}$$

where A_e is the eroded area and $D_{n50} = (M_{50}/\rho_r)^{1/3}$ is the nominal diameter of the armour stones. The damage parameter can be visualized as the number of cubic stones of dimension D_{n50} eroded within a D_{n50} wide strip of the breakwater. $S = 2$ corresponds to the initiation of damage and is equivalent to the 0-5% damage defined in Shore Protection Manual (1984). Failure is defined as exposure of the filter layer and for a two diameter thick armour layer, this occurs for S-values of approximately 8 (van der Meer, 1988).

4 Evolution of Damage under Irregular Waves

In Figure 3 the damage parameter S is shown as a function of the number of waves for each of the four test sequences carried out.
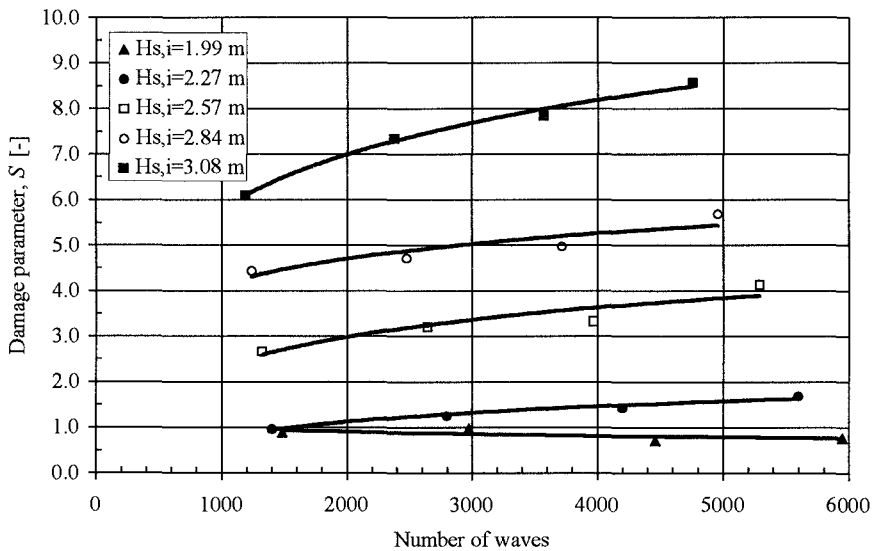
Figure 3 - *Evolution of damage for irregular waves.*

It can be seen from Figure 3 that the damage tends to stabilize in about 5000 waves. Based on approximately 50 tests with irregular waves, van der Meer (1988) proposed an equation for describing the damage evolution as a function of the number of waves. However, the results used to develop this equation were based on tests that started with zero-damage when the wave heights were changed. This means that the breakwater was rebuilt after each sea state. In this study the breakwater was not rebuilt after each severity of the sea state. Hence, the expression of van der Meer (1988) is not directly comparable to the evolution of damage shown in Figure 3. The evolution of damage is also seen in Figure 4 where the cumulative damage is plotted as a function of the cumulative number of waves.
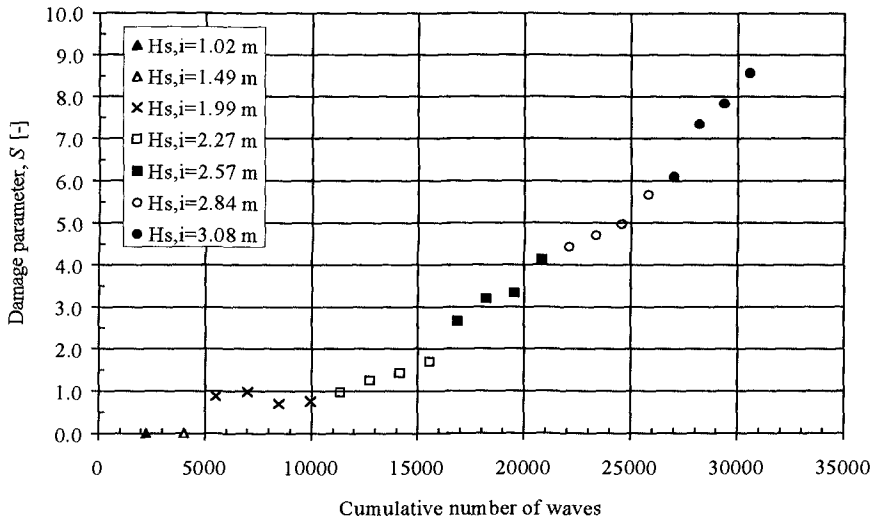
Figure 4 - Damage versus the cumulative number of waves (irregular waves).

5 Evolution of Damage under Regular Waves

According to Vidal et al. (1995) it takes about 100 to 400 waves for the damage to attain its equilibrium under regular waves. Hence, a 10 minutes long time series containing approximately 300 to 600 waves (depending on the period) was used in regular wave tests.

In Figure 5 the damage parameter S is shown as a function of the cumulative number of waves for the tests with regular waves. For $H_i = 3.7$ m the time series had to be repeated three times before the damage patterns stabilized. This indicates that the number of waves required to reach a state of equilibrium is larger than the value suggested in Vidal et al. (1995). Further tests should be carried out to verify this observation. It is possible that the number of waves required to stabilize the damage patterns can also vary with the degree of breakwater damage.
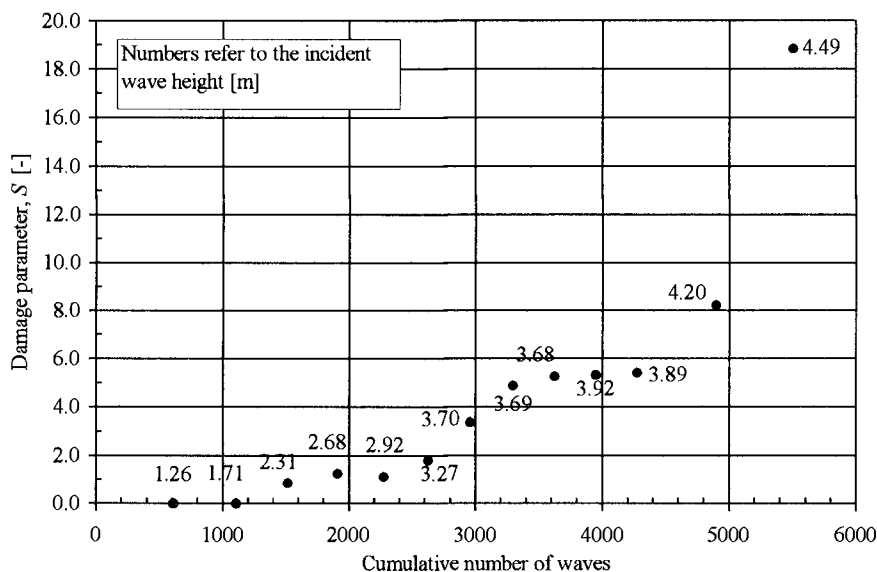
Figure 5 - Damage versus the cumulative number of waves (regular waves).

The regular wave tests showed that a damage level of $S = 2$ corresponds to a regular wave height of $H_i = 3.3$ m. In Hudson's formula, this would correspond to a value of $K_D = 4$, which agrees well with published values.

6 Comparison between Regular and Irregular Waves

In Figure 6 the damage is plotted against the stability number, N_s given by:

$$N_s = \frac{H}{\Delta D_{n50}} \quad ; \quad \Delta = \frac{\rho_r}{\rho_w} - 1$$

where H is the wave height and ρ_w is the mass density of water.

Figure 6 compares the damage caused by irregular waves characterized using four different wave height parameters in the calculation of the N_s parameter: $H_{1/3}$ ($\approx H_s$), $H_{1/10}$, $H_{1/20}$ and the average of $H_{1/10}$ and $H_{1/20}$.

A time-domain reflection analysis recently developed by Mansard (1994) showed that the average ratio between $H_{1/10}$ and $H_{1/3}$ and the ratio between $H_{1/20}$ and $H_{1/3}$ were 1.28 and 1.42, respectively, in the irregular wave tests. Since the corresponding values from the theoretical Rayleigh distribution are 1.27 and 1.40 respectively, this suggests that the incident wave heights were Rayleigh distributed.

From Figure 6 it appears that the best correspondence between the damage levels for regular and irregular waves is obtained when the irregular waves are

characterized by $H_{1/20}$. A comparison of the results of the present study with results of previous tests with the same breakwater is shown in Figure 7.
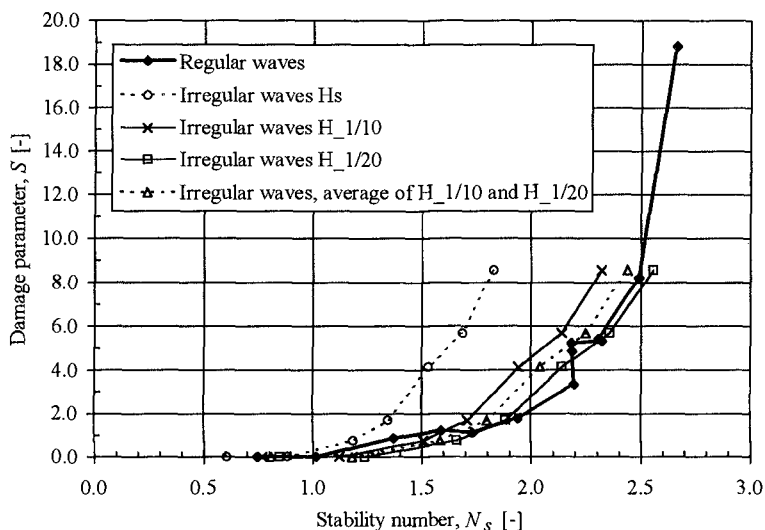
Figure 6 - Comparison between damage for regular and irregular waves.

In terms of Hudson's stability formula it is seen from Figure 7 that there is a good agreement between the results of the present study and the results obtained previously. The K_D-factor for this study using $H_{1/3}$ as the wave height parameter and $S = 2$ as damage level is approximately 1.4. In similar tests undertaken at CHC (e.g. Laurich et al., 1995) with the same breakwater the K_D-factor was found to range between 1.2 and 1.7.

Shore Protection Manual (1977) recommends the use of $H_{1/3}$ in Hudson's stability formula for design of breakwaters. The K_D-values suggested for breaking and non-breaking waves for rough angular quarry stone were 3.5 and 4.0, respectively (note that these K_D-values were established using only regular wave tests). Shore Protection Manual (1984) recommends the use of $H_{1/10}$ instead of $H_{1/3}$, and also recommends that the K_D-values for breaking and non-breaking waves should be changed to 2.0 and 4.0, respectively. The recommendation of $H_{1/10}$ in Hudson's formula rather than $H_{1/3}$ was supported by hydraulic tests by Feuillet et al. (1980); However, tests by Tanimoto et al. (1982) suggested a design wave height of $H_{1/5}$ when comparing regular and irregular wave tests. Furthermore, Allsop (1993) suggests that the application of $H_{1/10}$ in Hudson's formula is overly conservative.

For irregular waves, van der Meer (1988) proposed the following general damage equation:

$$N_s = a(K_D \cot \theta)^{1/3} \cdot S^b$$

where N_s is determined using $H_{1/3}$, θ is the slope of the armour, and a and b are empirical coefficients ($a = 0.7$, $b = 0.15$) determined by regression analysis. The K_D-value corresponds to Shore Protection Manual (1984), here the use of $H_{1/10}$ in the Shore Protection Manual is taken into account by the value of the a-coefficient. If this formula is applied to the results of the present study the damage to the breakwater is underestimated. This may be attributable to the fact that $H_{1/20}$ is a better parameter than $H_{1/10}$ for comparing regular and irregular waves for this structure (see Figure 6).
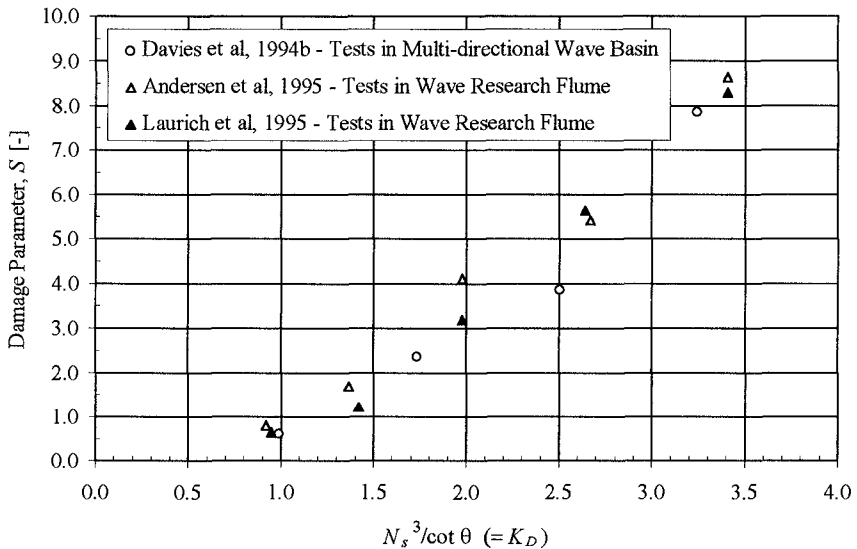
Figure 7 - *Comparison with previous studies. N_s is computed from $H_{1/3}$.*

Based on the results of a number of laboratory tests performed to study damage mechanisms of riprap, Ben Belfadhel (1993) found $H_{1/10}$ to be an acceptable wave parameter for use in stability formulae developed from regular wave tests. However, based on CHC data for steep slopes (1:1.5) Ben Belfadhel[*] has found that $H_{1/20}$ may be a more suitable parameter.

A comparison between the results of Vidal et al. (1992) obtained under irregular waves and the results established by Givler et al. (1986) using regular waves was carried out by Vidal et al. (1995). They found a good correspondence between the damage of regular and irregular waves with $H_{1/30}$ as the wave height parameter in the stability expression. However, from Figure 5 in Vidal et al. (1995) it seems that an

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[*] Personal communication between Dr. E.P.D. Mansard and Dr. B. Belfadhel.

even better correspondence could have been obtained if a wave height parameter closer to $H_{1/20}$ was applied.

7 Potential Sources of Inaccuracy

When dealing with irregular waves, the choice of an appropriate wave height parameter for use in the Hudson's formula is a continuous source of debate. Independent experiments by different researchers have failed to achieve consensus. Discrepancies between the findings of the various researchers could be partly due to different test methods used in different laboratories. Two potential sources of inaccuracies in experimental methods are described in the following.

Active Absorption of Waves

A commonly used technique for determining incident waves in flume tests is to calibrate sea states in the test flume with an efficient absorber in place before the breakwater is constructed, - thereby eliminating the need for sophisticated reflection analysis. During the breakwater stability tests, the damage is expressed as a function of the incident wave height obtained during this wave calibration procedure. Without active absorption, re-reflections can modify this incident wave height. This is discussed below:

Waves reflected by the breakwater propagate towards the wave generator and generally get re-reflected if the installation is not equipped with the capability for active absorption. These re-reflected components then propagate toward the breakwater as part of the incident waves. Depending on the differences between the phase angles of the original incident component and the re-reflected component, the net incident wave height attacking the breakwater could either be lower or higher than intended. Studies recently performed at CHC by Laurich et al. (1995) showed that active absorption can eliminate the inaccuracies introduced by these re-reflections.

The importance of using active wave absorption is shown in Figure 8, which shows the results of separate breakwater stability tests conducted with regular waves with periods of 2.0 and 1.94 s, respectively. No active absorption (position control) was used in these tests. For the test with $T = 1.94$ s, the phases of the re-reflected waves and the incident waves were such that they caused a higher than intended incident wave height at the structure (and correspondingly, a high degree of damage). The reverse was true when $T = 2.0$ s – the phasing of the re-reflected waves was such that incident wave energy (and damage levels) were reduced. If these test results were interpreted neglecting the effects of re-reflections, the damage levels seen in the two tests would have suggested two quite different conclusions. Using the results of the present study, the $T = 1.94$ s test would suggest that $H_{1/10}$ is the appropriate

wave parameter for comparison of irregular and regular waves, while the $T = 2.0$ s test would suggest that $H_{1/50}$ is more suitable.

This example shows that if active absorption is not used (or if the actual incident waves attacking the breakwater are not determined accurately), the interpretation of results could be quite different depending on the experimental installation.
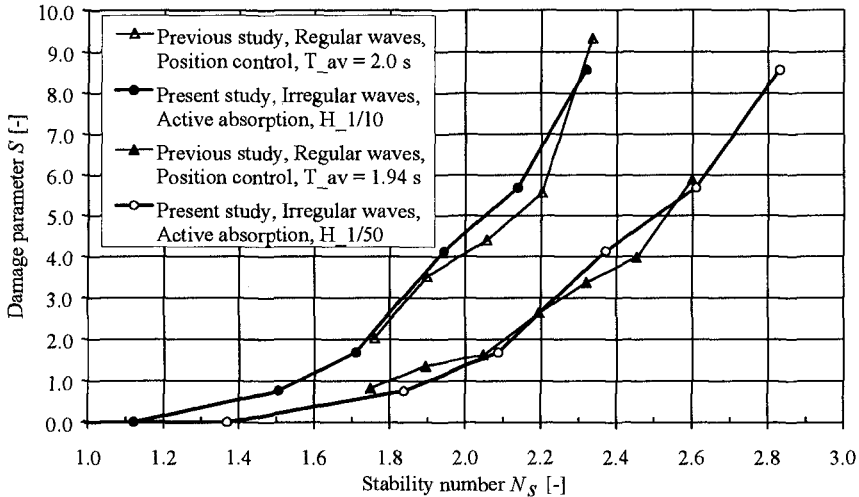
Figure 8 - *Comparison of damage between two regular and two irregular wave tests.*

Stabilization of Damage

In order to perform a reliable comparison between the damage obtained under regular and irregular waves, the rate of damage progression must be considered. Ideally one should compare regular and irregular wave tests at some equilibrium damage level. This can be difficult because true equilibrium is rarely attained in breakwater testing, however the rate of damage progression does slow with time. Choosing an equivalent number of cycles for regular and irregular wave tests is thus somewhat arbitrary. It takes fewer waves (i.e. 300 to 600) for the damage to stabilize if these waves are regular. In the case of irregular waves, 5000 waves or more are required to approach stabilization. It is important to make damage comparisons after ensuring that the level of damage "equilibrium" in the regular and irregular wave tests is equivalent. Otherwise misleading conclusions can be drawn.

9 Suitable Wave Height Parameter

Although $H_{1/20}$ has been identified as the most appropriate irregular wave parameter in this study, it does not consider the role of storm duration. Since each of the four

tests was carried out by recycling the same time series, the value of the $H_{1/20}$ parameter remains the same (i.e. $1.42 \cdot H_{m0}$), whether one or four cycles were used. Vidal et al. (1995) suggest a technique to incorporate both the effects of wave height and the number of waves to which a structure is exposed. They introduce H_n, which is defined as the average of the n highest waves used in achieving a certain damage. To apply this concept to the present study, H_n would be computed based on all four cycles of the time series used in the experiments. For instance, since a total of nearly 5000 waves were used in this particular study, the average of the highest one 20th corresponds to a wave height parameter H_{250}. Vidal et al. (1995) report that the parameter H_{250} describes not only the damage that has attained its equilibrium but also the intermediate ones.

To illustrate the suitability of the H_n parameter, Vidal et al. (1995) assumed that damage is proportional to the 5th power of the H_n wave height as follows:

$$S_{nz} = K_1 \left(\frac{H_n}{H_{1/3}} \right)^5$$

where S_{nz} is the damage after n_z waves, and K_1 is a constant. The damage can also be predicted by applying the following equation of van der Meer (1988):

$$S_{nz} = S_{5000} \, 1.3 \left[1 - \exp(-3 \cdot 10^{-4} \, n_z) \right] = K_2 f(n_z)$$

Equating these two damage levels gives:

$$\frac{K_2}{K_1} = \frac{\left(H_n / H_{1/3} \right)^5}{f(n_z)}$$

i.e., the right hand side of this equation should be constant for different n_z. Vidal et al. (1995) show that for Rayleigh distributed wave heights an appropriate value for n is approximately 100. It is presumed that this proportionality is applicable to a breakwater section that was tested if it had undergone no damage by previous wave heights. Since in this study, lower wave heights have already caused some initial damage, a somewhat different form of the relationship was anticipated. Consequently, both linear and parabolic relationships were considered in determining a suitable relationship, i.e.

$$S \propto H_n \quad and \quad S \propto H_n^2$$

For each cycle, the wave height parameter ($H_{1/n}$) is determined corresponding to the actual damage level (Rayleigh distribution of wave heights is assumed. This assumption has been verified in the wave analysis). Since $H_{1/20}$ was shown to be the best parameter in Figure 6, it is also used in the following illustration:

For example: after the 1st cycle of $H_{s,i} = 3.1$ m the damage level is 72% of the corresponding equilibrium damage level. Hence the wave height parameter is $0.72\,H_{1/20}$ for the linear assumption and $\sqrt{0.72}\,H_{1/20}$ for the parabolic assumption, corresponding to $H_{1/3.0}$ and $H_{1/6.8}$, respectively, using the Rayleigh distribution. Based on the cumulative number of waves (determined from the mean period) the corresponding n-value can be determined for H_n.

Using this approach for the present study, the n-value can be determined for all the test series (i.e. including the intermediate damage levels). The average value of n was found to be 250 with a coefficient of variation of approximately 10 %. This indicates that the H_n parameter is a more suitable parameter to characterize breakwater damage than the conventional $H_{1/n}$ (e.g. $H_{1/10}$ or $H_{1/20}$) which does not take into account the number of waves.

10 Conclusions

The best correspondence between regular and irregular waves in terms of the damage is obtained when the irregular waves are characterized by $H_{1/20}$. Other researchers have proposed e.g. $H_{1/10}$ and $H_{1/30}$ to characterize irregular waves. These different results may be associated, to a certain extent, with variations in test methods.

Vidal et al. (1995) proposed the H_n concept to characterize irregular waves. This concept, defined as the average of the n highest waves, has the advantage of including the length of the time series, the number of times it is recycled, and the statistics of the highest waves. In this particular study, an n-value of approximately 250 was found to be suitable. Further research is necessary to verify the suitability of H_n to characterize breakwater damage under irregular waves and to determine a suitable n-value.

Acknowledgements

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