WAVE REFLECTION FROM LOW CRESTED BREAKWATERS

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SUMMARY

Low crested breakwaters are rubble mound structures with crest underwater or slightly emerged, so to be easily overpassed by incoming waves. Recently they have received a growing attention, mainly because of their low intrusion in the coastal landscape. The frequent employment of such structures indeed requires that any aspects of the wave-structure interaction is properly investigated. This paper deals with wave reflection, which represents a very important item, being related to scour phenomena occurrence in front of the barrier.

The research discusses results of a series of regular wave experiments conducted at the laboratory of the Hydraulic and Environmental Engineering Department "G. Ippolito", of the University of Naples "Federico II". Scope of these experiments is achieving a better insight on the role played by some of the most important wave/structure parameters on reflection properties of low crested breakwaters.

1. NATURE OF THE PROBLEM

Low crested breakwaters (LCB, Figure 1) are rubble mound barriers constructed with the crest close to the mean sea level (submerged or slightly emerged), so to be frequently over passed by incoming waves. Recently they are receiving a growing attention, mostly because of their capability of meeting both the requirements of shore erosion control and landscape safeguarding. Accordingly, significant efforts are being addressed at predicting quantitative features of wave-structure interaction. Most of researchers focused on wave transmission (van der Meer et al., 2005, Buccino and Calabrese, 2007), that is to say on the fraction of incident wave energy transferred in the shadow zone; yet a number of supplementary hydraulic items deserve to be adequately investigated. Among them, wave reflection from the structures represents a major variable, as it enhances horizontal water velocities in front of the barriers, possibly provoking severe scour events at the toe of the breakwaters (Whitehouse, 1998).

Surprisingly, reflective properties of LCB have been little researched in the past, although a reasonable body of literature exists for both beaches and non overtopped structures (e.g. Miche, 1951, Ahrens, 1980, Seelig and Ahrens, 1981 a and b, Ahrens and Seelig, 1993). Only recently, Zanuttigh and Lamberti (2004) and Zanuttigh and van der Meer (2006) proposed, in the frame of the EU funded research project DELOS, two empirical formulae for calculating the fraction of wave energy reflected back from LCB, as a function of both structure and wave parameters.

Nevertheless, to calibrate the formulae, the authors gathered experiments not really designed to study wave reflection; consequently governing parameters may have been not varied enough and this may affect reliability of design equations. For this reason a new series of experiments have been designed and conducted at the small scale wave flume of the University of Naples (UoN) “Federico II”. Both wave and structure parameters (wave height and period, crest freeboard, crown width, front slope) were systematically varied to get an ensemble of nearly 600 tests. Moreover only regular wave attacks have been run, to facilitate the comprehension of basic physical mechanisms ruling the phenomenon. The paper discusses results of these experiments.

2. LITERATURE

Wave reflection from sloping coastal structures, including both beaches and breakwaters, has long been studied through over 60 years. One of the earliest research is that by Miche (1951), who faced the case of steep monochromatic breaking waves on impermeable sloping beaches. The author found the proportion of reflected energy, expressed by the reflection coefficient $K_r$, to be proportional to the following non dimensional parameter, $M$ (Miche number):
\[
M = \frac{4g \tan^2 \alpha}{(2\pi)^2 (H_i f^2)},
\]

(1)

where \( \tan \alpha \) is beach gradient, \( H_i \) is incident wave height and wave frequency \( f \) is reciprocal to wave period \( T \). More in details, the reflection coefficient represents the reflected to incident wave height ratio and consequently, in the frame of a low order wave theory, the root square of the reflected to incident wave energy (being wave energy proportional to \( iH \)).

Thus we have:

\[
K_r = \frac{H_r}{H_i} = \sqrt{\frac{E_r}{E_i}}.
\]

(2)

The author reasoned that,

\[
K_r = 1 \text{ if } M \geq 1,
\]

(3)

\[
K_r = M \text{ if } M < 1.
\]

Battjes (1974,1975) rearranged parameters of Miche number, making reflection coefficient proportional to the surf similarity number,

\[
\xi = \frac{\tan \alpha \cdot \sqrt{g \cdot T}}{\sqrt{H_i} \cdot 2\pi}.
\]

(4)

The following equation valid for regular waves on impermeable slopes was given:

\[
K_r = 0.1\xi^2.
\]

(5)

Ahrens (1980) investigated the reliability of Miche and Battjes formulae for random wave experiments; the author came to the conclusion that Battjes equation provided more accurate estimates of \( K_r \) than Miche’s one, but still overestimated experimental data for \( \xi > 3 \). The author employed significant wave height \( H_s \) and peak period of wave spectrum to calculate surf similarity number in random wave contests. This will be indicated as \( \xi_p \) hereinafter.

As far as rubble mound non overtopped breakwaters and jetties are concerned, Seelig (1983) furnished the following equation as a conservative estimate of \( K_r \) for design purposes:

\[
K_r = \frac{a_i \xi_p^2}{b_i + \xi_p^2},
\]

(6)

where \( a_i = 0.6 \) and \( b_i = 6.6 \) for rock armour units.

Postma (1989) performed 298 random waves experiments on trapezoidal rubble mound non overtopped breakwaters. Most of them were conducted in a small scale, whereas nearly 10 experiments were performed in the large scale \textit{Delta Flume of Delft Hydraulics}, the Netherlands. In the small scale tests permeability of the structures were changed while in the large scale experiments the structure cross section was the same for the overall set.

The author came to the following equations:

\[
K_r = 0.140\xi_p^{0.73};
\]

(6)

\[
K_r = 0.081P^{-0.14} \cdot \tan^{0.78} \cdot s_p^{-0.44}.
\]

(7)

The former closely resembles Battjes equation while the latter separately considers structure slope and wave steepness \( s_p = \frac{H_u \cdot 2\pi}{g \cdot T^2} \), and includes the notational permeability \( P \) (van der Meer (1988)). It is to remember that \( P \) equals 0.6 for homogeneous breakwaters, 0.5 for two layers cross sections made up on armour and permeable core, 0.4 for armour + under layer + permeable core and 0.1 for armour + under layer + impermeable core.

As mentioned in the previous section, the problem of predicting wave reflection from low crested rubble mound structures has been addressed only in the most recent times. Within the EU project DELOS, two distinct formulae have been carried out, one by Zanuttigh and Lamberti (2004) and the other by Zanuttigh and van der Meer (2006). The former relies on a multiple regression analysis applied to a huge amount of experimental data including structures with different types of armour units. Herewith the expression valid for rock armoured breakwaters is given, as reported in Calabrese et al. (2005):

\[
\ln K_r = 0.95 \cdot \ln \frac{H_u}{D_{so}} + 0.19 \cdot \ln \frac{h_c}{d} - 0.25 \cdot \ln s_p - 2.45.
\]

(8)

Equation (8) holds for \( -1.5 < \frac{R_c}{H_{ui}} < 1.5 \) and for structures not placed in the surf zone, i.e. for \( \frac{H_u}{d} \geq 0.6 \).

The Zanuttigh and van der Meer formula for rock armoured breakwaters reads:

\[
K_r = \tanh(0.75 \cdot \xi_p^{1.5}) \cdot \left( 0.37 \frac{R_c}{H_i} + 0.67 \right).
\]

(9)

It’s valid for:

\[
-1 \leq \frac{R_c}{H_i} \leq 0.5, \text{ and } I \leq \frac{H_i}{D_{so}}, \text{ } \xi_p \geq 0.01.
\]
Note that $\xi_m$ uses the fictional mean wave steepness, $s_m$, calculated by means of significant wave height and mean spectral period $T_{m,10}$ defined as the ratio between the spectral moment of order -1 and the spectral moment of order 0.

3. LABORATORY STUDY

3.1. EXPERIMENTAL SETUP

Experiments have been conducted in the small scale wave channel of LINC Laboratory of the Hydraulic and Environmental Engineering Department “G. Ippolito” of the University of Naples “Federico II”. (Figure 2).

![Wave flume](image)

Figure 2. Wave flume.

The flume is 23.50m long, 0.80m wide, 0.75m deep and is equipped with a piston-type wave generator, capable of producing both regular and irregular waves. Moreover, the facility is provided with a dynamic wave absorber. At the end of the flume opposite to the wave maker, an absorbing gravel beach was mounted with a convex shape to enhance its performances (Svendsen, 1985).

Most of tests used homogeneous models of rubble mound breakwaters, approximately at 1:20 Froude scale. Further experiments have been conducted on a smooth impermeable cross section, but results of these experiments will be not commented in the following. As far as homogeneous permeable breakwaters are concerned, five crest freeboard $R_C$ (-0.05m, -0.025m, 0m, 0.025m, 0.05m), two crown width $B$ (0.35m and 0.80m), two front slope $\tan\alpha_{f}$ (1:2 and 1:5) have been employed, to get twenty different models on the whole. Rear slope (1:1.5) and the grade of rock ($D_{50} = 0.061m$) were kept constant; most of experiments were conducted with a 0.315m still water depth ($d$), whereas nearly 50 tests were performed using 0.29m and 0.34m. In situ porosity of the breakwater resulted around 0.45.

For each structure model, 3 series of regular wave attacks were run using 3 different period (1s, 1.5s, 2s). For each period, wave height was increased from 0.02m to 0.12m at a step of 0.01m. These conditions allowed, for submerged breakwaters, to analyse wave reflection even if wave breaking at the structure did not occur.

3.2. WAVE SURFACE ACQUISITION AND ANALYSIS

Fluctuations of water surface were acquired through an array of five twin wires resistive probes; three of them were placed in front of the structures for the reflection analysis, while two devices were mounted leeward the barrier to study the transmitted wave field. The probes were sampled at 25 Hz and time series were recorded for 60s. Following Goda and Suzuki (1976), the minimum distance between the toe of the structure and the nearest probe of the reflection array was left larger than 0.2$L$, being $L$ the local wavelength calculated by linear dispersion relationship.

Wave records were first “Hanning windowed” to suppress spectral leakage and then subjected to Fast Fourier Transform (FFT) after applying a broadband power correction factor to compensate for artificial power loss induced by the window weighting.

In principle, the simultaneous acquisition of water surface allows separating incident and reflected waves by means of a number of different methodologies (Di Pace, 2006, Isaacson, 1991). Among them, the widely used Goda and Suzuki (GS, 1976) and Mansard and Funke (MF, 1980) methods have been here employed. The former needs, for regular waves, two fixed probes while the latter requires three elevation measures for both monochromatic and spectral waves. Each of these techniques returns both the incident and the reflected wave spectrum.

Thus the total power of incident and reflected signals can be calculated and reflection coefficient can be estimated as the root square of the reflected to incident power. This because for weakly non-linear waves spectral power basically corresponds to wave energy. GS technique has been applied to each of the three couple of probes and values of $K_r$ were averaged. For the entire ensemble of data, this average was found to agree quite well with the values coming from MF (an example is given in Figure 3); for this reason the final estimate of $K_r$ for each test has been taken as the mean of GS (previously averaged) and MF methods.
4. RESULTS

4.1. PARAMETRIC ANALYSIS OF EXPERIMENTAL DATA

Wave reflection is a compound outcome of wave-structure interaction; it may be formally described by the following dimensional equation:

\[ H = f(H_i, T, R_c, B, d, \tan\alpha, D_{50}, g, \rho, \mu) \]  

where, besides the symbols already introduced, \( g \) is gravity acceleration, \( \rho \) is water mass density, and \( \mu \) represents dynamic viscosity of the fluid. By invoking Buckingham theorem, and after some rearrangements of variables, equation above can be rewritten as:

\[ K_r = \left( \frac{gT^2 R_c}{D_{50}^5}, \frac{B}{D_{50}}, \frac{d}{D_{50}}, \tan\alpha, \frac{D_{50}}{H_i}, \frac{\mu}{\rho g D_{50}^2} \right)^{1/2} \]  

which is suitable to our purposes since the median diameter of rock units was kept constant in the experiments (\( D_{50} = 0.061m \)). This allows commenting the role of most of variables at the right hand side of Equation (10), one by one.

However, it must be recalled that permeability of structures, represented by the last parameter at the right hand side of Equation (11), has not been changed in the experiments; thus its effect cannot be evaluated.

Similarly, the non dimensional water depth \( \frac{d}{D_{50}} \) was not varied systematically, so that only approximate conclusions can be derived about it.

Influence of crest freeboard \( R_c \) and incident wave height \( H_i \)

It is intuitively expected that the larger is crest freeboard, the higher \( K_r \). This was detected in the majority of cases here analysed, although effects of clearance seems to be more complicated than one may expect. As an example, Figure 4 shows results obtained for \( B = 0.80m, \tan\alpha_{off} = 0.5, T = 1s \). Influence of crest freeboard is well clear for small waves; here the emerged structure is near non overtopped and potential energy of run up wedge is completely disposable for reflection through run down. This obviously leads to high \( K_r \). As soon as wave height increase part of run up wedge is transferred behind the structure by overtopping and, consequently, reflection coefficient diminishes.

Influence of incident wave height and crest freeboard (\( B = 0.80m, \tan\alpha_{off} = 0.5, T = 1s \)).

On the other hand for the submerged structure \( K_r \) remains low and almost independent of \( H_i \) for small waves; this is consistent with findings of a number of researchers, who investigated reflection from...
submerged obstacles from both experimental and theoretical points of view (e.g. Longuet-Higgins, 1967). On contrary, when the incoming wave height increases, reflection coefficient seems to become somehow proportional to $H_i$ (submerged breakwaters). This probably why when wave height is large compared to submergence, the structure doesn’t remain underwater over a wave cycle; the wave trough makes the structure emergent for a while and the amount of water mass transferred in the protected area starts to be reduced by the structure height. So, wave transmission is lowered and, consequently, wave reflection is enhanced. The two opposite trends by which $K_r$ varies with $H_i$, makes the response of submerged and emerged breakwaters quite close each other for large waves; obviously their behaviour will approach that of a no freeboard structure ($R_c = 0$). This would support the use of the relative crest freeboard $R_c / H_i$ as a governing parameter for $K_r$ (Zanuttigh and van der Meer, 2006); in fact with increasing wave height $R_c / H_i$ tends to zero whether for submerged ($R_c < 0$) or protruding ($R_c > 0$) barriers.

Finally, it should be noted that from a hydrodynamic point of view large waves correspond (for given front slope and wave period) to the occurrence of plunging breakers either on the crest or on the front face of the barrier. Hence, the aforementioned reduction of crest freeboard influence on $K_r$ may occur when heavy plunging breakers take place. This seems to be partly corroborated by the fact that for more gentle slopes, where plunging occurrence is more probable, influence of $R_c$ resulted somehow less pronounced (Figure 5).

**Influence of depth of placement $d$**

As mentioned before, water depth in front of the structure was not systematically varied in the experiments. Accordingly, no general conclusion can be drawn in this regard. Yet in a number of cases the authors found water depth to cancel out effect of crest height, indicating that $K_r$ possibly increases as soon as waters become shallow (Figure 6). However, new systematic investigations are recommended for deepening this matter.

**Influence of incident wave period $T$**

Results basically confirm findings of previous studies that indicated incident wave period as one of the major variables ruling wave reflection. As shown in Figure 7, $K_r$ enlarges with increasing wave period. Note that in the graph the deepwater wavelength $L_0 = \frac{gT^2}{2\pi}$ has been used to make $T$ non dimensional.

**Influence of front slope $\tan \alpha_{off}$**

Commenting influence of front slope, submerged ($R_c < 0$) and emerged ($R_c \geq 0$) breakwaters should be separately considered. For underwater barriers the ratio between the submergence and the incoming wave length seems to be crucial. If the structure is well submerged and incoming waves short enough, wave reflection seem scarcely influenced by front slope (Figure 8). With increasing wave period the relative submergence $R_c / L_0$ decreases in such a way that a significant set-down would be produced; the reduction of the mean water level makes possible the wave trough to expose the structure, so that the incoming waves must necessarily “feel” the slope (Figure 9). Consequently $\tan \alpha_{off}$ has a strong influence on wave reflection for low submergences and long waves. On the other hand for protruding breakwaters front slope angle is a leading parameter for whatever value of wave period; this because waves break on the front slope and hence it is indeed affected by it (Figure 10).
This probably because crown width is involved in many aspects of the wave structure interaction, some tending to increase wave reflection and others tending to diminish it. For submerged permeable breakwaters an increase of $B$ leads to an increase of the surface of contact between waves and structure; this enhances wave reflection. On the other hand the longer the structure the greater are dissipations in the porous material, that reduces $K_r$. For emerged breakwaters wave overtopping is undoubtedly reduced when crest width increases. Consequently, more water mass is theoretically disposable for run down, and wave reflection would increase. Nevertheless, dissipation experienced by water wedge in the down rush process acts in the opposite sense, cancelling out the former effect. An example of results obtained in the analysis is shown in Figure 11. Further experiments are suggested to better analyse this point.

**5. CONCLUDING REMARKS**

The paper has presented results of an experimental investigation conducted of LINC Laboratory of the Department of Hydraulic and Environmental Engineering “G. Ippolito” of the University of Naples “Federico II” with the purpose of studying reflection properties of frequently overtopped breakwaters. The tests included a good deal of breakwater configurations, ranging from submerged to exposed (rarely overtopped). The same regular wave attacks were run for each structure to assess the influence of wave/structure parameters one by one. The analysis indicated that front slope angle, and incident wave period should be considered as leading parameters for wave reflection process. Moreover, the authors have the feeling that depth of placement also plays a relevant role, although this variable was not systematically changed during experiments; this obviously makes previous conclusion not sufficiently supported by data.

One of the most interesting aspects of the analysis is about the influence of crest freeboard and incident wave
height. Results seem to suggest that influence of $R_c$ is significant for small waves and reduces for larger waves (Figure 4). At the same time reflection coefficient seems to lower with incident wave height for emerged breakwaters, whilst, for the submerged ones, a weakly increasing trend was detected. For zero freeboard influence of incident wave height appeared negligible. Altogether these results support the proportionality between $K_r$ and the non-dimensional crest freeboard $R_c / H_i$. Since for submerged breakwaters $R_c$ is negative, with increasing wave height $R_c / H_i$ increases as well, together with reflection coefficient. This is consistent with data shown in Figure 4. Similarly, for protruding breakwaters $R_c$ is positive, and hence, with increasing wave height, $R_c / H_i$ decreases together with $K_r$. Finally, for large waves $R_c / H_i$ tends to zero whether for emerged or submerged structures and this would justify the fact that for high waves, $K_r$ seems less affected by $R_c$. It has also been noted that influence of crest freeboard seem less pronounced for gentle slopes (Figure 5). This suggest the idea that crest freeboard becomes somehow less effective to wave reflection when heavy plunging breakers take place at the front face or onto the crown of the breakwater.

Figure 12 shows the comparison between present data and Zanuttigh and van der Meer formula (2006). It is to remember that this equation has been derived for random waves and so its application to periodic waves must be done with some caution. However, a certain scatter appears.

![Figure 12. Performance of Zanuttigh and van der Meer (2006) formula.](image)

Altogether the authors believe this to be due to a persistent lack of knowledge on the hydraulic processes which govern wave reflection. Hence parametric studies like the one we have here presented may significantly aid to cope with this problem and improve reliability of design equations.

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