INTRODUCTION

Field measurements have been carried out on a breakwater built up to protect a water intake dockyard for the power plant of Montalto di Castro (Tyrrenian Sea, Italy). Cover consists of two strata of rocks, randomly placed by land based equipment, with average diameters equal to about 1 meter. The weight of primary armour ranges from 2 to 4 tons and the seaward slope is 1:2. The water depth in front of the structure is about 5 m. The structure is faced by one Waverider Datawell and one Wave-Track directional Endeco: the first installed in 1978, the second in 1986. From December 1991 Wave Track Endeco was replaced by a directional wave rider Datawell.

Breakwater and instrument position are illustrated in Figure 1. A typical section of investigated structure is shown in Figure 2. A field analysis of breakwater behaviour is necessary to evaluate the capability of commonly used design formulae for predicting the damage level, due to wave attacks, attained by mound structure. Photogrammetric techniques combined with adequate ground control represents a common method for monitoring breakwater damage progression (Ackers, 1983; Gebert, 1984; Kluger, 1982; Pope, 1983; CETN, 1984).

Accurate field investigations relied upon the use of photography, supported by topographic measurements. Both metric and non-metric cameras were used to photograph the breakwater cover layer. Metric photographs were analysed using a stereoscope; this allowed both a better evaluation of cover blocks movements and a good estimation of the rotation of visible stones.

An accurate time and frequency domain study was made on recorded wave climate; moreover wave grouping aspects, its statistical properties and possible effects on breakwater stability were analysed. Recorded wave parameters were introduced in four commonly used stability theories. Comparison between the empirical design methods and prototype behaviour were made and found to be satisfactory. Analysis of wave records and results of field inspections were also used to calibrate a computer program for predicting breakwater damage progression. By this model, damage progression can be estimated as a function of real sea state records.

THEORETICAL DAMAGE PREDICTION

Breakwater stability analysis has been made over the past 30 years by using the Hudson formula (Hudson, 1959). In recent years, experiments in large wave flumes with irregular waves and careful analysis of prototype damage have led to the formulation of new theories which comprehend random wave characteristic effects. Losada and Gimenez-Curto developed an exponential model as a function of surf similarity parameters to represent rubble-mound breakwater stability (Losada and Gimenez-Curto,
For an exact significance of $E_{sumj}$, the reader is referred to Sawaragi, 1985. The linear relationship between sum energy of grouped waves ($E_{sumj}$) and the spectrum peak parameter ($Q_p$) has been verified using wave data recorded at Montalto di Castro. Both parameters have been evaluated at wave rider depth. Computed values of $E_{sumj}$ were very similar to those estimated using

$$A and B are coefficients with values that are the function of foreshore slope and type of cover layer block. In plan H, T expression (1) could be plotted as an “interaction curve”, equal to a set of the points which produce the same value of armour unit weight (Losada, 1980).

More recently Sawaragi and Ryu (Sawaragi, 1983) have analyzed in detail group effects on mound stability. These authors have discovered that a strong correlation exists between GODA spectral peakedness parameter $Q_p$ and the energy sum of grouped waves $E_{sumj}$ (Ryu, 1986). This relationship is expressed as follows:

$$E_{sumj} = H_s (0.042 Q_p + 0.125) \quad (2)$$

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Equation 2. The comparison is reported in Figure 3.

Irregular wave experiments showed that the percentage of damage suffered by breakwater (D%) is a function of the above-mentioned parameters according to the following expressions:

\[ D\% = 153.8 \left( \frac{E_{sumj}}{\gamma \cdot I_{z} \cdot tga \cdot \tau} \right) - 30.1 \]  

\[ D\% = \left( \frac{\chi \cdot H_{s}^2 \cdot tga \cdot 6.15 \cdot Q_{p} + 20}{\gamma^3 \cdot tga^2 \cdot P_3^2} \right) - 30.1. \]

SAWARAGI defines the degree of damage as the percentage of destroyed volume with respect to the total volume of cover layer. According to Sawaragi, a displacement takes place when armor units are moved over a distance greater than the overall size of blocks (SAWARAGI, 1983).

More than 300 experiments were done by VAN DER MEER to analyse the effect of storm duration, wave period, group characteristics and spectral shape (VAN DER MEER, 1987, 1988). More recently, stone shape and layer thickness influence were also analyzed using an expression comparable with the VAN DER MEER formulation (BRADBURY et al., 1990) and new stability formulae were found for both overtopped and submerged breakwaters (VAN DER MEER, 1991). Van der Meer found two different relationships to determine stable rock block dimensions, for plunging and surging waves.

Plunging waves \((I_{z} < I_{c})\)

\[ \frac{H_{s}}{\Delta D} \cdot \sqrt{I_{z}} = 6.2 \cdot p^{0.18} \cdot \left( \frac{d}{\sqrt{n}} \right)^{0.2}. \]  

Surging waves \((I_{z} \geq I_{c})\)

\[ \frac{H_{s}}{\Delta D} = p^{0.13} \cdot \left( \frac{d}{\sqrt{n}} \right)^{0.2} \cdot \sqrt{\cot \alpha} \cdot (I_{z})^{p}. \]

Transition from plunging to surging is given by the following expression:

\[ I_{c} = \left( 6.2 \cdot p^{0.21} \cdot \sqrt{tga} \right)^{1/\left( p + 0.5 \right)}. \]  

Using VAN DER MEER’s theory, a mathematical method capable of describing revetment damage progression has been calibrated. This analysis is possible by introducing a parameter that can be referred to as “equivalent sea state”. This is equal to a time parameter describing the duration of a fixed sea state capable of causing the same amount of damage of different wave climate conditions

\[ D_{1} = a \cdot \left( \frac{d \cdot p^{-0.2}}{H_{s}^2} \right)^{2} \cdot \left( \frac{\Delta D^2}{H_{s}} \right)^{2} \cdot T_{m}. \]  

\[ D_{2} = b \cdot \left( \frac{d}{p^{0.66}} \right)^{2} \cdot \left( \frac{I_{z} \cdot \cot \alpha}{H_{s}^2} \right)^{5} \cdot (\Delta D^2)^{0.5} \cdot T_{m}. \]

where \(D_{1}\) and \(D_{2}\) are for plunging and surging waves respectively and \(a\) and \(b\) are dimensional coefficients depending on the units used in the above formulas.

The following steps were used to implement the stability model:

1. Evaluation of data from gauge records and calculation of wave parameter at the front of the structure depth;
2. Computation of minimum sea state capable of causing significant damage to the structure, in order to attain a lower limit below which the wave data recorded can be disregarded;
3. Interpolation between sea states and computation of VAN DER MEER’s damage parameter with free interval time steps.

All these operations have been implemented on a Vax Digital Computer. Superimposition of the effects of a sequence of sea states, such as those registered by wave gauges, and the damage description starting from whatever revetment condition is thus modeled.

SITE INVESTIGATION

The investigated breakwater is provisional and undergoes large damage during severe storms.
From autumn 1984 the structure has been regularly surveyed by contractors. An accurate zonation of the cover layer according to the degree of suffered damage and the number of blocks necessary to rebuild the original sections have been registered at the end of each major event. Since 1988, a more accurate survey of damage has been programmed. Two stretches of the structure measuring about 15 meters each have been delimited by permanent rods and more visible blocks at different levels have been marked by water resistant paints (Figure 4).

Since no underwater inspection was scheduled the following considerations refer only to the above water level part of the structure. However it is believed that this information provides a reasonable indication of the general condition of the breakwater cover layer. Once every two months,
RESULTS AND DISCUSSION

The sea state characteristics of the major storms registered have been treated with the above-mentioned methodology. Bathimetry is regular in front of the structure and bottom slope is equal to 1:80. The GODA technique for random sea waves was applied for transforming Waverider information. The photographs have also been viewed in pairs under a stereoscope; this provided a three-dimensional image of the breakwater primary layer and good estimation of rotation of all visible blocks. A higher degree of accuracy in cover blocks displacements estimation was possible by using the photogrammetric workstation; this was an appreciable advantage for the evaluation of global damage.

Field surveys were made on the following dates:

<table>
<thead>
<tr>
<th>Topographic</th>
<th>Photographic</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 14 1988</td>
<td>July 14 1988</td>
</tr>
<tr>
<td>Nov. 02 1988</td>
<td>Nov. 05 1988</td>
</tr>
<tr>
<td>Jan. 11 1989</td>
<td>Jan. 27 1989</td>
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<td>Mar. 06 1989</td>
<td>Mar. 15 1989</td>
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<td>May 02 1989</td>
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<td>June 12 1989</td>
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<td>July 06 1989</td>
<td>July 10 1989</td>
</tr>
<tr>
<td>Nov. 15 1989</td>
<td>Oct. 21 1989</td>
</tr>
<tr>
<td>Jan. 10 1990</td>
<td>Jan. 10 1990</td>
</tr>
<tr>
<td>June 26 1990</td>
<td>July 20 1991</td>
</tr>
<tr>
<td>July 09 1991</td>
<td>July 18 1992</td>
</tr>
<tr>
<td>July 16 1992</td>
<td></td>
</tr>
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</table>

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<td>July 10 1989</td>
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</tr>
<tr>
<td>July 16 1992</td>
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</tbody>
</table>

The strongest storms were recorded in January and November 1987 and at the end of March 1988. Significant wave heights and average zero crossing period evolution are reported in Figures 5 and 6. The cover layer, which was seriously damaged, was repaired at the end of these storms. Frequency and zero up-crossing time domain analysis of wave gauge records were made. An accurate study was also carried out to analyse wave grouping aspects registered by Datawell Waverider. The lengths of runs were determined for two threshold values: the significant wave heights and average heights of the highest one-tenth waves. Possible resonance conditions were analyzed according to Sawaragi and Bruun and Gunbak theories (Sawaragi, 1985; Bruun and Gunbak, 1978). Thus a continuous knowledge of structure behaviour and wave regime was available.

RESULTS AND DISCUSSION

The sea state characteristics of the major storms registered have been treated with the above-mentioned methodology. Bathimetry is regular in front of the structure and bottom slope is equal to 1:80. The GODA technique for random sea waves was applied for transforming Waverider information.
from deep to shallow water (Goda, 1975, 1985). A comparison between predicted and measured damage was done for the major storms registered. The Hudson, Losada, Sawaragi, and Van der Meer theories have been applied to compute theoretical damage level. Two severe events were registered in January 1987 and March 1988. The results of analysis of wave records for the March 1988 storm are shown in Table 1. In both cases the structure suffered very severe damage. Run lengths of groups exceeding significant and average of highest one-tenth waves were analyzed. During the March 1988 storm, nearly all groups with heights over significant ones were within Sawaragi critical region.

Lengths of runs greater than the significant wave height registered by Waverider are the same if normal or critical groups are considered (Figure 7).

During the March 1988 storm, the breakwater suffered the highest observed damage: part of the filter was discovered and large portions of cover blocks were carried away. Table 2 shows damage estimations according to Hudson, Sawaragi and Van der Meer theories.

In January 1987, the damages were in the same order mentioned previously, 20, 60 and 7 (Table 2). The maximum significant wave height and the average height of the highest one-tenth waves both computed in front of the structure were introduced using Hudson formula. Predicted damage was closer to measured damage using significant wave height.

Interaction and breaking limit curves for the Montalto breakwater have been plotted and compared with wave characteristic estimated in front of the structure. Interaction curve separates on plane (H, T) stability and instability zones (Losada, 1980). Significant wave characteristics, estimated at structure depth, for the March 1988 storm is compared with interaction curve in Figure 8. All significant waves fall inside the stability area. To attain the instability region the average height of one tenth of waves should be used (Figure 8).

In shallow water, this wave parameter seems more suitable for comparison between wave attacks and Losada stability function. For the Sawaragi theory, the maximum value of D% parameter has been considered. For each wave gauge record, significant wave heights at structure depth were computed with the theory of Goda while Qp was estimated directly from Waverider records (Equation 4). No theory is available to transfer Qp parameter at different depths (Mansard, 1987, 1988). However, because of the complexity of the shoaling mechanism, an exact transfer of wave data, especially those referred to wave grouping from deep to shallow water depth, is not possible through the existing theories (Mansard and Funke, 1988; Nelson, 1988).

Since the variability of Qp is not predictable,
Table 1. Wave statistics from Waverider data—30 March through 1 April 1988 (significant wave heights below 2.5 m are disregarded).

<table>
<thead>
<tr>
<th>Date</th>
<th>Hour</th>
<th>H1/10 (cm)</th>
<th>T1/10 (sec)</th>
<th>H1/3 (cm)</th>
<th>T1/3 (sec)</th>
<th>Qp</th>
</tr>
</thead>
<tbody>
<tr>
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<td>18</td>
<td>307</td>
<td>6.8</td>
<td>253</td>
<td>6.9</td>
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<tr>
<td></td>
<td>21</td>
<td>356</td>
<td>8.1</td>
<td>302</td>
<td>7.8</td>
<td>2.9</td>
</tr>
<tr>
<td>31-03-88</td>
<td>0</td>
<td>396</td>
<td>7.5</td>
<td>310</td>
<td>7.5</td>
<td>2.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>537</td>
<td>8.0</td>
<td>415</td>
<td>8.0</td>
<td>2.4</td>
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<td></td>
<td>6</td>
<td>642</td>
<td>8.9</td>
<td>501</td>
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<td>9</td>
<td>508</td>
<td>8.9</td>
<td>405</td>
<td>8.8</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>426</td>
<td>7.6</td>
<td>351</td>
<td>8.0</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>425</td>
<td>7.8</td>
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<tr>
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<td>265</td>
<td>7.0</td>
<td>206</td>
<td>6.6</td>
<td>2.6</td>
</tr>
</tbody>
</table>

the peak value introduced in the Sawaragi formula was estimated at wave rider depth. This could explain the higher differences between measured and predicted damage levels (Table 2). However bathimetry is regular in front of the structure and the bottom slope, lower than 1:80; as a consequence, Qp computed at the Waverider depth is believed to be comparable with that in front of the structure (RYE, 1982). To find out the degree of variability of the Qp parameter with water depth, some wave records at different depths were carefully analyzed. Data are referenced to different Italian sites where two or more wave riders are located at different depths but aligned orthogonally to the coastline. As shown in Figure 9 the range of Qp variability seems to be not significant in the range 13 to 50 meters of depths. Moreover, time-variation of this parameter has the same trend at different depths (Figure 10).

The stability model was applied to determine the damage using the Van der Meer theory. Significant wave heights below 2.5 m were disregarded. After many computer trials and in site measurements to calibrate the program, this value of wave height value was found to be the minimum below which the estimated damage using the Van der Meer method, for a three hours wave attack, is negligible (MURACA, 1989). Figure 11 shows computer simulation of damage progression for the January 1987 storm. Final damage estimation does not change if waves below 2.5 m are neglected. A computer simulation of damage progression for the March 1988 storm is shown in Figure 12. In 1988 the breakwater cover layer was repaired. During the first four months of 1989, the structure suffered only a slight damage. At the end of April 1989, the measured damage according to Van der Meer theory was equal to 2.5.

Table 2. Comparison of measure and predicted damage in selected events.

<table>
<thead>
<tr>
<th>Event</th>
<th>Measured Damage</th>
<th>Predicted Damage</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hudson</td>
<td>V.d.-Sawar.</td>
<td>V.d.-Sawar.</td>
<td></td>
</tr>
<tr>
<td>Jan. 1987</td>
<td>20</td>
<td>6.5-7</td>
<td>15-20</td>
<td>6.8</td>
</tr>
<tr>
<td>Mar. 1988</td>
<td>30</td>
<td>8</td>
<td>100</td>
<td>20-30</td>
</tr>
<tr>
<td>Period</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan.-Apr. 1989</td>
<td>2.5</td>
<td></td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Jul. 1990-1992</td>
<td>4</td>
<td></td>
<td>5.2</td>
<td></td>
</tr>
</tbody>
</table>

Figure 7. Lengths of runs greater than significant wave height (March '88 storm).

Figure 8. Interaction curve for Montalto breakwater. Plotted points indicate wave heights estimated at the front of the structure depth. March 1988 storm.
To test the capability of the computer program in reproducing damage evolution, wave data of January–April 1989 were inserted as input. The test results were completely satisfactory since the final predicted damage was very close to the measured damage (Figure 13). A lengthy duration test was made inserting recorded wave data from June 1990 through July 1992. Also for the two year period, the predicted damage was very close to the measured damage (Figure 14).

A continuous description of damage level evolution, based on Van der Meer’s theory, then seems possible with satisfactory approximation. Nevertheless, some indetermination remains in estimation of wave parameter at shallow water depth, although the site measurements and wave data were carefully analysed.

CONCLUSION

Photographic and topographic surveying have been applied for monitoring the stability of a rubble mound breakwater. Reliable estimation of cover block displacement can be obtained using a non-metric camera and the direct linear transform method of analysis. The use of a metric camera and digital photogrammetric workstation has allowed a high degree of accuracy.

A comparison between in-site measured and predicted damage for a prototype breakwater has been made applying four different theories. A mathematical model for continuous damage description, based on Van der Meer stability analysis, has been calibrated with in site measured data. This model describes superimposition of different sea states and evaluates the damage starting from different structure conditions. Predicted damage with a stability model has been satisfactory also...
for very long simulation periods. A knowledge of the response of the structure for a real or project sequence of sea states can be helpful in programming maintenance or to decide if damage attained will be critical for the stability of the structure.

ACKNOWLEDGEMENTS
The authors would like to thank the E.N.E.L. staff at the Montalto di Castro Power Plant and the technicians of the Montalto Mare Group for their assistance in the collection and elaboration of the data. This study was supported by the Italian Ministry of Scientific Research.

LIST OF SYMBOLS

\(d\) = Van der Meer damage level  
\(D\) = median diameter of stones of armour layer  
\(D^\%\) = Sawaragi damage index  
\(E_{sumj}\) = Energy sum of grouped waves  
\(H\) = wave height  
\(H_s\) = significant wave height  
\(I\) = Iribarren number  
\(I_c\) = transition Iribarren number  
\(I_b\) = Iribarren number for breaking condition  
\(I_z\) = Iribarren number using Rice average wave period and \(H_s\)  
\(l_a\) = average dimension of armour stone  
\(m_n\) = spectral moment of \(n\) order  
\(n\) = number of waves  
\(Q_p\) = Goda spectral peakedness parameter  
\(p\) = Van der Meer permeability parameter  
\(P\) = weight of stable stone  
\(S\) = specific gravity of armour rock \((\gamma/\gamma_s)\)  
\(T_m\) = average wave period \((\sqrt{m_0/m_1})\)  
\(\alpha\) = angle of seaward slope of the structure  
\(\gamma\) = specific weight of stone  
\(\gamma_s\) = specific weight of water  
\(\varphi\) = natural angle of repose of stone  
\(\Delta\) = relative mass density \((S - 1)\)

LITERATURE CITED