

CHAPTER 105

A ROBUST ARMOR DESIGN TO FACE UNCERTAINTIES

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ABSTRACT

A new rubblemound breakwater cross section design named D-armor breakwater is presented. The D-armor cross section shows a significantly larger active armor area than the conventional cross section. The D-armor breakwater shows a similar resistance to the initiation of damage, but a significant increase of resistance to total failure. The observed structural response has a wave height range about 30% wider than the range corresponding to the conventional breakwater; the new section reshapes to an efficient S-shape armor near the total failure point. The D-armor breakwater appears to be a reasonable first step towards a convenient evolution from the conventional breakwater to more efficient designs; the wider structural response makes it appropriate for construction sites with large uncertainties in the estimation of the worst wave conditions in its lifetime.

INTRODUCTION

During the last decades, a continuous effort has been developed towards a better understanding of the structural and hydrodynamic factors affecting the stability of rubble-mound breakwaters. There are two main goals of the research effort: a) New calculation procedures for a more reliable and accurate estimation of the structural response in lifetime to optimize the designs; and b) New designs to reduce the construction cost, maintenance and risk of failure in its lifetime.

The design waves of a variety of maritime projects can only be decided assuming large uncertainties (see Goda, 1988). On the other hand, there are still significant differences in the calculation procedures proposed by different authors to estimate the structural response of conventional breakwaters for given wave

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conditions (see SPM, 1984; Van der Meer, 1988; Bruun et al., 1990; Teisson, 1990; and Medina et al., 1990). Additionally, a number of concrete armor unit designs have been proposed and used in conventional cross sections; however, some of the most costly failures involved the use of special concrete armor units (Sines, San Ciprián, Tripoli, Arzew, Giona Tauro, etc.). Finally, some new breakwater cross sections are being proposed to reduce construction cost or to increase armor resistance; however, some failures have been reported recently (St. Paul berm breakwater) with only a few unconventional breakwaters actually built. This paper focuses the attention on a new breakwater cross section which may be considered a rational alternative design to allow a safe evolution from the conventional section used worldwide to more economically efficient unconventional designs.

A variety of alternative designs to the conventional rubblemound breakwater cross section have been proposed; the S-shape and berm type breakwaters are the most popular unconventional designs. In spite of the limited number of prototypes built according to these new designs, there is an increased number of laboratory results which indicate some of their advantages. However, the current practice for the design and construction of mound breakwaters is conservative; the frequent breakwater failures, and the unknown risks associated with designs that lack experimental verification, may explain the general opposition of designers to adopt radical changes in the classic mound breakwater cross section.

This paper describes a new rubblemound breakwater design: the D-armor breakwater. A comparative tentative analysis of the functional performances of the conventional, the S-shape, the berm, and the D-armor breakwater cross sections is given. The D-armor design appears to incorporate most of the best features of the different alternatives. It has a failure function that covers a wider range of wave heights than the conventional breakwater making the new design a robust solution to face large uncertainties associated with long term wave actions at a construction site.

D-ARMOR: A ROBUST DESIGN

From a structural point of view, the D-Armor breakwater is similar to a conventional design with a significant increase of the armor thickness in the area where the mean water level crosses the external armor profile of uniform slope. Figs. 1-a and 1-b show the cross sections corresponding to the conventional and D-armor breakwaters. Before damage, the external profile is the same; however, when armor erosion increases the D-armor design progressively transforms to an S-shape breakwater (see Fig. 1-c). Because of this characteristic, the structural performance is similar to the conventional breakwater at low levels of armor erosion, but the reshaping process significantly increases the resistance capacity as an S-shape breakwater. Therefore, the D-armor design has the large structural response flexibility required to face the high levels of uncertainty usually associated with the design wave storms. Contrary to the conventional or S-shape breakwaters, the D-armor breakwater may be designed to reshape significantly during its lifetime.

Although the D-armor design is expected to reshape in its lifetime, it is totally different than the berm or reshaping breakwater illustrated in Fig. 1-d. The berm type breakwaters also have failure functions covering a wide range of wave heights and extremely high acceptable armor erosion levels. However, the berm breakwaters have large rock movements along the breakwater which are not present in the D-armor breakwater. The armor elements may be compacted in the reshaping process, but the rock displacements in the D-armor design are very short in comparison with the displacements observed in berm breakwaters.

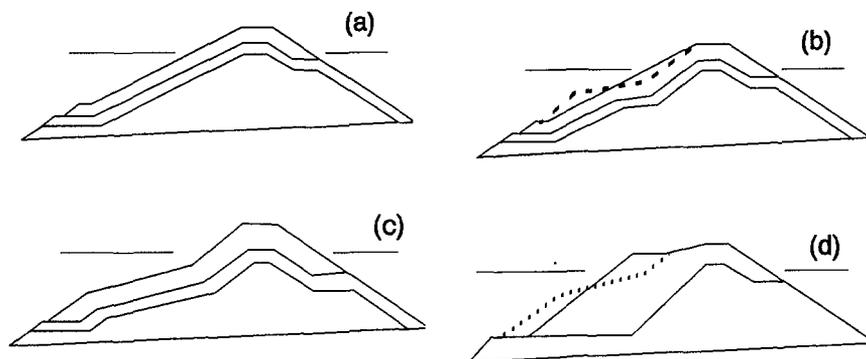


Figure 1.- Rubble-Mound Breakwater Cross Sections: a) Conventional; b) D-Armor; c) S-Shape; and d) Berm Type.

The methodology used by Medina et al.(1990) to study the stability of the armor layer of rubble-mound breakwaters has been applied to experiments in the wave flume at the Universidad Politécnica de Valencia (30x1.2x1.2 m). The wave flume was divided in two parts to check simultaneously a conventional and a D-armor breakwater cross section. The stability of a deep water model with $W_{50}=130g$ was analyzed and a preliminary test result using regular waves is given in Fig. 2, and compared to the data provided by Ergin et al.(1989), by Torum and Naess(1988), and by the SPM(1984). The D-armor breakwater shows a start of damage limit similar to a conventional breakwater. It shows an acceptable damage limit (reshaping) similar to the S-shape breakwater. The D-armor design is more resistant to total destruction than a conventional breakwater, it is more flexible than the S-shape breakwater, and it shows far shorter rock displacements in the reshaping process than the berm type breakwater.

Most breakwaters are built at a construction site where the long term wave climate or the maximum water depth (MSL to sea bed) can only be estimated with large uncertainties. In those cases, low risk and economically-efficient solutions demand robust designs with a flexible structural response having a wide margin

between start of damage and total destruction. To face large uncertainties in wave action, economic optimization leads to very conservative and expensive designs for brittle structural responses, and to less expensive and safer designs for flexible structural responses.

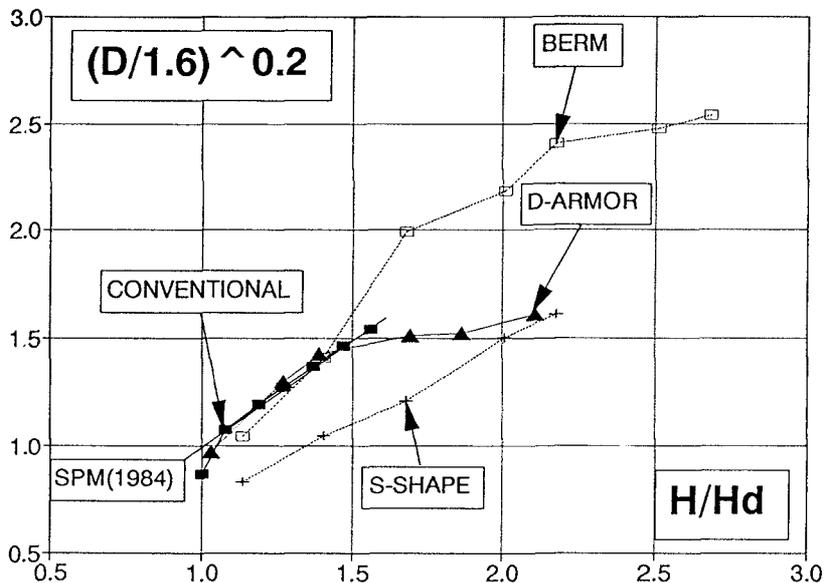


Figure 2.- Normalized Failure Functions Corresponding to Conventional, D-Armor, S-Shape, and Berm Breakwaters.

DESCRIPTIONS OF EXPERIMENTS

In order to analyze the structural performance of the D-armour breakwater, series of 2-D experiments were conducted at the UPV wave flume (30x1.2x1.2 m), divided in two parts to test simultaneously a conventional and a D-armour cross section. A transparent glass divider was used for the verification of the same wave attack on the two cross sections during the experiments. Two capacitance wave gauges were placed in front of the model to analyze the incident and reflected wave train using a modified version of the method of Goda and Suzuki(1976). Fig. 3 shows the longitudinal cross section and plan view of the wave flume used in the experiments. The piston type wave paddle, hydraulically controlled with a servomechanism, was able to move according to the desired time series given by a PC used for the wave generation, recording, and analysis.

Fig. 4 shows the cross section of the conventional and D-armour breakwaters used for the experiments. Fig. 4a describes a typical deep water conventional section similar to that proposed by SPM(1984), with a cap on the top of the structure to

minimize overtopping. On the other hand, Fig. 4b describes a deep water D-armor section in which the armor thickness has been significantly increased in the area where the MWL crosses the armor, while the maximum armor water depth has been reduced from $2H_d$ to $1.5H_d$.

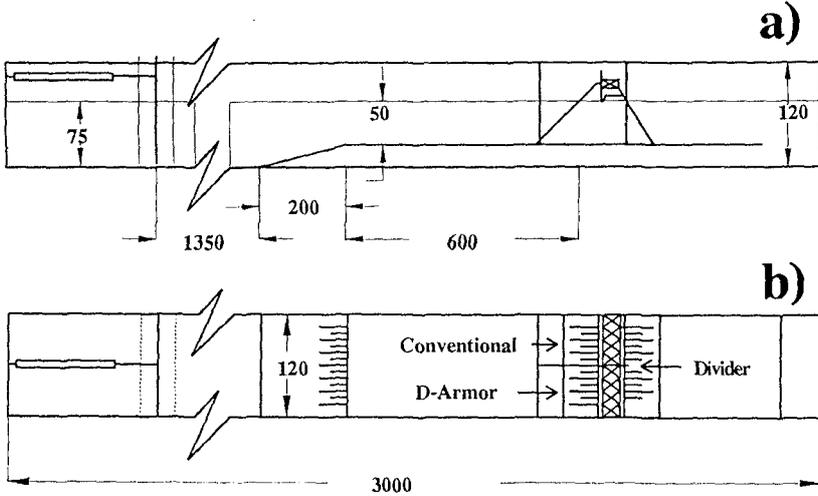


Figure 3.- UPV Wave Flume: a) Longitudinal Cross Section, and b) Plan View.

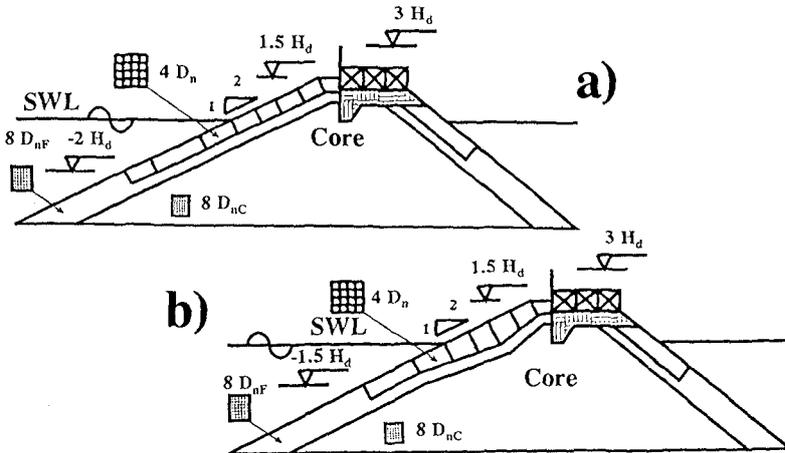


Figure 4.- Breakwater Model Cross Sections: a) Conventional, and b) D-Armor.

The armor was built of angular quarystones with a uniform gradation, a median weight of $W_{50}=130$ g, and a maximum deviation of 25% according to the SPM recommendations. The mean mass density was $\rho_r=2.65$, the slope was $2/1$, and the zero-damage design wave height according to SPM(1994) was $H_d=12$ cm

($K_D=4$). The mean weight of the filter material and core was $W_F=7.5$ g and $W_C=4.3$ g, respectively; the corresponding equivalent cube sizes (Iribarren's terminology) or nominal diameters (Van der Meer's terminology) were $D_n=(W_{50}/\rho_r)^{1/3}=3.66$ cm, $D_{nF}=(W_F/\rho_r)^{1/3}=1.42$ cm, and $D_{nC}=(W_C/\rho_r)^{1/3}=1.18$ cm. The armor thickness was $2D_n$ in the conventional section, while it varied from $2D_n$ to $3.5D_n$ in the D-armor section. The filter thickness was about $3.5D_{nF}$ in both sections. The stability of the cap was not analyzed, but its stability was guaranteed using extra lead ingots to avoid cap displacements. The stones of the armor were painted with different colors and placed in five bands of $3D_n$ width above the SWL, and two bands of width $4.5D_n$ and $7.5D_n$ below SWL. The conventional section had an additional stone band to complete the armor section.

Regular Waves and Random Waves

To evaluate the structural response of the D-armor breakwater, 10 tests with regular waves and 10 tests with random waves were conducted in the UPV wave flume from the no damage level to the total failure point of both the conventional and the D-armor breakwater models. The conventional section reached the total failure point first in all the tests; therefore, it was necessary to protect the destroyed conventional armor to continue the test with the D-armor model, in order to avoid a total collapse of the overall conventional structure. Once both armors were destroyed, all the armor stones were removed and classified by colors, to rebuild the profiles of both filter layers to put the armor units in their corresponding place for a new test.

The tests with regular waves were planned to be free of paddle reflected waves. The Iribarren's number ($Ir=[\tan \beta]/[2\pi H/gT^2]^{0.5}$) was kept constant for all the runs of each test; different values of Ir were used for each test in the range $1.7 < Ir < 4.2$. Starting from the zero-damage design wave height, $H_d=12$ cm, the wave height was increased 10% each run ($H=H_d [1.1]^k$; $k=0,1,2,\dots$) until total failure of the armor layer. Only a few waves were generated each run to avoid reflections on the paddle; therefore, the run of each energy level was repeated many times until an equilibrium profile was obtained in both breakwater models.

The tests with random waves were planned for not being free of paddle reflected waves. Seven minutes of random wave generation of JONSWAP spectra ($\gamma=1$ and $\gamma=10$) using the DSA-FFT method produced between 200 and 300 waves depending on the Ir value of the run. An Iribarren's number for random waves defined as $Ir=[\tan \beta]/[2\pi H_{m0}/gT_{02}^2]^{0.5}$ was constant for all the runs of each test; different values of Ir were used for each test in the range $2.2 < Ir < 3.5$. Starting from the zero-damage design wave height, $H_{10}=H_d=12$ cm, the wave height was increased 10% each run ($H_{10}=H_d [1.1]^k$; $k=0,1,2,\dots$) until total failure of the armor layer.

The measured characteristics of the incident wave trains do not exactly fit the desired waves; therefore, the results shown in this paper refer to the wave

characteristics actually measured and not the theoretical characteristics indicated as generated waves. According to the planned experiments, each test should keep constant the Iribarren's number (Ir); however, the measured value of Ir for each run in each test showed small variations about the mean value, most of them in the interval $\pm 1\%$. The mean value of the measured Ir , of all the runs of each test, was taken as the actual measured Ir of the test.

EXPERIMENTAL OBSERVATIONS

For a first evaluation of the D-armor versus the conventional breakwater, 10 tests with regular waves were carried out with $1.7 < Ir = [\tan \beta] / [2\pi H / gT^2]^{0.5} < 4.2$. Fig 5 shows the stability numbers ($N_s = H / \Delta D_p$) corresponding to the start of damage and total failure point for both sections. SPM(1984) indicates a stability number for start of damage of $N_s = 2$ ($K_D = 4$), and a stability number for maximum damage (40% to 50%) of $N_s = 3.12$; the N_s values suggested by SPM(1984) are near to the minima of the stability curves represented in Fig. 5, in agreement with the fact that the design method proposed by the SPM(1984) does not take into consideration the design wave period. The D-armor breakwater shows a start of damage curve similar to the conventional breakwater, but the total failure curve is qualitatively different, showing in all the cases analyzed a significantly higher resistance to total failure.

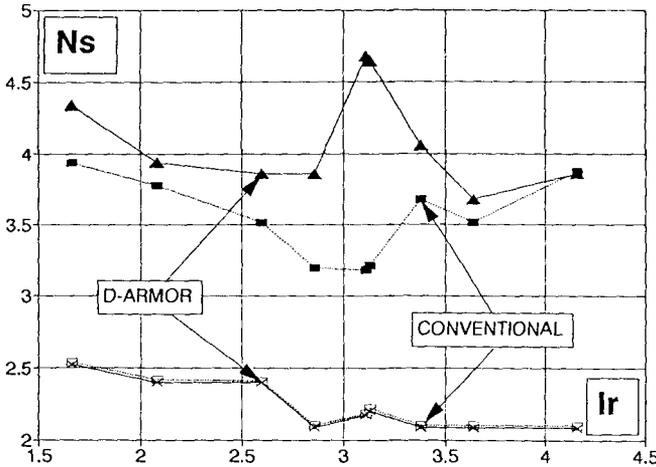


Figure 5.- Stability Numbers for start of Damage and Total Failure Corresponding to the Conventional and D-Armor Breakwaters.

According to the structural performance characteristics represented in Fig. 5, the D-armor breakwater shows a minimum stability number for total failure 15% higher than that corresponding to the conventional breakwater. However, the sea wind waves are not regular waves but irregular waves; a more realistic analysis of the structural response may be achieved using random waves.

Criteria for the Measurement of Armor Damage

There are significant discrepancies in the literature about the quantitative definitions of the start of damage, the partial damage, and the total failure criteria. Therefore, it is convenient here to define first the concept of armor damage used in this paper.

Iribarren(1965) used the equivalent cube size, $(W/\rho_r)^{1/3}$, to normalize armor damages. Iribarren defined the total failure point as the erosion of the armor section affecting 100% of (his definition) the active zone, which had an area of $9 D_n^2$. Van der Meer(1988) also used the same concept re-named as nominal diameter, $D_n=(W/\rho_r)^{1/3}$, to normalize the measurements of the erosion of the armor layer. The erosion corresponding to the failure criterion given by Van der Meer(1988) was $8 D_n^2$ (filter layer visible, slope: $\cotan(\beta)=2/1$). On the other hand, SPM(1984) defined the damage as a percent of the armor units displaced from its breakwater active zone. However, while Iribarren(1965) defined his active zone as $9(W/\rho_r)^{2/3}$ (two layer armor), SPM(1984) defined its active zone as that one which "*extends from the middle of the breakwater crest down the seaward face to a depth equivalent to one zero-damage wave height below the still water level*". Therefore, Medina et al.(1992) found that the total failure criterion is equivalent to an erosion of the armor layer of $8 D_n^2$, $9 D_n^2$, or $14 D_n^2$, depending of who is the author in the above referred publications. On the other hand, Medina et al.(1990) presented a large scale experiment with partial armor damages higher than $20 D_n^2$. It is evident that the total failure point is quite subjective in the literature. In this paper, the total failure point is defined as the level of armor erosion which suddenly shows a sharp decrease of armor resistance with significant displacement of stones from the filter layer.

The start of damage point is also difficult to define. Van der Meer(1988) considered the start of damage point as an erosion of $2 D_n^2$; however, SPM(1984) indicates an erosion of 0 to 5% of the active armor zone (0 to $1.6 D_n^2$) as a no-damage condition. In this paper, the start of damage is defined as the point with a minimum, but detectable erosion of the armor. After 20 subjective evaluations of start of damage points, it was found that $1.0 D_n^2$ is a reasonable estimation of the minimum detectable damage: the start of damage point. This quantitative definition of the start of damage point is in agreement with the no-damage condition in SPM(1984) because $1.0 D_n^2 \approx 3\%$ of the active armor zone.

Between the start of damage and the total failure points, the armor damage was calculated as the eroded area of the armor profiles corrected for errors and settlement. Single profiles centered in both the conventional and the D-armor models were obtained using rods separated $1.25 D_n$ with articulated circular feet having a diameter of $0.75 D_n$. A flexible aluminum chain mat was placed on the eroded armor to regularize the penetration of the rods into the armor. The distances in the profiles were normalized by $D_n=(W/\rho_r)^{1/3}$. The origin of coordinates was located at the waterline where the SWL crossed the original armor profile. The normalized dimensionless profiles obtained in this experiment were similar to those shown by

Medina et al.(1990) from the large scale experiment conducted in the O.S.U. Wave Research Facility. For the lower damage levels, damages were estimated counting colored stones moving among bands of different color (assuming 38% porosity), and high levels of damage were estimated from the rod profiles.

Experimental Results

In this experiment, and in the large scale experiment described by Medina et al.(1990), the erosion profiles show an almost constant neutral point between the erosional and the accretional armor areas. The water depth of that neutral point, h_n , was observed to be about $h_n = H_d$ independently of the level of armor damage. Fig. 6 shows the maximum depth of the neutral points, h_n , for both D-armor and conventional breakwaters. The depth of the neutral points appears to be independent of Ir with an 80% confidence band covering the range $0.8H_d$ to $1.5H_d$. The D-armor breakwater was built with filter stones below $h = 1.5H_d$, and no significant movement of those small stones were observed during the tests; therefore, there seems to be no activity of the armor stones below $h = 1.5H_d$.

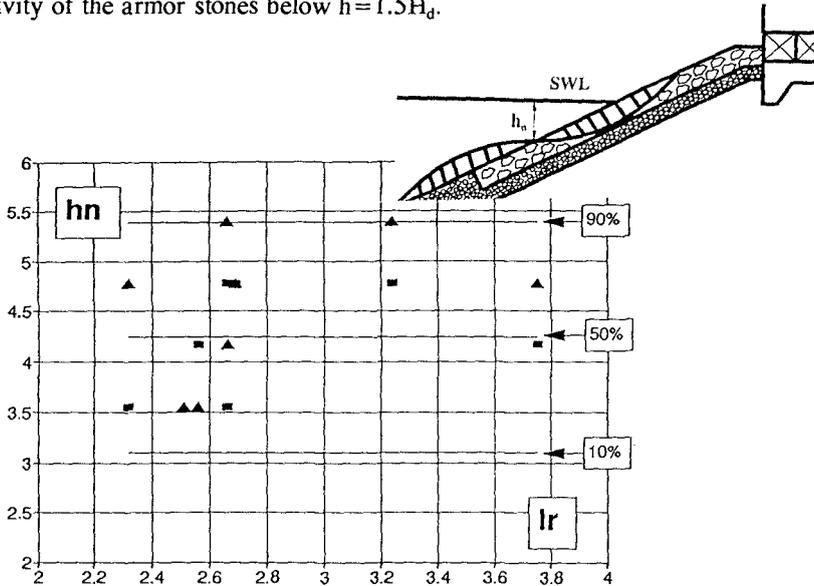


Figure 6.- Maximum Depth of the Neutral Point.

Fig. 7 shows the maximum damage measured, D_{max} , in both the conventional and D-armor breakwaters. The maximum damage seems to be independent of Ir ; the D-armor breakwater showed D_{max} larger than the conventional breakwater, and the difference roughly corresponds to the increase of the D-armor thickness (area: $11.8D_n^2$). The extra volume of armor stones used to increase the armor thickness of the D-armor breakwater appears to be fully active during the armor erosion process

before the total failure of the structure, contrary to the armor stones placed below $h=1.5H_d$ (area: $7.3D_n^2$) in the conventional section, which appear to be inactive during the erosion process. From both Figs. 6 and 7, the inefficiency of the conventional breakwater is apparent, as is the possibility of improving the breakwater cross sections by concentrating the volume of armor stones in the active zones where the wave action is more intense.

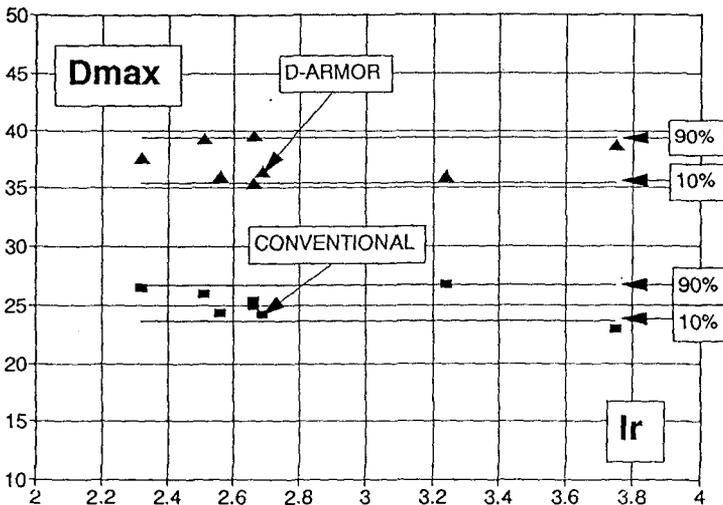


Figure 7.- Maximum Armor Damages Measured During the Experiment.

Although it is evident that the D-armor breakwater has as many as 50% more active armor stones than the conventional breakwater, it is convenient to present the measured failure functions with random waves, in order to give an appropriate description of both structural responses. The first problem to be solved concerns the definition of variables in the representation of failure functions of breakwaters under random waves. A simple representation of damage versus $H_{m0}=4[m_0]^{0.5}$ is not appropriate because it is well known that the damage depends also on the number of waves in the run, the value of I_r , etc. Therefore, it is necessary first to normalize the variables used in the representation of failure functions.

In this paper, the damage D is a dimensionless variable of the armor erosion because the magnitudes of lengths and distances have been normalized by the nominal diameter D_n . Taking into consideration the fifth power relationship between damage and wave height shown by the failure functions given by SPM(1984), Van der Meer(1988), and Medina et al. (1990) for conventional breakwaters, the ordinates in the failure functions of this paper have been transformed using the expression $[D/1.6]^{0.2}$. According to Medina et al.(1992), the failure function suggested by SPM(1984) for rough quarrystones fits the line

$$\frac{H_{10}}{H_d} = \left[\frac{D}{1.6} \right]^{\frac{1}{5}} \quad (1)$$

The runs of random waves had different number of waves, N_w , Iribarren number, Ir , and groupiness parameter, α . In order to normalize the characteristic wave heights (abcisa), the formulas of Van der Meer(1988) and the preliminary conclusions given by Medina et al.(1990) were used. According to Van der Meer(1988), the damage D is proportional to H_{m0}^5 , $[N_w]^{0.5}$, and $[Ir]^{2.5}$ if $Ir \leq 3.5$. The minimum armor stability using the Van der Meer's formulae is obtained for $Ir=3.5$; if $N_w=1000$ and $Ir=3.5$, the estimation of damage provided by this formulae is fairly coincident with the estimation given by the SPM(1984). On the other hand, according to Medina et al.(1990), the damage D is proportional to H_{m0}^5 , and $[\alpha]^{0.5}$, in which α is the envelope exceedance coefficient used by Medina et al.(1990) to characterize the wave groupiness of irregular wave trains attacking rubblemound breakwaters. Therefore, the standard characteristics of the irregular wave train to represent failure functions were: $N_w=1000$, $Ir=3.5$, and $\alpha=1$.

In the conditions for normalization given above, the failure functions of the conventional breakwater fairly showed the expected fifth power relationship between dimensionless damage and wave height for the complete wave height range, but the observed mean stability numbers for all the damage levels were about 7% lower than the stability number predicted by the Van der Meer's formula. On the other hand, the D-armor breakwater showed two radically different parts in the failure functions: a)A fairly fifth power relationship between damages and wave heights for low and moderate damages up to $50\% D_{max}$, and b)A higher resistant upper tail of the failure function. The stability numbers of the start of damage point, and the low levels of damage, appear to be 5% to 10% smaller than the conventional breakwater; however, the stability number corresponding to the total failure point is 10% to 15% higher than the conventional breakwater.

Analyzing the results obtained from the tests using regular and random waves, the global structural response patterns are clear. In all the cases tested, the D-armor was significantly more resistant to total failure than the conventional breakwater. The difference in resistance to total failure depends on Ir for regular waves, and is relatively constant for random waves. The stability number of the D-armor for total failure and random waves is 10% to 15% higher than the conventional breakwater. In most cases tested, the D-armor was less resistant to the start of damage point and to low levels of damage. The stability number of the D-armor for the start of damage point and low damage levels appears to be 5% to 10% lower than the conventional breakwater.

The above described behavior of the D-armor breakwater suggests an alternative definition of armor damage, appropriate for these kind of structures with as much as 50% more armor erosion capability before total failure. One could

consider only "initiation of damage" the identifiable damage levels below the extra active armor area of the D-armor section ($11.8 D_n^2$ in this experiment), which approximately corresponds to the difference between the maximum damage of the D-armor and the conventional breakwaters. The new definition of the armor damage applicable to D-armor breakwaters in this experiment is $D^* = D - 11.8$. One could also reduce the stability numbers corresponding to the D-armor breakwater by a factor of 1.15 to fit the total failure point of both breakwaters, which is equivalent to a reduction of the weight of the armor stones by a factor of 2/3. With this new definition of armor erosion to equalize the maximum armor damage, and with the reduction of median armor mass to equalize the total failure point, the new failure functions are those represented in Fig. 8.

Using lighter armor stones (factor 2/3), and the definition of damage which equalizes the maximum armor erosion, the D-armor breakwater shows fairly the same failure function as the conventional breakwater. This D-armor breakwater would show identifiable minor damages ($D^* = [D - 11.8] < 0$) for values of H_{10} in the range: $0.65H_d < H_{10} < 1.15H_d$; and the same structural response until total failure as the conventional breakwater with heavier stones. Therefore, a prototype based on the D-armor concept is expected to be significantly cheaper than the conventional breakwater with the additional advantage of having a wider wave height range of acceptable minor damages. These properties make the D-armor breakwater a cost-efficient design especially indicated for construction sites with large uncertainties on the design wave condition.

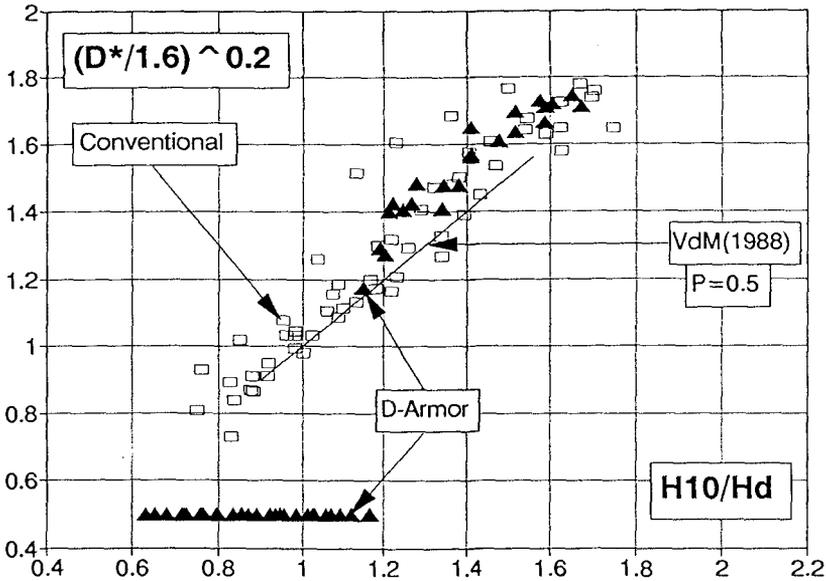


Figure 8.- Equalized Failure Functions Corresponding to the D-Armor and the Conventional Breakwaters.

All the cases were tested with approximately constant mean water level; therefore, appropriate designs for construction sites with large tide ranges may require a modification of the D-armor breakwater shown in Fig. 4b. On the other hand, the results are consistent with the intuitive concept on which the D-armor design is based: it is structural and cost efficient to concentrate the armor volume in the area where maximum erosion is expected. Therefore, the first tentative D-armor design shown in Fig. 4b should be adapted to the specific tidal range and overtopping conditions to get full advantage of the new concept. As a matter of fact, some preliminary tests not shown in this paper indicate that a D-armor breakwater may be extremely cost-efficient with respect to conventional breakwaters, if significant overtopping is acceptable for the design conditions.

CONCLUSIONS

A D-armor mound breakwater cross section is presented and conceptually compared with conventional, S-shape, and berm breakwaters. A systematic comparison between a conventional and the corresponding D-armor design is given. The UPV wave flume (30x1.2x1.2 m) was divided in two parts to test simultaneously both sections with exactly the same wave attack. The results were obtained from 10 tests with regular waves and 10 tests with random waves, all of them developed from the no damage level to the total failure of the armor layer. The observed damages are consistent with the interpretation that the increase of the armor volume near the SWL fairly corresponds to the increase of the active volume of the armor layer. Considering this extra volume of armor stones as an acceptable erosionable part of the D-armor (minor, but identifiable damages), the D-armor structural performance is similar to the conventional breakwater with two clear advantages: 1st) A 15% higher stability number for moderate and high levels of armor erosion, and 2nd) A 50% wider wave height range of acceptable damages about the SPM design wave condition. The first advantage allows the design of a cheaper armor layer in prototypes using lighter stones and achieving the same stability; the second advantage makes the design appropriate to face large uncertainties in the design wave action in its lifetime.

The global view of the results noted above points out the inefficient design of the conventional breakwater. The deep water cross section proposed by SPM(1984) indicates the use of armor stones in inactive areas ($h > H_d$), while critical armor areas near the MWL are only protected by a $2D_n$ thick armor layer. New structural and cost-efficient breakwater cross sections are necessary to surpass the old conventional design. However, any unconventional design implies new and unknown risks due to new problems like control of construction versus design, new and unexpected modes of failure, etc. The uncertainty to new and unknown risks due to any new design, and the reasonable aversion to risk of most designers, constructors, and decision makers, breaks down the application in practice of new laboratory tested efficient designs. Therefore, the migration from the conventional design to new cost-efficient designs will be an step by step process, in which each step does not imply an excessive jump on the generally accepted designs. The D-

armor breakwater is a new concept which has shown in the laboratory to have significant economic advantages over the conventional design, and does not appear to be an excessive conceptual and constructive jump from the conventional breakwater.

As noted above, the D-armor breakwater may be a reasonable first step for a safe migration from the inefficient conventional breakwater to new structural and cost-efficient designs. It shows less inactive armor area, and 50% more active area with a wide acceptable wave height range of minor damages (initiation of damage) and 15% higher stability number for high damage levels up to total failure. It appears to have the same or low construction cost with higher stability. On the other hand, it has a reasonably low risk to unknown aspects. The D-armor design shows a structural response with extremely high flexibility, making it appropriate to face high uncertainties in the design wave conditions at the construction site; therefore, it seems to be a reasonable economically-efficient alternative to the conventional design in both deep and shallow waters.

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