CHAPTER 139

WAVE CLIMATE SIMULATION AND BREAKWATER STABILITY

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Abstract

The nonstationarity of the sea waves is usually significant for periods longer than 3 hours (1,000 waves) with a correlation coefficent $r_{HH}(3)$ below 95% on most coasts of the world. Therefore, the actual engineering problem is to define an appropriate armor damage model applicable for nonstationary processes and not only for stationary sea states with a given storm duration. In deep water, five conditions are identified for any rational armor damage model to properly take into consideration the storm duration. A new exponential model applied on individual waves is shown to acomplish the five necessary conditions identified, and is compared with the Teisson's and Vidal's models based on totally different assumptions, but acomplishing most of the conditions. The three models provide similar results for a wide range of values if the parameters are n63%=30 and N_W=50 for the exponential and Vidal's models respectively. An example of the application of the exponential model to nostationary processes is given, applying the new method to the case of the partial failure of the Zierbana breakwater (Port of Bilbao, Spain) under construction in February 1996.

Introduction

The duration of the storm and the wave groupiness are design factors not explicitly addressed by the Hudson formula. The storm duration obviously increases the expected damage on the armor layer because it increases both the number and the size of the individual waves above the corresponding damage threshold level. On the other hand, some characteristics of wave groups are related to the variability of variance and storm duration. A number of models and stability formulae considering storm duration have been

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developed during the last decade, introducing a variety of empirical relations based on different interpretations of test results up to 16,000 waves. The relationships between wave groupiness and extreme waves was considered by Vidal et al.(1995) to introduce a new discrete wave parameter to characterize design storms, instead of wave spectrum and storm duration. However, the problem of defining an appropriate and simple method to take into account storm duration and stochastic variability, is not solved.

The stationarity of the process is a basic assumption of most methods for analyzing breakwater stability; however, although the random sea waves were stationary, the significant wave height shows a natural variability that increases when spectral peakedness increases and record length decreases. Therefore, a larger storm duration increases the expected armor damage. Moreover, it is obvious that sea waves are never stationary and stationarity only should be considered a reasonable approximation for short periods of time. When long storm durations are considered for designing (N>1,000 waves), the influence of nonstationarity on sea waves should be taken into consideration.

This paper compares first some of the most common methods and formulas proposed to analyze the influence of storm duration on the estimation of the armor damage. Secondly, the influence of the intrinsic variability of variance of the stationary processes related to wave groups on the expected armor damage is evaluated. Thirdly, the wave climate simulators and the need of multivariable stochastic models to describe the wave climate are justified. Fourthly, the necessary conditions are given to build up rational armor damage models for nonstationary processes. Fifthly, a new exponential model on individual waves are described and compared to the Teisson's and Vidal's models. Finally, an example of application is given to the case of the partial damages occurred at the Zierbana breakwater (Port of Bilbao, Spain) under construction during February 1996.

Armor Damage as Function of Storm Duration

Using SPM(1984) as the basic methodology for designing rubblemound breakwaters, the expected armor damage would not be dependent of the storm duration. However, it is obvious that given a design storm, the armor damage increases if the storm duration increases. There are significant discrepancies among different authors to quantify the relationship D versus N. Taking into consideration a reference storm duration of 1,000 waves, Fig. 1 shows the relative damage estimated by the models proposed by SPM(1984), Van der Meer(1988), Medina and McDougal(1990), Teisson(1990), Smith et al.(1992), and Vidal et al.(1995). Vidal's model has been applied considering a Rayleigh wave height distribution; therefore, the model may be considered an upper limit of the armor damage as function of the number of waves of a given sea state. The models compared in Fig. 1 have different variables and assumptions; the relative armor damages, D(N)/D(1000), have been calculated referring the damage estimated by each model to the damage estimation for 1,000 waves in deep water conditions.

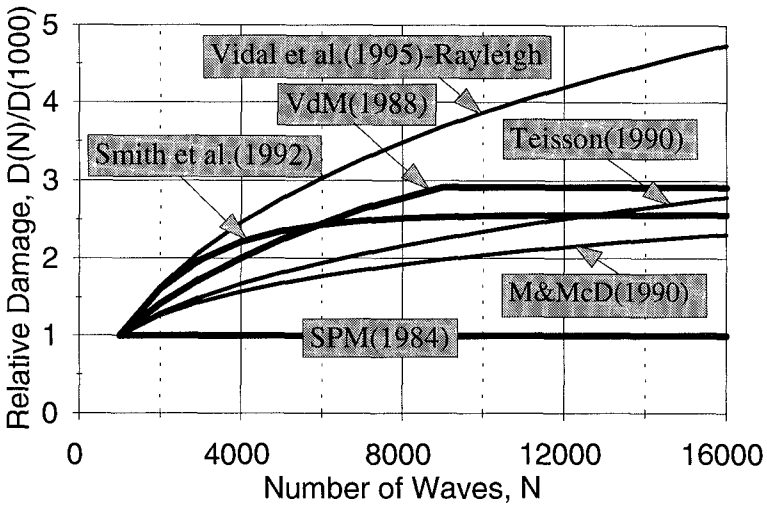

Figure 1.- Armor Damage as Function of Storm Duration.

Variability of Variance

The stationarity of the process is a basic assumption of most methods for analyzing breakwater stability; however, although the random sea waves were stationary, the significant wave height shows a natural variability that increases when spectral peakedness increases and record length decreases. Therefore, the storm duration affects not only the expected armor damage but also the variability. According to Medina and Hudspeth(1987), the coeficient of variation of H_{m0} is given by

$$CV(H_{m0}) \approx \sqrt{\frac{Q_e}{8\,N}} \qquad (1)$$

in which Q_e is the spectral peakedness parameter given by

$$Q_e = \frac{2m_1}{m_0^3} \int_{f_{min}}^{f_{max}} S_\eta^2(f)\, df \qquad (2)$$

in which $S_\eta(f)$ is the variance spectrum, and m_0 and m_1 are the zeroeth and first spectral moments respectively. Assuming a perfect stationarity of the process, the variable H_{10} of finite records of 1,000 waves, $\{H_{10}\}_{1000}$, is a Gaussian random variable; therefore, the maximum $\{H_{10}\}_{1000}$ increases when the storm duration increases so do the corresponding armor damage (see Medina et al.,

1994) according to

$$\frac{H_{10}}{H_d} = \left[\frac{D}{1.6}\right]^{\frac{1}{6}} \qquad (3)$$

in which $H_{10}=1.27\ H_{m0} > H_d$, H_d is the design wave height, and D is the armor damage. Eq. 3 fits the failure data given by the SPM(1984) and may be considered a reasonable good approximation for estimating damages of tests with wave runs around 1,000 waves. For very long wave runs, it may be reasonable to select the highest $\{H_{10}\}_{1000}$ to estimate the damage according to Eq. 3.

Fig. 2 shows the relative damage corresponding to the highest $\{H_{10}\}_{1000}$ when the storm duration increases up to N=16,000 waves; the values of the spectral peakedness parameter Q_e= 2.3, 4.4, and 6.2 correspond to the JONSWAP spectral shapes γ=1, 5, and 10 and are related to different wave grouping characteristics (see Medina and Hudspeth, 1990). The variability of variance increases significantly the expected damage, but far below that observed in Fig. 1.

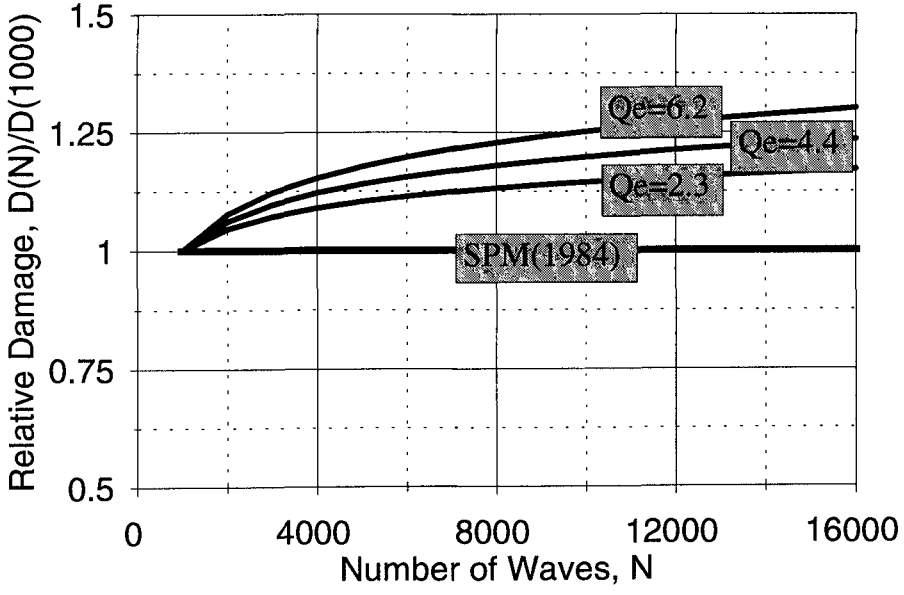

Figure 2.- Effect of Variability of Variance on Expected Armor Damage.

WAVE CLIMATE SIMULATION

The quantitative estimation of storm-induced beach change, as function of the time-dependent forcing conditions, has proved to be a powerful technique that may be applied in project designing of coastal structures. The basic input of the numerical models for simulating beach profile changes are the evolution in time of the wave climate variables. By analogy, it is reasonable to consider that reliable models for quantitative estimation of armor damage as function of the time-dependent forcing conditions will be developed in the near future. In this scenario, new wave climate models will be necessary to provide the appropriate input variables as do the wave climate simulators.

Fig. 3 shows a typical wave climate simulation corresponding to the first hours of the year on the coast of Oregon, according to Medina et al.(1991). The nonstationarity is significant for periods as short as 3 hours (N<1,000 waves). The correlation coefficient of Oregon, $r_{HH}(3) = [r_{HH}(6)]^{1/2} = +0.92$, is similar to the $[r_{HH}(6)]^{1/2}$ given by Rossouw(1988) for the South African coast, slightly lower than the $r_{HH}(3) = +0.94$ given by Bettencourt et al.(1995) for the coast of Portugal, but higher than the $r_{HH}(3) = +0.89$ calculated for the coast of Valencia with shorter fetchs in the Western Mediterranean. Therefore, the actual problem is not to estimate the armor damage corresponding to a given design storm, $D[H_{m0}, T_{01}, N]$, but to estimate the evolution in time of damage for nonstationary processes, $D[\{H_{m0}, T_{01}\}(t)]$.

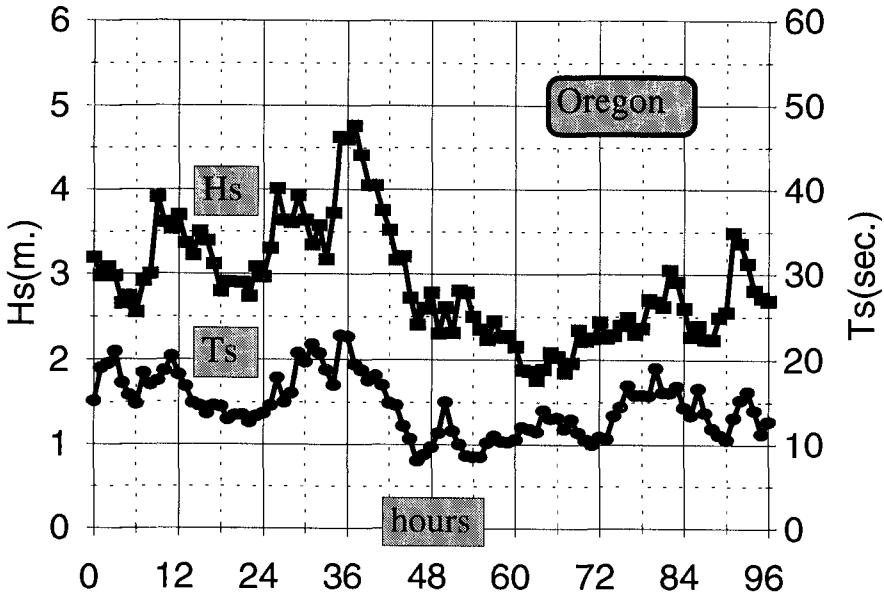

Figure 3.- Hourly Simulation of {Hs,Ts} Corresponding to January (Oregon)

Armor Damage Models for Nonstationary Processes

The armor damage models compared in Fig. 1 are based on different assumptions, some of them unappropriate for nonstationary processes. The design method proposed by SPM(1984) is based on a wide range of laboratory tests only with regular waves; the extrapolation to random waves has been changing in time with $H_{D=0}=H_s$ proposed by SPM(1975), and $H_{D=0}=H_{10}$ proposed ten years later by SPM(1984). In deep water conditions, the storm duration obviously affects the armor damage, because changes both the expected largest wave and the number of waves above any threshold level; therefore, armor damage must increase when storm duration increases, and no fixed equivalence independent of duration exists between regular waves and random waves.

The models proposed by Van der Meer(1988) and Smith et al.(1992) are based on laboratory tests only with random waves; both have an equilibrium limit for the armor damage under random wave attack in stationary conditions. However, armor damage must increase when storm duration increases for the reasons given above; therefore, no equilibrium limit is acceptable for random waves in deep water conditions. The empirical model of Medina and McDougal(1990) based on the Van der Meer's results avoid this problem but is difficult to be applied to nostationary processes.

The method proposed by Teisson(1990), based on the test results with random waves given by Lepetit and Feuillet(1979) is directly aplicable to nonstationary processes. The design wave height, H_D, producing similar damages with regular waves is

$$H_D = 1.18\ H_{1/3}\ t^{0.095} \qquad (4)$$

in which H_D and $H_{1/3}$ are given in meters and t in hours. For rubblemound breakwaters and nonstationary process the damage is calculated according to:

$$D(t) = 0.706 \left(\sum \left[H_{1/3}(\Delta t)\right]^{10.54}\ \Delta t\right)^{0.37} \quad ;\ t = \sum \Delta t \qquad (5)$$

in which Δt is small enough to admit the nonstationary process correctly composed by a succesion of short stationary sea states characterized by $H_{1/3}$.

Vidal et al.(1995) proposed the use of the average height of the $N_W=100$ largest waves in a sea state, as the characteristic wave height which take into consideration both the intensity and duration of the sea state. Although these authors indicated that the most suitable value of N_W is still a subject of continuing research, their method may easily be applied to nonstationary processes. The armor damage would then be related to the average of the highest N_W waves in the storm, whatever was the evolution of the significant wave height in time during the storm.

Any rational model to estimate the evolution in time of the armor damage of rubblemound breakwaters in deep water conditions should fit reasonably well the existing test results with regular and random waves. Therefore, the following necessary conditions for acceptability of an armor damage model applicable to nonstationary processes are:
(1) Under regular wave attack, the maximum damage must be limited by the existence of an equilibrium profile, $D(H,T,N)<D_0(H,T)$.
(2) The maximim damage under regular waves may be approximated (if wave periods are not considered) by Eq. 3.
(3) There is no equilibrium profile under random wave attack in deep waters; the damage must grow with the storm duration without any limit.
(4) The armor damage should be insensitive to the waves below some threshold level related to the design wave height, $H_i<H_d$; only the large waves are relevant to the armor damage.
(5) The characteristics of a few highest waves should have a significant effect on the armor damage.
(6) The method must be aplicable to nonstationary processes where the characteristics of the sea states are continuously changing in time.

Only Teisson's model, based on hypothesis of equivalence for damages, and Vidal's model, based on a parameter $N_W=100$ not well justified, may be applied to nonstationary processes and accomplish most of the necessary conditions given above. Laboratory test are rarely conducted considering nonstationarity; furthermore, long runs in wave flumes have a low reliability because of the difficulty in controlling the stochastic structure of random waves for a long time. As a result, the overall quality of data suporting the models compared in Fig. 1 has to be considered low, because observations are partially contaminated by assumptions not compatible with the necessary conditions given above.

Exponential Model on Individual Waves

A simple method that accomplish the six conditions given above is the exponential model applied on individual waves. The exponential model is normally used for describing events occurring at random time with a constant future lifetime, and in life-testing applications. The evolution of damage of the armor layer of a rubblemound breakwater in deep waters shows some of the characteristics suitable for the exponential model.

The exponential model for regular waves may be described by

$$D(H,T,N) = D_0(H,T)\,[1 - \exp(-k\,N)] \quad ; \quad k = \frac{1}{n63\%} \qquad (6)$$

in which n63% is the number of waves that causes 63% of damage in a regular wave attack. Fig. 4 shows the evolution of damage in time, under regular waves, corresponding to the exponential model.

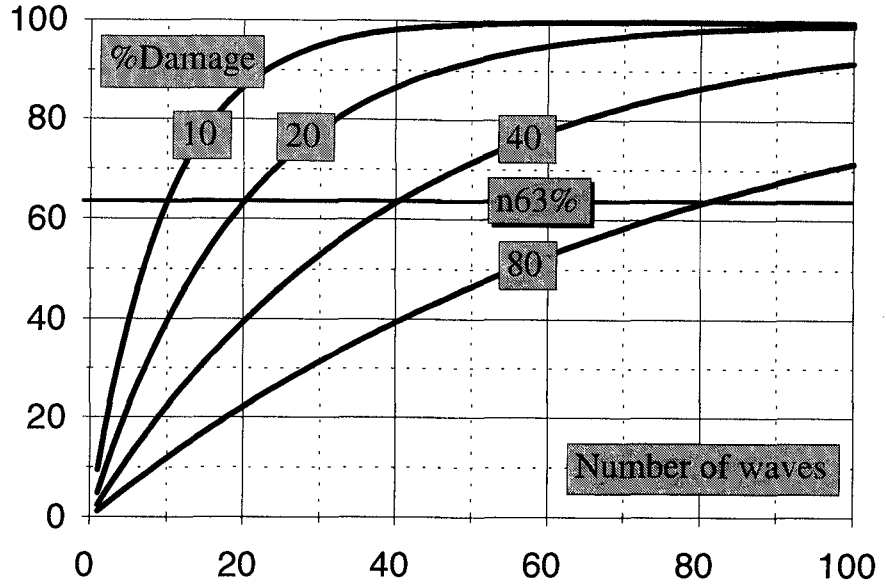

Figure 4.- Parameter n63% Controlling the Evolution of Damage in Time.

The exponential model for random waves requires first the identification of the individual waves, $\{H_i,T_i\}$, in a continuous record. In this paper, the orbital criterion proposed by Giménez et al.(1994) has been selected. The model could be extended to both wave heights and periods; a simplified version considering only wave heights may be descibed by

$$D_I = D_{I-1} + \left(\frac{1}{n63\%}\right)\left[D_0(H_I) - D_{I-1}\right] \quad \text{if} \quad D_0(H_I) > D_{I-1} \qquad (7a)$$

$$D_I = D_{I-1} \quad \text{if} \quad D_0(H_I) < D_{I-1} \quad \text{or} \quad H_I < H_{min} \qquad (7b)$$

in which $D_0(H_i)=1.6\ (H_i/H_d)^5$. Eqs.7 are the discrete derivative version of Eq. 6, if wave periods are not included in the armor damage model. The different armor damage models were compared on the basis of numerical simulations and the orbital criterion shown by Giménez et al.(1994). Fig. 5 shows the evolution of damage corresponding to the exponential model with n63% ranging from 10 to 40, compared to the estimations given by the models of Teisson(1990) and Smith et al.(1992). Teisson's model gives similar results to the exponential model with n63%=30; on the contrary, Smith's model shows radically different results. On the other hand, Fig. 6 compares the exponential model and the model of Vidal et al.(1995).

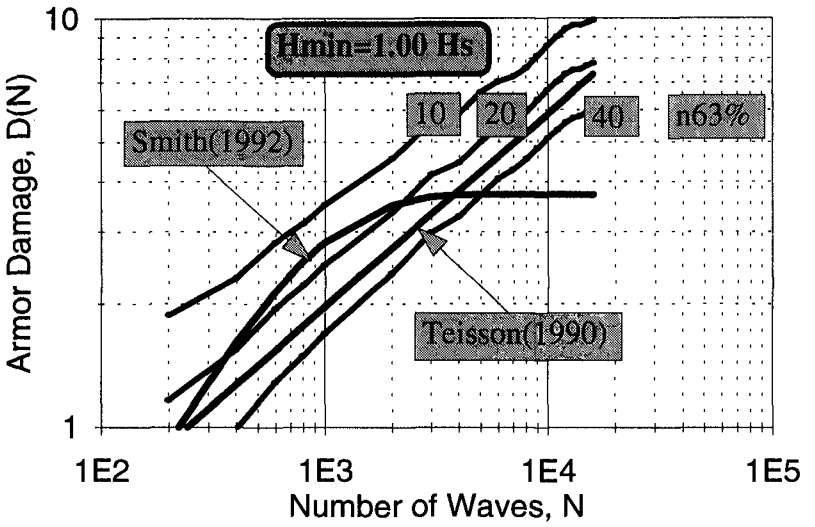

Figure 5.- Damage Estimations Compared to Teisson's and Smith's Models.

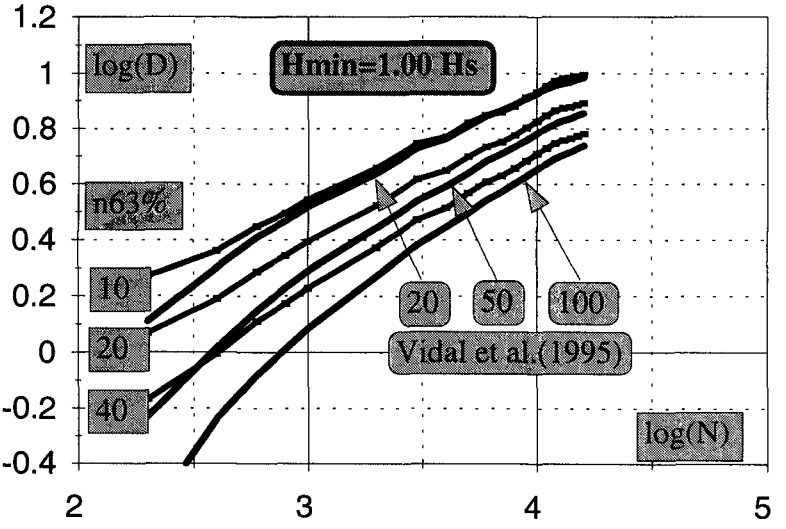

Figure 6.- Damage Estimations Compared to Vidal's Model.

The results provided by the exponential and Vidal's model for N>5,000 waves suggest that only the largest waves in a record are relevant to estimate armor damages. Fig.7 shows the sensitivity of the armor damage estimated by the exponential model to the threshold level H_{min} of Eq. 7b. The significance of

damages caused by the lowest waves ($H_i<H_{min}$) in a storm decreases when the storm duration increases. Considering the exponential model with n63%=30, 95% of damage are produced by less than 1% of the highest waves in a random wave train of 10,000 waves.

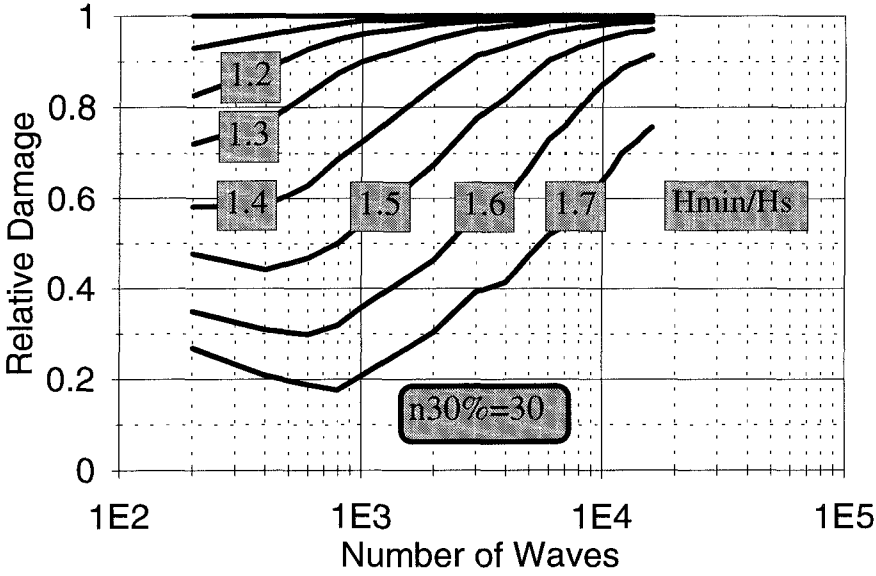

Figure 7.- Sensitivity of the Exponential Model to H_{min}/Hs.

Example: Partial Damages on the Zierbana Breakwater in the Port of Bilbao (Spain) under Construction During February 1996.

The development of the Port of Bilbao in the outer Abra is described by Losada et al.(1996). The Zierbana Breakwater under construction in 1996 was partially damaged in February 1996. The part of the breakwater with the armor completed with the 100 tn concrete blocks was not damaged; on the contrary, the advanced part of the breakwater protected only with the secondary layer of 8 tn concrete bocks collapsed. Additionally, the crane used for installing the armor units was lost in the storm.

Uzcanga and González(1992) describe the technical and aconomic design criteria and laboratory tests conducted to justify the breakwater design under construction. Taking into consideration the technical information given by the above mentioned authors, the data provided by the waverider buoy operating during the peak of the storm (7-8 February 1996), and the observation of armor damages in different parts of the breakwater, the Zierbana case has been selected to show the possible applications of the exponential model to the analysis of the evolution of the armor damage.

Fig. 8 shows the evolution in time of the H_{m0} and T_{01} measured by the waverider buoy off Bilbao during the damaging storm in February 1996. The wave measurements were taken at one hour interval (about 300 waves/hour), the peak of the storm had about 1,000 waves with $H_{m0} \approx 8$ meters and $T_{01} \approx 12$ sec.; according to Fig. 7, more than 95% of damage is caused by $H_i > 1.2\ H_{m0} \approx 9.6$ meters. The maximum individual wave height is expected to be $(H_{max}) \approx 1.7\ H_{m0}$; therefore, the minimun H_{m0} significant for estimating the armor damage is $(H_{m0})_{min} \approx 9.6/1.7 = 5.6$ meters. Therefore, the damages are expected to be suffered between the 15:00 p.m. (7 Feb 96) and 7:00 a.m. (8 Feb 96) with a probable maximum wave height of 15 meters.

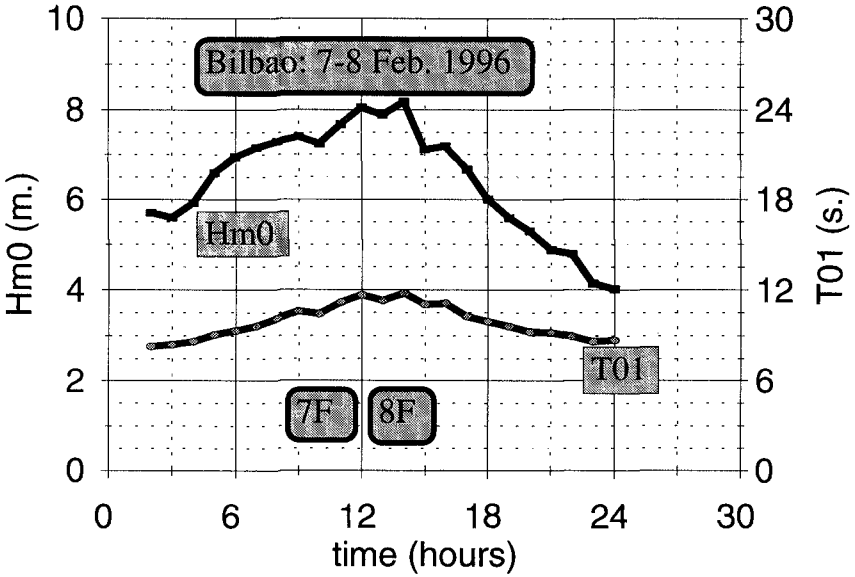

Figure 8.- Evolution of H_{m0} and T_{01} (7-8 February 1996).

Considering the evolution in time of H_{m0} and T_{01} given in Fig. 8, and the numerical simulations described by Giménez et al.(1994) used in Figs. 5 and 6, it is possible to estimate the evolution in time of the damage on the armor layer aplying the exponential model described previously. The technical information provided by Uzcanga en González(1992) are used to link the waverider measurements and structural response to the wave characteristics. Fig. 9 shows the estimations of the armor damage evolution in time considering the exponential model on individual waves with n63%=30, and three different armor blocks (W= 8 tn, 16 tn and 100 tn). The model predicts only minor damages on the armor of 100 tn blocks, and total destruction of the armor of 8 tn, in agreement to what was observed in the Zierbana Breakwater. According to the exponential model, a secondary layer of more than 16 tn was

necessary to resist the wave storm measured during February 1996.

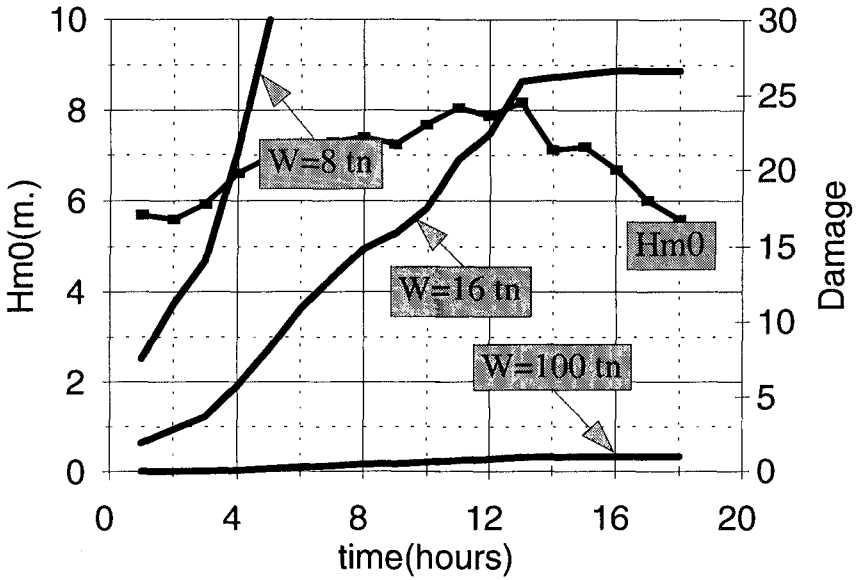

Figure 9.- Estimation of the Evolution in Time of the Armor Damage.

The damages caused in the Zierbana Breakwater suggest the need of re-analyzing the risk associated with the construction during winter months. Considereing the measurements given by the Bilbao waverider buoy before 1991, ROM 0.3-91(1992) provides an estimation of R=7 years (R=3 years is the 90% confidence band limit) for a wave storm of H_s=8 meters. However, it is possible to apply the method given by Rossouw and Medina(1996) to estimate the risk associated with each month independently. Taking into consideration the wave measurements before 1994, and based only on the monthly average and standard deviations, it is possible to estimate the intensity of the storm having a 10% and 2% annual risk of being overpassed during a specific month. Fig. 10 describes the monthly average, $E[H_{m0}]$, the monthly standard deviation, $std[H_{m0}]$, and the H_{m0} (storm duration: Dt = 3 hours) having 10% and 2% annual risk (R=10 and R=50) of being overpassed during each month.

According to Fig. 10, during the months of December and February it is probable to suffer a wave storm of H_{m0} =8 meters (1/R≈10% each month) while there is a low additional risk during January, March, April, October, and November (1/R≈2% each month). The total annual risk of the event {H_{m0}=8 meters, Dt = 3 hours} is approximately 30%; therefore, the damaging storm measured by the waverider buoy off Bilbao shown in Fig. 8 (H_{m0} ≈8 meters, Dt ≈ 3 hours) was likely to happen during the four year construction period of the Zierbana Breakwater. Furthermore, February is the most risky month to work

with partially protected breakwater during construction. The failure of the forecasting system used in Bilbao during construccion may be responsible for loosing the main crane, able to place the 100 tn blocks at 50 meters; however, from the climatic point of view, it is difficult to justify the work in February.

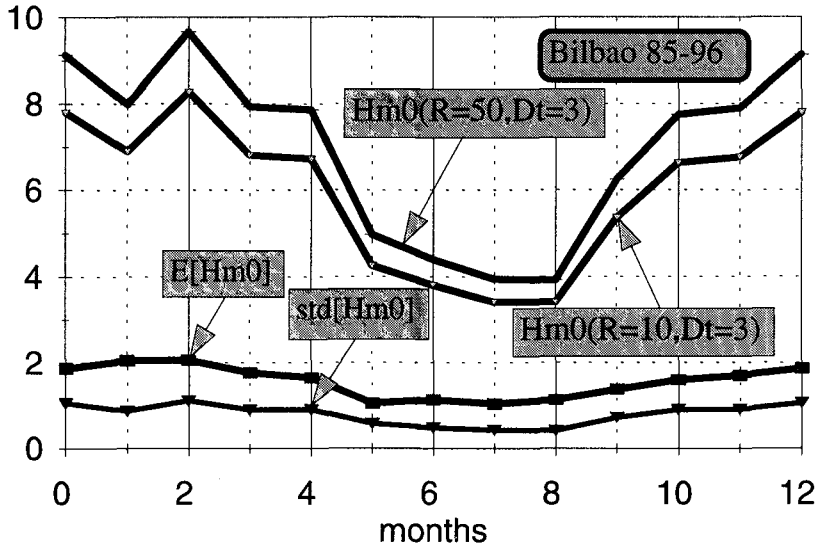

Figure 10.- Characteristics of the Wave Climate in the Coast of Bilbao.

CONCLUSIONS

New models for describing the armor damage evolution in time have to be developed. Existing D(N) models are not appropriate to be applied for nonstationary stochastic models which are the most adequate for modelling real waves. The models must accomplish four basic conditions: (1)unlimited damage if unlimited duration, (2)unsensitive to characteristics of waves below a threshold level, (3)higher relative effect associated with the largest waves, and (4)simple to be applied to nonstationary stochastic models.

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