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Integrated Design of Coastal Structures and Adaptation to Climate Change Hazards

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Abstract

The main objective of this study is to propose an integrated methodology for the design of breakwaters for coastal defence, considering the wave climate change and its impact on the littoral transport in the area sheltered by the breakwater. The joint probability density function of significant wave height and mean wave period, used here, reprentative of the wave climate for a sufficiently long period of years, is estimated from wind data of frequencies of wind velocity combined with fetch length. This methodology has been proved to provide sufficiently accurate results. Besides, a probabilistic design methodology is presented, which uses the aforementioned long-term joint probability distribution of significant wave height and mean wave period and computes the probability of damage levels during the structure's lifetime. This advanced design method is applied to the most prominent subject in the design of coastal structures, i.e. the calculation of the required armour stones in rubble mound breakwaters. For the assessment of the hydrodynamic and morphological impact of the structure, Xbeach model has been applied, based on the probabilistic framework of wave climate. This methodology was implemented by using actual data of a specific location in the northern Aegean; Greece.

Keywords: Probabilistic design, Breakwater, Damage, Armour layer, Morphological change

1. Introduction

It is widely realized that coastal zones are increasingly threatened by environmental forcing under a changing climate. Coastal engineering works should, therefore, be designed to meet these requirements. To ensure coastal structures' stability we need to refer to a probabilistic framework and application of performance based design methods. The Full Probability method is theoretically the right approach for the calculation of structural reliability. However, the estimation of the complete joint probability density function of all stochastic variables involved is difficult. Therefore, First Order Reliability Methods have been developed transforming the initial set of variables into an independent multi-normal set, based on which one can determine the probability of failure by linearization around the design point. Furthermore, Monte Carlo Methods have become a viable tool for engineering design and analysis. Restrictions are often imposed through multivariate cases in which the stochastic parameters involved are correlated. In order to estimate the joint probability density function of the stochastic variables involved in the probabilistic design process, more and more researchers are focused on the study of multivariate distribution models and correlation between the variables. Several methods have been adopted to study bivariate distributions describing the wave climate. One of the first approaches was proposed by Ochi (1978), who adopted the bivariate lognormal distribution to significant wave heights and periods, resulting from an exponential transformation of the bivariate Normal distribution. This approach although simple to apply, requires that the logarithms of the data are normally distributed, and although this may happen for low and moderate significant wave height (Hs), it is not applicable for extreme Hs. A bivariate lognormal with correction for skewness (Fang and Hogben, 1982) was an attempt to improve the bivariate lognormal model. Furthermore, a model based on the marginal distribution of Hs and conditional distribution of peak wave period (Tp) can increase its flexibility and accuracy and has been adopted by Haver (1985) and Bitner-Gregersen and Haver (1989). Guedes Soares *et al*. (1988), Lucas and Guedes Soares (2015) and Papanikolaou *et al*. (2016) have used a similar model based on the marginal distribution of Hs and the conditional distribution of mean zero-crossing wave period (Tm), referred hereinafter as the conditional model. Other approaches have also been developed for the same purpose (e.g. Athanasoulis *et al*., 1994 etc). However, in the cases examined by Lucas and Guedes Soares (2015) and Papanikoalou *et al*. (2016) it was showed that the conditional model fitted more accurately to Hs and Tm data than the other models tested. Thus, in this paper the conditional model is used to fit bivariate distributions of Hs and Tm to the available data sets. Within this framework, the investigation of the sea bed evolution under the presence of a shore-parallel breakwater is required, in order to assess the environmental impact on the coastal zone. Thus, XBeach model (Roelvink *et al*. 2009; Smit *et al*. 2010) has been used for the analysis of the hydrodynamic and morphodynamic conditions in the coastal area near of the structure, considering four different scenarios of sea states.

2. The proposed design methodology and results

2.1 *Wave data fitting*

In our case two data sets have been examined and compared; the first set was wave data of joint frequencies of Hs and Tp during 7 years (1999-2006) derived from the electronic Wind and Wave Mediterranean Atlas (WWM Atlas) and the second one was wind data of frequencies of wind velocity and direction for 64 years (1951-2014) derived from the Hellenic National Meteorological Service (HNMS). It is noted that Tm has been extracted from Tp considering their relationship based on Jonswap energy spectrum of wind waves. The second data set combined with fetch length, has been used to calculate the long-term joint probability density function of Hs and Tm in wind waves at a location in deep waters in Northern Aegean Sea (40.5^{o}N, 25^{o}E). Results from the two data sets have been treated by the conditional model transmitted to 1year period, via the notion of return period associated to each sea state, and are depicted in Figure 1. In Figure 2 the marginal probability density function of Hs from the two data sets is presented. In conditional modelling, the Weibull distribution has been fitted to the marginal Hs data, being an extreme value distribution that could cope with climate change, while the conditional distribution of Tm given Hs followed the lognormal. The Pearson correlation coefficient between Hs and Tm is equal to 0.823 in the case of Atlas wave data and 0.868 in the other set. Results are presented in Figure 1, after the exclusion of swell waves in WWM Atlas data set following Thompson *et al*. (1984). Figures 1 and 2 show good comparisons of the results estimated via conditional modelling using WWM Atlas wave data against HNMS data. Besides, the correlation coefficient is slightly higher in HNMS data than WWM Atlas data. This can be attributed to the structure of the methodology applied to determine waves from wind data, which considers a high correlation between Hs and Tm. Furthermore, HNMS data give also information on the wind direction and therefore of the main wave direction. Thus, the joint probability density distribution of Hs and Tm can be estimated in intermediate waters considering such processes as wave propagation, wave shoaling and refraction. Following this, the joint probability density distribution of Hs and Tm, directed from the open sea towards the structure at the depth of 7.5 m, the supposed site of the structure under design is presented in Figure 3 (left). This depth lies outside the surf zone; to take wave breaking into account would demand knowledge of short term additionally to long-term wave statistics. This topic will not be analysed here.

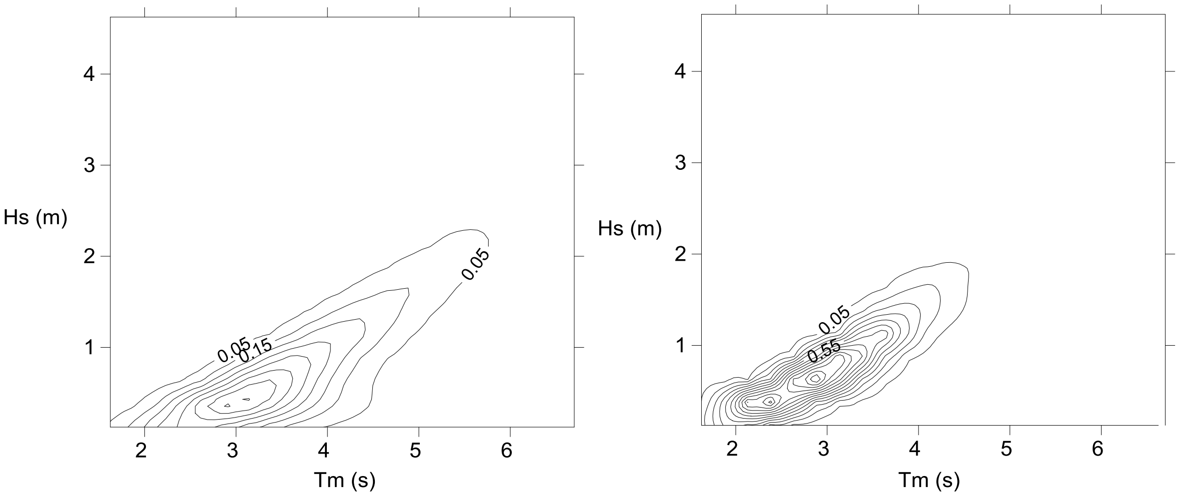

Figure 1. Joint probability density function (1/s m) of Hs(m) and Tm(s) estimated via conditional modelling using Atlas wave data (left) and HNMS wind data (right) – (0.1/s m contour step)

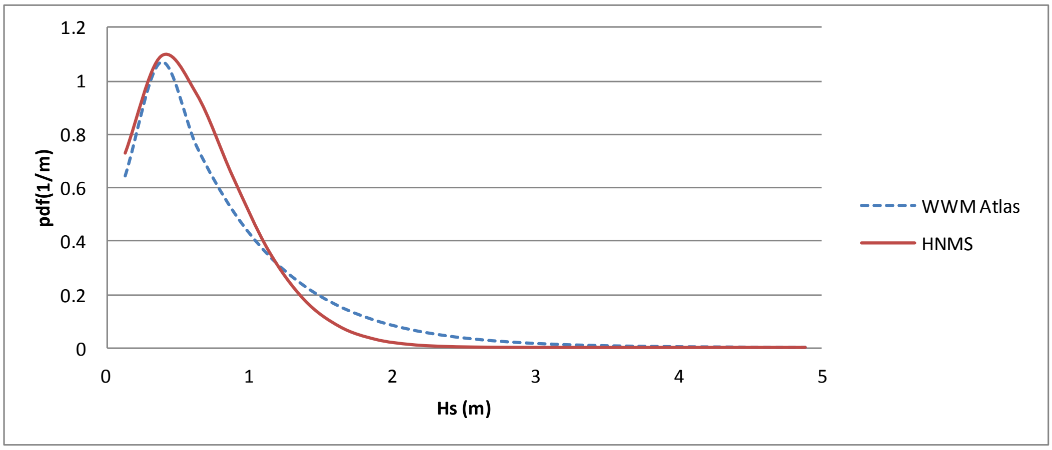

Figure 2. Comparison of probability density function (1/m) of Hs (m) estimated via conditional modelling using Atlas wave data and HNMS wind data (Weibull distribution)

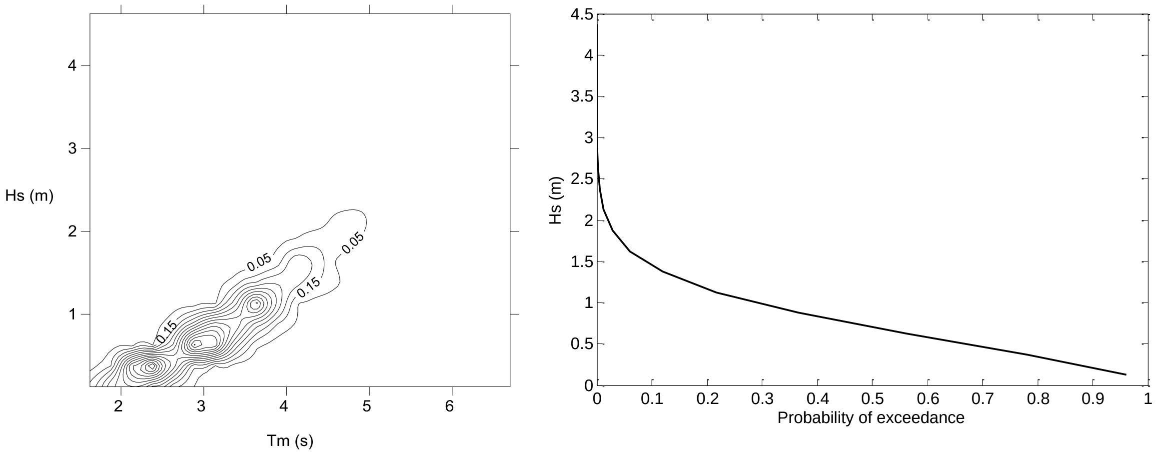

Figure 3. Joint probability density function (1/s m) of Hs(m) and Tm(s) at the depth of 7.5m, estimated via conditional modelling using HNMS wind data (0.1/s m contour step) (left) and Hs exceedance probability curve (right)

The corresponding Hs and Tm at certain exceedance probabilities can be extracted from Figure 3, e.g. from the right panel it is obtained that Hs 2.4 m corresponds to a 0.01 probability of exceedance, while from left panel that the most probable Tm associated to this Hs is 5.1 s.

2.2 Probabilistic armour layer design methodology

In this paper, the direct integration method has been applied to calculate the failure probability of the rock armour layer in rubble mound breakwaters, the most prominent subject in the design of such structures, by giving full consideration of the correlation between two long-term statistical wave characteristics, Hs and Tm. The joint probability density function of Hs and Tm estimated via the aforementioned procedure (§2.1) is used as input data into the failure probability calculation. In this probabilistic methodology only Hs and Tm are considered as stochastic variables, while all other variables are regarded as deterministic. Further research is needed to include more stochastic variables in the probabilistic design procedure, by regarding the joint probability density function of all those variables. In this case, the probability of damage has been calculated via the following integral:

$$\Pr(1\,year) = \int\int_{Z<0} f(H_s, T_m)\,dH_s\,dT_m \qquad (1)$$

where: $f(H_s, T_m)$ is the joint probability density function of Hs and Tm in one year and Z is the reliability function associated with the level of damage. Given the lifetime of the structure (L), the probability of damage or failure during the structure's lifetime is calculated via the following equation:

$$\Pr\,(L\,years) = 1 - [1 - \Pr\,(1 year)]^{L} \qquad (2)$$

It is noted that if the joint probability density function of Hs and Tm in L years was used in equation (1), then the probability of damage in L years would be directly calculated by this equation. The design formulae of Van der Meer (1988) have been used here for the calculation of the reliability function. The damage levels have been classified according to Van der Meer (1988) by the damage parameter S which is equal to 2, 4-6 and more than 8 for initial damage, intermediate damage and failure respectively referred to a breakwater with a 1:2 slope.

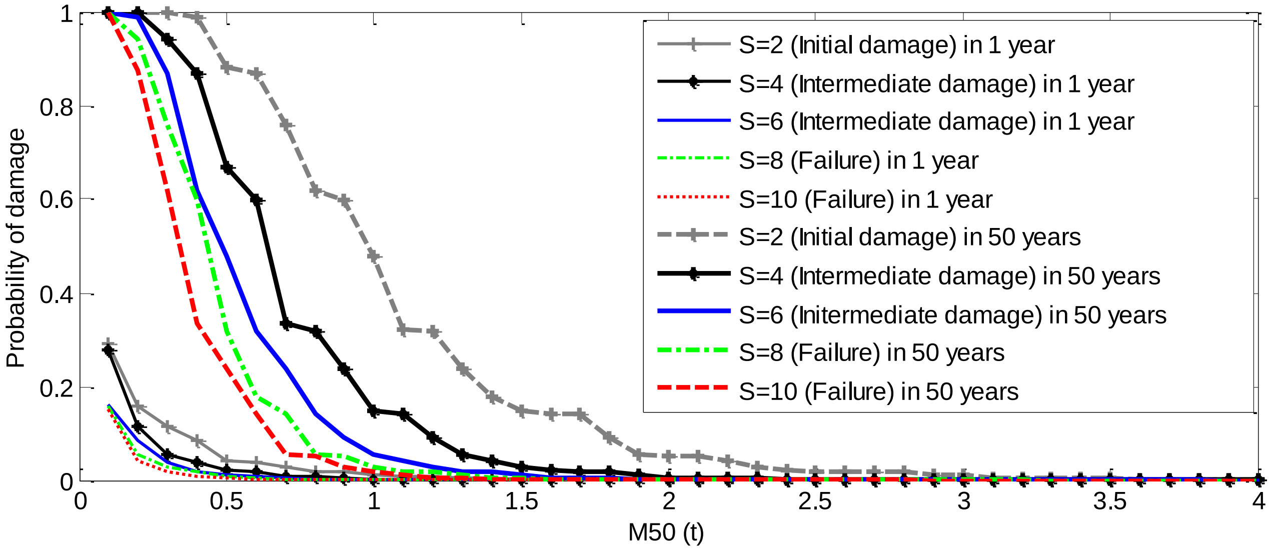

Figure 4. Probability of damage to a rock armour layer of nominal units

The probability of damage to the armour layer in one year and in 50 years, for the supposed lifetime of the rubble mound breakwater under design, is presented in Figure 4. According to design requirements, the acceptable probability of damage level can be chosen firstly and, secondly, this can be associated with the corresponding armour rock unit mass (see Figure 4). This probabilistic methodology can also be applied to other failure mechanisms, e.g. rear-side slope instability, toe instability,etc.

2.3 Morphological impact model

For the assessment of the morphological response under the presence of the breakwater, four scenarios – sea states have been considered. The characteristics of the four scenarios are presented in Table 1. It is noted that scenario 4 is the sequence of the first three scenarios, tested to provide the accumulated erosion of these three scenarios as it happens in nature. The occurrence probabilities of scenarios 1, 2, and 3 have been derived from Figure 3. These three scenarios are considered to be correlated but not fully dependent. Therefore, the probability of occurrence of scenario 4 is estimated here as approximately 80% of the probability of sea state 3. The numerical simulation of the bed evolution has been carried out using the XBeach model. For the needs of this application the height of the breakwater and the distance between the structure and the shoreline have been set equal to 8.3 meters and 200 meters respectively. The goal of this task is to combine the stochastic analysis of sea states with the corresponding impact on the shoreline. The results of bed evolution under the four scenarios tested are presented in Figure 5. As it was expected, the higher level of beach erosion is observed for the sea state 4 (see Figure 4.d). The beach profiles obtained show that the amount of erosion is dependent on the intensity of sea states, and the change of the bed shape becomes negligible as the wave height decreases (see Figure 5.a). Besides, the final accumulated eroded area of the bed profile in sea state 4 is about 20% higher than in sea state 3 only.

3. Conclusions

Based on the above analysis, it can be seen that the proposed wave data estimation methodology can produce sufficiently accurate results and has significant advantages. The joint probability density function of Hs and Tm can be produced straightforward via this procedure and can be used as input data into the probabilistic design method introduced in this paper. Besides, the assessment of the hydrodynamic and morphological impact of the structure can also be based on this probabilistic framework of wave climate. By this manner wave climate and its changes can be incorporated during the design process.

Table 1. Characteristics of the four sea state scenarios tested

Sea sate scenario	Hs (m)	Tm(s)	Duration (hr)	Probability
1	1.5	4.08	6	0.0190
2	2.0	4.58	6	0.0056
3	2.5	5.11	6	0.0012
4	1.5,2.0,2.5	4.08, 4.58, 5.11	18	0.0010

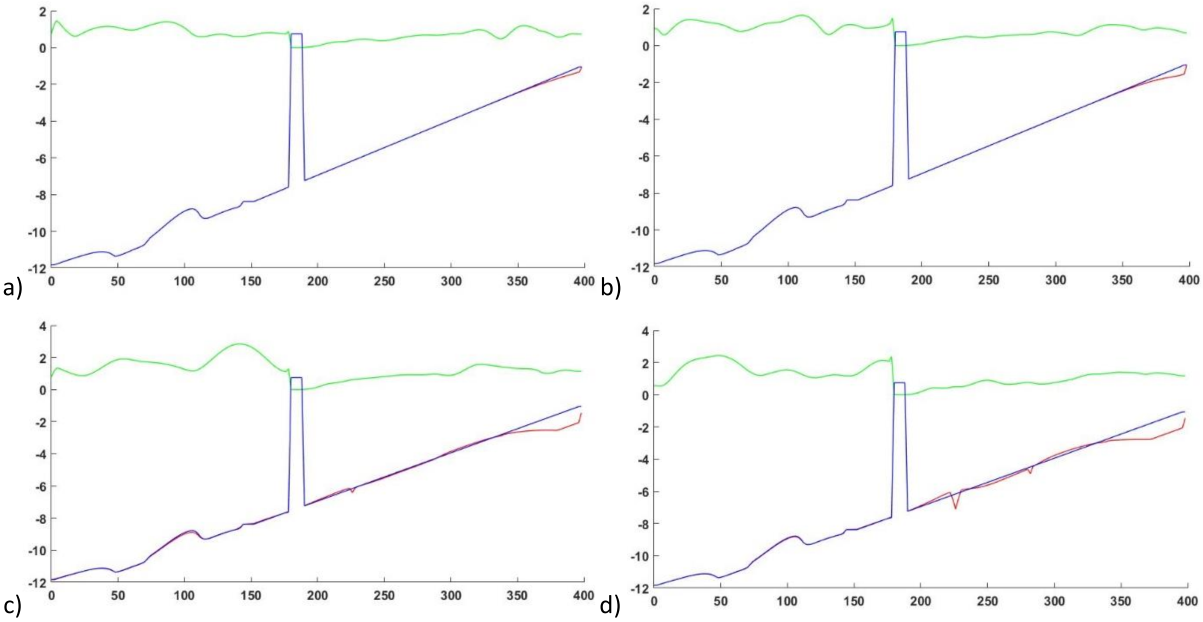

Figure 5. Bed evolution for scenario 1(a), scenario 2 (b), scenario 3 (c) and scenario 4 (d) obtained by XBeach model, under four selected sea states. Initial bed (blue lines), final bed (red lines) and wave height (green lines), illustrated for the last time step of each simulation

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