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Reliability Analysis of Berm Breakwaters based on the environmental data on the Northern coasts of Oman sea

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ABSTRACT

Berm Breakwaters (BB) are reshaping coastal structures which can be classified as Hardly, Partly, and Fully Reshaping (HR, PR, and FR) breakwaters based on their stability. This study evaluates the reliability of different berm breakwaters using environmental data from the Iran's southern coast region. The D-Vine structure was identified as the most suitable for analyzing dependencies between input variables. Two failure modes, wave overtopping and berm recession were used to assess reliability through current deterministic design methods. The findings show that dependencies between input variables generally had a minor impact on the calculated probability of failure (Pf). However, ignoring these dependencies can lead to underestimated designs, especially when wave height and water level strongly correlate. The study also found that the current deterministic design criteria for berm breakwaters are more conservative compared to conventional rubble mound breakwaters. This is due to the additional berm width based on Dn50. Among different berm breakwater types, HR breakwaters, with larger armor units and smaller berm recessions, had the lowest berm failure probability. In contrast, the overtopping Pf values is almost the same for all the BB types. The results of this study shows the importance of including dependencies between input variables and paying attention to the required safety margin in reliability design of berm breakwaters based on conventional meethods.

1. Introduction

One of the most important coastal structures for shielding harbor areas from waves is a berm breakwater (BB). The idea of Berm Breakwaters was first presented more than 40 years ago in order to improve the use of rock in the conventional rubble mound breakwaters. The provided berm in these breakwaters keeps changing shape in response to wave attacks until it stabilizes. Typically, the final stable profile resembles an S-shaped curve. Compared to traditional breakwaters, this idea enables the use of smaller armor stones within a wider berm [1]. The berm breakwaters are usually classified based on their reshaping state. Many studies were performed in the last years to investigate the stability and hydraulic performance of berm breakwaters such as [2], [3], [4], [5], [6], [7], [8], [9], [10], [11], [12], [13], [14], [15], [16], [17], [18], [19], [20], [21]. The stability number (H0) for damage prediction was first proposed by [22]. [23] added the wave period and presented a new stability number (H_0T_0) as part of their ongoing research. [24] discovered in another experimental study that $H_0\sqrt{T_0}$ is more capable of presenting the berm damage parameter S_d for HR breakwaters than H_0T_0. [9] and later [2] examined the cumulative effect of prior storms on the final berm recession and proposed a modified stability number ($H_0\sqrt{T_0}$) based on wave energy, taking into account the greater influence of wave height than the wave period. Research is still being done to determine the best stability number to use when presenting the BB behavior. However, the probabilistic response of berm breakwaters at the design point under different uncertainties has not been examined in any of the earlier studies.

Usually, the type of berm breakwater determines berm recession. Based on the dimensionless variables H_0T_0 and H_0, the PIANC guideline [25] classified berm breakwaters for the first time in 2003 into three groups: statically stable non-reshaped, statically stable reshaped, and dynamically stable reshaped berm breakwaters. Later, [16] introduced new classifications as Hardly reshaping (HR), Partly reshaping (PR), and Fully reshaping (FR) types, modifying this one based on their final reshaping against design wave (100 years in their definition). FR breakwaters that have considerable recession are classified as mass-armored berm breakwaters (PR-MA), while HR breakwaters with a limited recession are also known as Icelandic-type berm breakwaters (HR-IC). Both mass-armored (PR-MA) and Icelandic-type (PR-IC) berm breakwaters are susceptible to partial reshaping. The design of coastal and marine structures is fraught with uncertainty. Simple deterministic methods are typically preferred by designers, but they don't reveal anything about the probability of failure. Moreover, it is unclear—and occasionally disregarded—how safety factors are used in the current deterministic codes for coastal structures. Nearly all design formulas for coastal structures are derived using central fitting to test data, according to [26]. As a result, these semi-empirical equations did not model any uncertainty, and the only safety margin that is presented is the chosen return period. However, in order to account for a safety margin, characteristic loads and resistance values—typically with one fractile distance to the mean values—are frequently used in structural codes.

Few studies have been conducted for the fully probabilistic design of coastal structures, especially breakwaters, in contrast to the structural field. A new Bayesian formula for predicting the force acting on caisson breakwaters was presented by [27], [28], who also conducted a reliability analysis on these structures without taking environmental parameter dependencies into account. One of the elements affecting how applied loads are distributed is the correlation between input parameters. For instance, the design of breakwater crest elevation may be influenced by the close correlation between wave height and water level [29], [30], [31], which may change the probability of breakwater failures. A study on a rubble mound breakwater using different copula dependency structures was recently carried out by [32]. According to their findings, the independent, Gaussian, and two-variable Gumbel structures had lower construction costs than the D-Vine structure by 9.32%, 5.7%, and 3.12%, respectively. In addition, [33] suggested a novel probabilistic method for designing desalination outfall systems close to coastlines with the required safety levels, taking into account a correlation between environmental data. They demonstrated how dependency structures have a major impact on failure probability. Compared to coastal structures, offshore structures have a longer history of reliability analysis using dependency structures. For example, [34] compared the results of multiple pairs of copula families between wave height, period, and wind speed in order to examine the impact of copula dependency structures on offshore platform design. Their findings demonstrated that the best fits for these data sets were given by Gaussian and Gumbel copulas. As an additional illustration, [35] used independent, Gaussian, and D-Vine copula dependency structures to examine the probability of structural failure of an offshore platform on Canada's east coast. Furthermore, [36] used a mixed copula structure to examine the structural reliability evaluation of an offshore platform and showed that the dependency structure can significantly impact reliability analysis.

As was mentioned in the paragraphs above, no specific research was done to evaluate the dependability of berm breakwaters with various reshaping classifications. This study examines the effects of various parameters in an attempt to fill this gap. Following the results obtained in section 5 of this paper, the effects of the breakwater stability number and dependency structures on the results are discussed for this purpose. Before that, Section 2 reports the study region along with the statistical features and interdependencies among environmental data. Following that, section 3 discusses the deterministic design criteria, and section 4 introduces the reliability method concept and the used limit state functions.

2. Input data and Dependency modeling

2.1 Study area and Environmental data

The region under investigation is situated along the Oman Sea on Iran's southern coast (Figure 1). With a maximum depth nearly 13.7 meters in relation to the site Chart Datum (CD), this breakwater safeguards one of Iran's most significant harbors. In this region, monsoon waves predominate, and the regional wave rose shows that most of the waves come from the south. According to research by [37], the combination of higher frequency waves during non-monsoon times and large waves during the southwest monsoon is what causes the multi-peaked wave spectra in this area. Some researchers have examined the dominant wave systems and provided appropriate wave spectra for this region based on the wave measurements that are currently available [38], [39].

Figure 1. Locations of the study area in Iran.

Following the study by [32], the environmental data (Hs, Tp, and WL) recorded with a capture rate of 99% over 22 years, from January 1994 to December 2015, are used in this study. In contrast to their study, this one looks into the probability of failure against design conditions (with a 100-year return period). The Chabahar buoy provided the Hs and Tp data for this data set [38], while the IOC station measurements, whose dependability and quality were confirmed by earlier research, provided the WL data combining tidal levels and storm surges [40], [41], [42].

2.2 Dependency Structures

Copulas are effective tools for modeling how variables depend on one another. They make it possible to separate dependency structures from marginal distributions and to generate joint probability distribution functions. This separation provides flexibility in the modeling process and is especially helpful when working with intricate and unusual patterns. The relationship between copula functions and marginal distributions is shown in Figure 2.

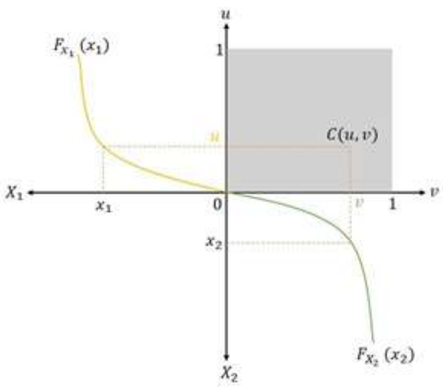


Figure 2. Relationship between bivariate marginal distributions and copula functions [43]**.**

A multivariate cumulative distribution function with uniform edges on a super plane $[0,1]^M$ is called an M-dimensional copula.

$$F_X(x_1, x_2, \dots, x_M) = C(F_{X_1}(x_1), F_{X_2}(x_2), \dots, F_{X_M}(x_M)) \quad (1)$$

$X = (X_1, X_2, \dots, X_M)$ indicates an m-dimensional random sequential marginal distribution functions $F_{X_1}, F_{X_2}, \dots, F_{X_M}$, and C is the copula. This allows us to express the joint distribution function using a copula.

Despite their popularity due to their ease of use, Gaussian copulas assume that marginal distributions follow a Gaussian pattern and have limitations when it comes to modeling tail dependencies [44]. Other copula families can be used to solve this problem. As an example, the coastal engineering field has made extensive use of the hierarchical and nested Archimedean copulas as well as Vine copulas [45].

In this study, four dependency structures have been studied in accordance with [32] and Copula modeling was done using MvCAT and Python's pyvinecopulib package. Independent variables, Gaussian copula for all variable pairs, bivariate Gumbel copula for the Hs-Tp pair, and D-Vine copula for all variable pairs are examples of these dependency structures.

3. Deterministic design Criteria

As mentioned in the previous paragraphs, the stability number Ns was used to define three types of berm breakwaters, including HR, PR, and FR. HR breakwaters typically have less berm recession and higher required armor weights than other types. Three designed sections that are compatible with HR, PR, and FR berm breakwaters have been taken into consideration in this study, in accordance with the previously mentioned classification. Fig. 3 shows a schematic view of the variables and the berm breakwater section.

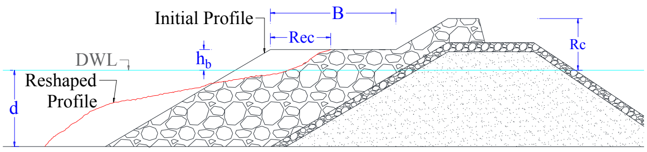


Figure 3. A schematic view of the variables and the berm breakwater section

3.1 Berm geometry: (B and hb)

As a crucial geometrical parameter, berm width should be sufficiently large to resist design waves and the resulting recession. As a result, the berm recession and the required safety factor determine its value. The introduction of a formula for predicting the berm recession has been attempted numerous times in the literature, including by [2], [3], [4], [5], [6], [7], [8], [9], [10], [11], [12], [13], [14], [15], [16], [17], [18], [19], [20], [21]. Numerous formulas based on various variables and presumptions have been proposed by these studies. While other formulas might include other variables such as berm level, wave period, water depth, and so forth, the formula put forth by [16], for instance, solely depends on H_0. A new berm classification and design methodology was presented by [16], who based it solely on H_0 as the most critical berm recession parameter.

Despite neglecting other factors, they ultimately came to the conclusion that berm recession is significantly influenced by other factors like berm level, seaside slope, toe depth, and cumulative recession. As a result, they applied positive or negative scores to represent the impact of these parameters. This study evaluates the berm recession in a deterministic design or when defining the limit state functions using a new proposed formula by [11]. Genetic programming and fitting to a wide range of parameters were used to derive this equation. A researcher can examine the uncertainties of various variables on the berm breakwater performance by using this equation in the reliability framework because it contains a large number of geometrical and environmental parameters. Therefore, it is applied in this study to predict berm recession under various hydrodynamic and geometrical circumstances.

The required berm width can be designed with a safety margin once the berm recession under design wave conditions has been determined. However, according to (Van der Meer and Sigurdarson, 2016), this value should be verified using the minimum permitted berm width, which is $B_{min} = \max(3D_{n50}, Rec + 1D_{n50})$. The literature suggests limited design recommendations about the safety margin value. [25] made a well-known suggestion that restricts the residual berm width following reshaping to $4D_{n50}$ for the ultimate limit state. Since Rec is the berm recession under the design wave, the required berm width should be at least $B_{Pianc} = Rec + 4D_{n50}$. (Van der Meer and Sigurdarson, 2016) provided the alternative suggestion, which was based on the breakwater type and resiliency level. According to Table 1, they suggested that the minimum berm width be $B_{VS} = Rec/P\%$, where P is the desired resiliency. Based to their definition, a structure's resilience is its ability to recover back from or withstand design waves. They stated that an HR breakwater is more resilient than the other two types and proposed a resiliency value based on the classification of berm breakwaters.

Table 1. The proposed resiliency values for berm design [25]

BB Category	Ns	P%	B/Rec
HR	1.5-2.0	10–20	5-10
PR	2.0-2.5	20–40	2.5-5.0
FR	2.5-3.0	≤70	1.4-2.5

as a result, the recommended berm width for HR breakwaters is (5~10) Rec, whereas for FR type, it is (1.4~2.5) Rec. The required berm width would be more in FR than in HR because the recession in FR breakwaters is more than in HR. Finally, because resilience (P%) can increase the cost of the Berm breakwater, (Van der Meer and Sigurdarson, 2016) recommended that the resilience should be selected based on the required safety level by the designer.

The berm level can be a little above the design water level or much higher, as discussed by [16]. A minimum berm level of h_b=0.6H_{sD} above DWL was also recommended by them. According to (Van der Meer and Sigurdarson, 2016), the minimum berm level should not be less than MHWS+dW, where dW varies between 0.5 and 1.5 m depending on the annual wave characteristics.

3.2 Crest Freeboard height: (Rc)

The crest elevation is one of the geometrical parameters that affects the overtopping rate and hydraulic response of berm breakwaters, in addition to the berm geometry. In other words , the value of Rc is determined for each permitted overtopping. The design overtopping rate q_{Do} (one standard deviation higher than the mean overtopping ($\overline{q_o}$), which will be used later for the definition of the overtopping limit state function), was proposed in the Eurotop manual as follows based on the research conducted by (Van der Meer et al., 2018):

$$q_{Do} = 0.1035 \times exp\left[-\left(1.35\frac{R_c}{H_{m0}\gamma_{BB}\gamma_\beta}\right)^{1.3}\right] \times \sqrt{gH_{m0}^3} \tag{2}$$

where, especially in deep waters, the spectral wave height, H_{m0}, is close to the value of Hs. The height of the crest above the design water level is known as the crest freeboard height (Rc). When S_m is the wave steepness, the influence factor for a berm is γ_{BB}, which is γ_{BB}=0.68-4.1S_m-0.05 B/H_{m0} for HR and PR berm breakwaters and γ_{BB}=0.7-8.2S_m for FR breakwaters. Finally, since most of waves come from the south side, the effect coefficient of the wave angle, γ_β, is one. The allowed overtopping value, q_{all}, is depends generally by the breakwater's required performance under design and annual waves.

3.3 Other geometrical parameters

Table 6 shows the values and design criteria taken into consideration for the other parameters. The thickness of the armor layer (ha), filter layer (u), and crest width (Cw) can be calculated after the armor diameter is determined as Dn50. The height of the filter layer above the core materials (hf) depends on the require freeboard and is calculated as $hf = Rc + DWL - hc - ha$. Finally, as shown in the table, the height of the core layer (hc) is chosen to provide a minimum safe working level for construction activities.

Table 2. The geometrical parameters criteria in the berm breakwater

Geometrical parameters	Considered design criteria
ha	2 D_{n50}
u	0.8 D_{n50}
C_w	4 D_{n50}

hc	d0+WL_{1y}+0.6Hs_{1y}

4. Reliability Analysis and Limit State Functions

Limit state functions should first be defined for each failure mode i in order to conduct reliability analysis and failure probability investigation. $G_i = R_i - S_i$ is the limit state function that shows the structural resistance minus the load effect on the structure. This definition states that when the structure response surpasses its resistance, or when G_i becomes negative, the structure fails. A several of reliability methods can then be used to determine the probability of failure after introducing the R and S as probable variables. The probability of failure $P_{fi} = P(G_i \leq 0)$ is calculated in this study using the Monte Carlo simulation method. It is calculated as $P_{fi} = N_f/N_t$, where N_t is the total number of samples and N_f is the number of samples with $G_i \leq 0$. When the calculated Pf is less than the target or accepted probability of failure, the design is accepted.

The mechanism through which the structure or one of its elements occurs is indicated by the failure mode [47]. Limit state functions can be used to assess the various types of failures that berm breakwaters can undergo. These include overtopping, breakwater core settlement, bed soil settlement instability, and—above all—the failure of the berm breakwater width. The damage brought on by the berm breakwater core settling is not considered critical given the properties of the berm breakwater and the soil at the study site; as a result, it has not been included in the analysis of this study. Finally, this study examined two failure mechanisms as independent limit state functions, denoted by G_i, where $i = B, O$ represents failure resulting from excessive berm recession and failure resulting from excessive overtopping, respectively. The following sections provide a more thorough introduction to these functions.

4.1 Failure due to berm recession

This type of failure occurs when environmental loads cause the berm recession (Rec) to be exceeds than the allowed berm recession or structure resistance. As discussed in the previous sections, the formula presented by [11] is used in this study to evaluate the Rec (as the load S in the limit state function). According to (Van der Meer and Sigurdarson, 2016), the remaining berm width must be at least equal to $1D_{n50}$ so as for the structure resistance R, also known as the allowed berm recession, to be defined. Stated differently, when the predicted recession against probabilistic variables exceeds (B - $1D_{n50}$), berm failure occurs place. The limit state function for berm failure, as described above, would be:

$$G_B = [B - 1D_{n50}] - C_R\left[\left(0.02067\left(H_0\sqrt{T_{0P}}\right)^{1.971}\left(\frac{d}{D_{n50}}\right)^{-0.1087} Ln\left(\frac{N}{3000}\times\frac{d}{D_{n50}}\right) + 21133\left(H_0\sqrt{T_{0P}}\right)^{2.094}\left(\frac{h_b}{H_s}\right)\left(\frac{d}{D_{n50}}\right)\times\left(Ln(f_g)\right)^{7.187^{fg}} - 81.74\times10^{-5}\times 1.378^{\left(\frac{h_b}{H_s}\right)}\left(H_0\sqrt{T_{0P}}\right)^{3}\left(\frac{N}{3000}\right)\left(\frac{h_b}{H_s}\right)^{2} - 1.245\right)\times D_{n50}\right] \tag{3}$$

There are some uncertainties resulting from the prediction formulas for wave overtopping and berm recession in addition to the uncertainties of the environmental variables that show up in the S parameter. This uncertainty is actually represented by C_R in this equation. [11] proposed a distribution around the mean value of berm recession as Eq.(3). According to their findings, the variable C_R's mean and coefficient of variation (CoV) are chosen for this study to be 1.0 and 0.1, respectively.

4.2 Failure due to wave Overtopping

The second failure mode this study addresses is failure mode on by overtopping. According to the guidelines proposed by Eurotop 2018, the overtopping discharge is calculated [46]. It should be noted that the distribution of overtopping discharge is assessed in this section using the mean and standard deviation of the overtopping rates. In section 5.3, however, the design overtopping formula (one standard deviation higher the mean values) was used for the deterministic design. The following definition of the corresponding limit state function to the overtopping failure is based on the discussions above:

$$G_O = q_{all} - \left(C_{O1}\times exp\left[-\left(C_{O2}\frac{R_c}{H_{m0}\gamma_{BB}\gamma_\beta}\right)^{1.3}\right]\times\sqrt{gH_{m0}^3}\right) \tag{4}$$

The overtopping rate is utilized in this equation, and its distribution around a mean value is controlled by the coefficients C_{O1} and C_{O2}. According to Eurotp 2018, these coefficients have respective mean values of 0.09 and 1.5, as well as coefficients of variance of 0.0135 and 0.15. The other parameters are the same to those that are included in Eq.(4). To create a design condition with a safety factor, the C_{O1} and C_{O2} coefficients were actually changed to constant values (one standard deviation from the mean) in that equation ($C_{O1+1\sigma} = 0.1035$, $C_{O2-1\sigma} = 1.35$).

5. Results and discussions

5.1 Design environmental data

The wave height time series and the histogram plot for environmental variable pairs appear in Fig. 4.

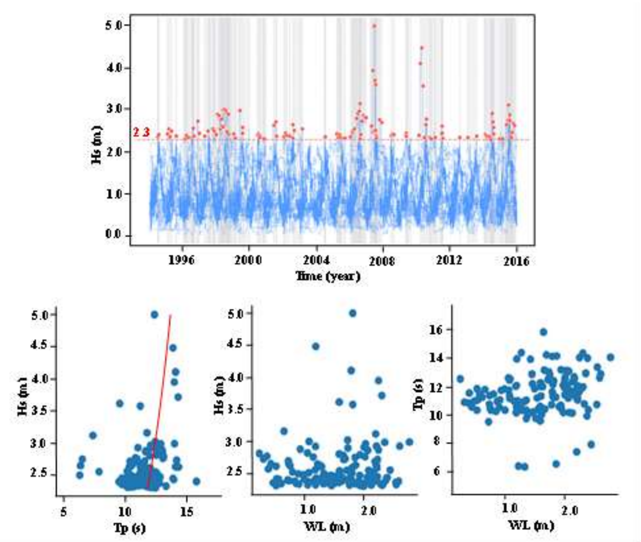


Figure 4. Histogram plot for pairs of environmental variables

In order to calculate the design values with a specific return period, as well as evaluate the dependencies between the variables, these data were utilized in this study. Finding a threshold that distinguishes large waves from those presenting the characteristics of design waves is the first step in both of these analyses. Following that, design events can be estimated as discussed in the paragraphs that follow, and section 3.2 can be used to calculate the dependency structure between the variables. The threshold wave height of 2.3 was threshold by [32] using two mean residual life and parameter stability techniques on the same data set in order to identify the waves with design characteristics. Using the same method, the design wave with 100 years of return periods was found to be H_{sD}=5.03 m by fitting the Weibull distribution to the chosen waves that were higher than 2.3 m. According to [48], Figure 4 shows a solid red line that represents the relationship between the mean wave height and peak wave period, which is $Tp=10.1Hs^{0.19}$. T_{pD}=13.6 s would be the mean value of the design wave period based on this relationship. The deterministic design of the berm breakwater was based on this design wave. However, in order to account for the uncertainties in a reliability design, a distribution function around these mean values is required. The normal distribution was chosen for the DWL variable and the Lognormal distribution for the H_{sD} and T_{pD} variables based on the measurements. The coincident design WL is an important variable in addition to the design wave height and period. The predicted berm recession and wave overtopping rate are impacted by changes in the values of the crest freeboard (Rc) and the berm level (hb), which are both dependent on the DWL value. A reliability design takes consideration of the uncertainties surrounding the mean design values (H_{sD}, T_{pD}, and DWL). For calculating the design values separately, wave parameters (Hs, Tp) and water level (WL) can be examined separately. For example, in the area under research, a design wave with H_{sD}=5.03m and T_{pD}=13.6m occurs once every 100 years. The maximum water level, however, is 3.04 meters every 100 years. The correct combination of (or jointed) mean design values for any desired return period is up for discussion, though. Whether or not these maximum events occur at the same period every 100 years is the question. The joint distribution of the variables in the region determines a response to this question. In general, it is expected that the return period corresponding with a concurrent situation will exceed 100 years. A common practice for initial or traditional design purposes is to make conservative assumptions that take into consideration all of the maximum values at once. For example, assuming that design waves are jointed with DWL= MHWS+storm surge. The storm surge for the 1-year and 50-year return periods was 0.21m and 0.57m, respectively, and the MSL (Mean Sea Level) and MHWS (Mean High Water Spring) elevations in the studied area are 1.55m and 2.47m with respect to the site CD. Such an assumption results in a DWL of 3.04m based on these values. If a joint probability analysis (JPA) were conducted, this value would be less.

Table 3 shows the jointed and not-jointed design variables that have been modified for the annual and 100-year return periods. H_{sD}=5.03m, T_{pD}=13.6s, and DWL=2.59m are the mean values of the joint design variables that were used to design the berm breakwater in this study. Therefore, the jointed design water depth would be d=16.3m based on the still water depth at the location of the berm breakwater under study (d0=13.7 m, CD). The table shows the water depth and annual wave parameters in addition to the chosen 100-year design period. The top level of the core layer and the minimum height of the berm level will also be determined by the design using these values.

Table 3. Design wave parameters in the area based on the performed JPA

Return period (years)	Hs_y (m)	Tp_y (s)	Jointed WL_y (m)	Not jointed WL_y (m)
Annual: 1	2.23	11.67	1.64	2.68
Design: 100	5.03	13.6	2.59	3.04

As previously mentioned, the breakwater is situated on the northern coasts of the Oman Gulf, where the design water depth is d=d0+DWL. The mean value of DWL is 2.59 meters, and d0 is 13.7 meters in respect to CD. Table 4 shows environmental variables along with their distribution functions. Actually, these variables cause uncertainty in the outcomes. The uncertainty in H_{sD} is corresponding to a coefficient of variation of approximately 5 to 20 percent, as per [26]. The small sample size, measurement errors, hindcast model errors, etc., may all contribute to this uncertainty. According to the measurement type, the suggested CoV (Coefficient of Variation) for the design peak wave period T_{pD} falls within the same range. As shown in Table 4, a CoV of 0.15 is chosen for both wave height and period in this investigation. Storm surges and tidal levels consists DWL, but tidal uncertainty is lower than storm surge uncertainty. As a result, the literature recommended that the CoV of storm surge and tide elevation be between 0.1-0.25 and 0.001-0.07, respectively [26]. To study for the uncertainties in both surface water level and seabed variations, a CoV value of 0.2 is chosen for DWL in this investigation. In addition, all of the variables' distributions and standard deviations are selected to have a physical basis in accordance with the characteristics of the field under study and the materials at available. Since $N = Ts/Tm$, the number of waves is affected by storm duration Ts.

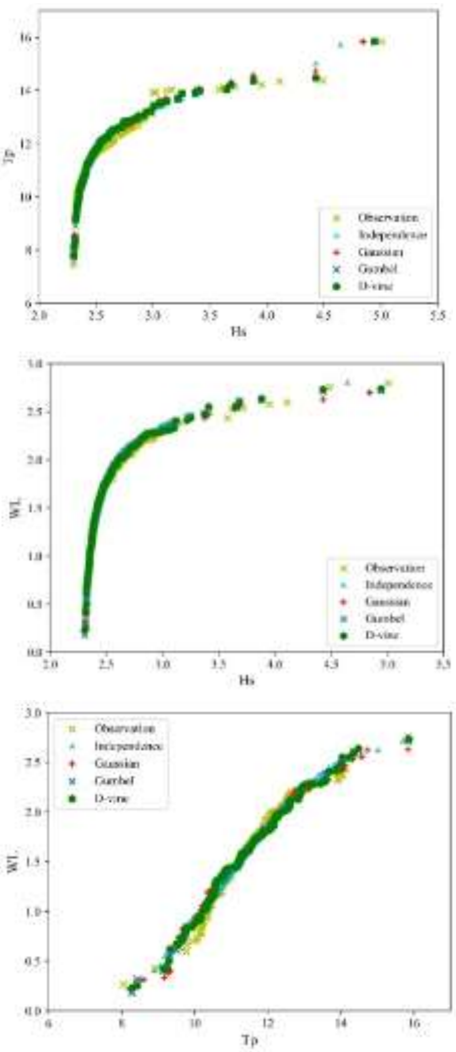
Table 4. Environmental variables and their distribution

Variable index	Variables	Distribution	Mean	CoV	Unit
5	H_{sD}	Lognormal	5.03	0.15	m
6	T_{pD}	Lognormal	13.6	0.15	s
7	ρs	Normal	2450	0.02	kg/m^3
8	ρw	Normal	1025	0.01	kg/m^3
9	DWL*	Normal	2.59	0.2	m
10	Ts	Triangular	9	1/3 (s.d.=3)	hr
11	fg	Normal	1.5	0.05	-

The seabed depth in respect to CD is indicated by the water depth, d=d0+DWL, where d0 is a constant value.

5.2 Goodness of fit for dependency structures

Four dependency structures are considered in this study: D-Vine, Gaussian, Gumbel, and independent copula. Figure 5 shows the performance of these structures compared to the measurements. The Log-Likelihood and Akaike Information Criteria (AIC) indexes are also computed in evaluating the fit's the suitability for these structures. The results showed that the D-Vine copula best fit the AIC index of -22.93 and the total Logik index of 13.47. Then, with total Logik values of 8.93 and 7.71 and AIC values of -13.86 and -9.43, respectively, the Gumbel and Gaussian copulas offer the best fits in turn. As anticipated, the independent condition produces the worst fit. As anticipated, since the same input data were used, the results were the same to those reported by [32]. The D-Vine copula is chosen as the base dependency structure based on the results.



a) Pair *Hs-Tp*

b) Pair *Hs-WL*

c) Pair *WL-Tp*

Figure 5. Comparison between the observations and the dependency model predictions for the pairs of environmental variables

5.3 Deterministic design of sections

Table 5 shows the required berm width based on the two above references. According to the table, the recommended berm width by [16] generally much larger (by a maximum of 2 to 4 times for HR type) than the values proposed by [25]. This could be because of the particulars of the area under study. The berm width is taken into consideration in accordance with PIANC's recommendations based on these values and the built berm breakwater in this area. In general, a more conservative structure with less failures is probability by a wider berm. Although being helpful in deterministic design, design recommendations don't tell us anything about the structure's probabilistic performance.

Table 5. The values of design parameters in the deterministic design

Design parameters	Berm Breakwater Category		
	HR	PR	FR
Ns	1.85	2.25	2.75
hb (m)	3	3	3
Rc (m)	6.55	6.29	6.96
B (m)	12	13.8	17
Rec	4.19	7.41	11.73
Rec+4Dn50	12	13.8	17
Rec/P%	21~42	19~37	17~29

In addition to the berm width, Eq.(3) indicates that the berm level (hb) affects the berm recession and the stability of the breakwater. According to previous studies, a higher berm generally dissipates more wave energy and results in less berm recession. Given that dw is 1.0 meters above MHWS, a minimum value of hb=0.9 is used in this research. The values reported in Table 5 can be derived from these values and the corresponding wave and water levels in Table 3.

5.4 Reliability analysis of the designed sections

The Pf of the berm width designed using the deterministic recommendations generally less than the Pf resulting from excessive overtopping discharge, as shown in Table 6. A safety margin is causes for the berm width by adding an extra width of 4Dn50 to the predicted mean value of the berm recession (Table 5), and for the overtopping limit state by applying an overtopping prediction based on a formula that is one fractile higher than the mean value (Eq.(4)). Table 6 shows that a safe HR breakwater with 3.88E-02 Pf was designed by applying a safety margin of $4D_{n50}$, while a FR breakwater designed using the same criterion had a Pf of 2.13E-01. In addition to the HR breakwater's smaller berm recession, the larger armor units (D_{n50}) are the cause. On one hand, the structure response (recession) was reduced, and on the other hand, the resistance (R) has been excessively raised. Compared to the other types, these two factors provided a more reliable HR breakwater. However, the calculated Pf for the overtopping limit state is almost the same for all of the HR, PR, and FR breakwater types as shown in the table due to the fractile concept was used for the deterministic design of the breakwater crest elevations. It should be noted that the Pf values for berm recession (in Table 6) would be less if a wider berm were applied according to (Van der Meer and Sigurdarson, 2016) recommendation based on the resiliency factor (P%). Similarly, if the remaining berm width was considered smaller than $4D_{n50}$, the *Pf* would be increased.

Applying different limit state functions, design criteria, breakwater types, and other areas with varying environmental uncertainties can alter the results presented above. For example, the required armor weights are typically determined using the predicted mean values without a safety margin in the deterministic design of conventional rubble mound breakwaters. As a result, the *Pf* determined for the berm breakwaters would be less than their corresponding *Pf*. In other words, based to existing deterministic criteria, a designed berm breakwater is typically more dependable than a designed conventional breakwater. In conclusion, a probabilistic design provides information about the probability of failure, while a deterministic design does not. Furthermore, in the case of probabilistic design, it is possible to introduce different target Pf values for different failure mechanisms. Designers can use it to investigate the reliability index of an existing structure, design a new structure based on the desired probability of failure, and account for all the uncertainties.

The results shows that the effect of dependency structures on the calculated Pf is much less than the effect of the types of the berm breakwater. However, the independent condition results in smaller Pf values in general for both the recession and overtopping failure modes. It means that neglecting the correlations between input variables leads to underestimated designed sections and it is necessary to include the dependency structures for a safe design.

Table 6. Probability of failure for the deterministic designed sections

Failure	Dependency Structures	HR	PR	FR
		Pf	Pf	Pf
Berm recession	Gumbel	3.73E-02	1.14E-01	2.12E-01
	Gaussian	3.47E-02	1.14E-01	2.15E-01
	D-Vine	3.88E-02	1.16E-01	2.13E-01
	Independent	2.58E-02	9.85E-02	1.99E-01
Wave Overtop ping	Gumbel	3.03E-01	3.09E-01	2.76E-01
	Gaussian	3.05E-01	3.11E-01	2.77E-01
	D-Vine	3.04E-01	3.09E-01	2.80E-01
	Independent	2.94E-01	3.00E-01	2.57E-01

6. Conclusions

By focusing on a berm breakwater on Iran's southern coast, this study examines the reliability of different berm breakwaters. The measurements in this region were used to extract the environmental data. The D-Vine structure was determined to be the most appropriate one based on the statistical indicators after different dependency structures between the input variables were studied to identify their importance on the results. Reliability of these structures was calculated following the design of different berm breakwater types using the deterministic methods currently in use. Two failure modes, such as wave overtopping and berm recession, were used for this. Dependency structures and the effects on different breakwater types were considered. Based on the results, the main conclusions are outlined below.

• In order assist designers achieve adequate berm widths that align with the required probability of failure, two deterministic design concepts were presented in this paper. While the second concept introduces a resiliency factor, the first concept involves a predetermined number extra armor units. In general, the proposed design values by Van der Meer and Sigurdarson, 2016 is much more conservative than using extra armor units as PIANC's recommendations.

• Because HR breakwaters have larger armor units and smaller berm recessions than the other two types, they have a lower probability of berm failure among the other types of berm breakwaters. However, the Pf for overtopping failure mode is almost the same for all types of berm beakwaters. In addition, the Pf due to overtopping failure mode is generally less than the pf values for berm recession criterion.

• In the studied area, dependency structures between the input variables had a generally insignificant impact on the calculated Pf values. Neglecting the dependencies, however, can result in an underestimated design for regions where wave height and water level have a strong positive correlation. This is especially true for the breakwater crest elevation, which depends on the water level and wave height that coincide. In addition, among the various dependency structures examined, the D-Vine structure turned out to be the most accurate. For both limit state functions, the independent assumption was associated to the maximum error. This result emphasizes how crucial it is to consider variable dependencies into account in specific situations in order to ensure accurate and reliable breakwater designs.

• The results show that, compared to conventional rubble mound breakwaters, the current criteria for the deterministic design of berm breakwaters are notably conservative. The primary cause of this result is the use of an additional berm width as a function of Dn50.

8. References

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