Reliability-based design optimization of a rubble mound breakwater in a changing climate

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ABSTRACT: The study concentrates on a fictitious rubble mound breakwater used to protect the marina situated near the entrance of the port of Le Havre, in France. A maximum failure probability is assigned and the structure should meet the requirements. Different failure modes and their relation to the overall failure of the structure are represented in a fault tree. Failure of the water retaining structure happens when the water enters the protected area uncontrolled. Both Ultimate Limit States (ULS) and Serviceability Limit States (SLS) are considered under present climate conditions, as well as utilizing basic assumptions for the future marine conditions in the study area. The set of acceptable geometries for the system is defined based on the probabilistic constraint of the maximum failure probability and an economic optimization of the total lifetime costs, namely the costs of construction and failure of the structure.

1 INTRODUCTION

The fact that a large amount of the world’s population and the majority of economic activity are assembled in coastal areas, combined with the general inception of a globally changing climate, increase exposure of coastal areas to flooding. Therefore, there is a widespread need for a more reliable design of marine and coastal structures able to withstand the increased loading, while ensuring that manufacturing and performance costs are kept low. During the last few years, probabilistic design approaches based on reliability and risk-based design concepts have been increasingly proposed and applied in the fields of civil engineering and water defences, because of their ability to provide an explicit measure of the safety level of the structured system under study. Reliability-based design optimization, implemented in the present study, deals with obtaining optimal designs characterized by low costs and by a low probability of failure.

Reliability-based design is defined as a design approach where the probability of failure is used as a measure of the performance of the coastal or marine structure. A maximum failure probability is defined and the structure should meet the requirements. In order to make a probabilistic optimization, the mechanisms that lead to structural instability or losing of a specific level of functionality should be considered. Therefore, different failure modes and their relation to overall failure of the structure should be represented in a fault tree. Quantitative analysis starts at the level of failure modes with the definition of limit state functions and the probabilistic analysis of all random input variables. From a quantification of the probability of occurrence of each failure mode, the total failure probability of the structure can be defined. Reliability-based optimization based on minimizing the total cost function of the structure can be performed, to find a cost-effective design which also meets the probabilistic requirements.

Reliability-based optimal structural design has been applied by numerous authors in the past (Van Dantzig 1956, Grigoriu et al. 1979, Stedinger 1997, Moses 1998 and many others). Voortman (2003) developed a methodology that enables the use of quantitative methods for the risk-based design of a large scale flood defence system. In his thesis, he used reliability-based design as a tool in risk-based design of flood defences. Buijs et al. (2003) described a test application of a reliability method developed for ring dykes in the Netherlands to the flood defence system in South Wales and provided recommendations for the future development of reliability-based methods for flood management in the UK. Steenberg et al. (2004) presented an overview of an advanced program called PC-ring for the reliability analysis of flood defence systems and described the global data requirements for the application and the setup of the model. Within the European Research Project FLOODsite (2004-2009, www.floodsite.net), flood defence reliability analysis has been further
developed to support a range of decisions and adopt different levels of complexity. The research in the field of reliability analysis within the project, focused on developing techniques and incorporating present process knowledge on individual failure modes as well as interactions between failure modes on the feasibility, the preliminary and the detailed design level of complexity. Burch & Sorensen (2005) presented results from numerical simulations performed with the objective of identifying optimum design safety levels of conventional rubble mound and caisson breakwaters, corresponding to the lowest costs over the service life of the structures. Dai Viet et al. (2008) presented a probabilistic approach for designing the breakwater system of the South of Dooson Naval Base in Vietnam, combining reliability analysis of the governing failure mechanisms with an economic optimization process. Van Gelder et al. (2009) provided an outline of methods and tools for reliability analysis of flood defenses, as well as for flood risk analysis. They focused on the background of probabilistic analyses, uncertainties and system analysis and presented an application of the methodologies at a study site of the European Research Project FLOODsite, the German Bight.

The present study focuses on reliability-based optimization of a rubble mound breakwater under present conditions and in a changing climate, in the marine area of the north-western French city Le Havre. Le Havre is situated on the English Channel and is a famous port city with a high touristic development. More specifically, the study will concentrate on a fictitious rubble mound breakwater used to protect the marina situated near the port entrance. The studied rubble mound breakwater is of a similar design with the one present in the port of Deauville, situated on the opposite bank of the Seine estuary. In Section 2 of the present study, the load data used to perform reliability-based design optimization are presented and analysed. The governing loading variables of the studied structure are represented as random variables and appropriate distribution functions are selected to represent uncertainties in their estimates. Some important design quantities are then estimated using simple formulas, by means of a deterministic design approach. In Section 3 the proposed probabilistic methodology is presented and implemented. The main failure modes of the structure are considered and appropriate limit state functions are defined in accordance with the selected failure criterion. A maximally acceptable failure probability is defined and reliability analysis is performed to define the space of acceptable geometries of the structure. Finally, the cost of all acceptable geometries within the solution space is estimated and an economical optimization is performed. Section 4 includes the principal conclusions of the study.

2 ANALYSIS OF AVAILABLE DATA

In the present research reliability-based optimization is applied in the marine area of the north-western French city Le Havre, located on the English Channel. The port of Le Havre is one of the largest in France. Le Havre marina, on which the present study focuses, is on the northern side of the port and is exposed to wave action and sea levels caused by high astronomical tide and storm surges. Hydraulic loading conditions are available for the site of Le Havre, by means of simulation data. Data of the marine climate cover a period of 10000 years. These data include significant wave height, storm surge, tide and water level simulations. Each value of the simulated datasets corresponds to the peak of each tide, i.e. one every 12-13 hours. The datasets available are stationary and are used in the present work to represent present climate conditions, ignoring the possible effects of climate change on the marine climate.

Limit states used for breakwater design, are states beyond which the structures no longer satisfy the requirements and can be distinguished in ULS (Ultimate Limit States), FLS (Fatigue Limit States), PLS (Progressive collapse Limit States) and SLS (Serviceability Limit States). In the present paper, only the ULS and the SLS of the studied breakwater system are examined. The former is assumed to happen under extreme marine conditions, while the latter is affected by the normal marine climate. Considering the Ultimate Limit State of the studied structure, extrapolation of the marine variables to high enough return periods has to be performed. Methods and techniques of the well established univariate and multivariate Extreme Value Theory (EVT) can be used for this purpose. Considering the Serviceability Limit State, normal wave height conditions are analyzed, using a distribution type for short term prediction.

Extreme value methods are powerful statistical methods for drawing inference about the extremes of a process, using only data on relatively extreme values of the process. The statistical methodology is motivated by a well-established mathematical theory (EVT), which relies on the assumption that the limiting models suggested by the asymptotic theory continue to hold at finite but extreme levels (Coles, 2001). Nevertheless, a crucial assumption in fitting distribution functions to data is that the data are independent and identically distributed (iid). Univariate analysis of significant wave height and storm surge level extremes is performed using the GEV (Generalized Extreme Value) distribution function for annual maxima. The GEV distribution function is of the form:

$$G_\xi(x) = \exp\{ -[1 + \xi \frac{x - \mu}{\sigma}]^{-\frac{1}{\xi}} \}$$  \hspace{1cm} (1)
where $\sigma > 0$ and $\theta = (\mu, \sigma, \zeta)$ the vector of parameters, namely the location ($\mu$), scale ($\sigma$) and shape ($\zeta$) parameters, determined by the tail behavior of $G$. The model of Equation 1 is fitted to annual maxima wave heights and storm surges and the parameter vector for both variables is estimated by means of the L-moments estimation procedure. For wave heights, the extreme tail is described by the parameter vector $\theta_1 = (4.19, 0.48, -0.078)$, while for extreme surges the respective parameter vector is $\theta_2 = (0.69, 0.14, 0.109)$.

Where the source of risk consists of more than one variable, it is necessary to consider their combined probability. Extreme offshore wave heights are often strongly dependent on high sea levels. The observed water level is the sum of a deterministic astronomical tidal component and a stochastic meteorologically induced component, the surge. Dependence between surges and waves is expected, since both are related to local weather conditions (Hawkes et al. 2002). Especially at extreme levels strong dependence is likely, when meteorological systems which generate extreme surges also cause strong onshore winds from a direction having a long fetch. Therefore, dependence between the variables of wave height and storm surge should be calculated. It should be emphasized that componentwise maxima do not necessarily correspond to maxima originating from the same storm. The complete pair of measures of extremal dependence $\chi$ and $\overline{\chi}$ (Coles, 2001), is informative for both asymptotically independent and dependent variables. When used for bivariate random samples with identical marginal distributions, both measures provide an estimate of the probability of one variable (e.g. wave heights) being extreme, provided that the other one (e.g. surge levels) is extreme. However, from the pair of measures $\chi$ and $\overline{\chi}$ estimated from the bivariate dataset of annual maxima, there is no strong evidence that extreme wave heights and storm surges in Le Havre are consistent with asymptotic dependence.

The high design water level (without the storm surge component) in front of the breakwater is described by means of a Normal distribution function with mean, $\mu \approx 7$, and variance $\sigma^2 = 0.25$. The Normal distribution function is also utilized to model wave steepness, $s \sim N(0.066, 0.006^2)$ during extreme wave conditions.

Considering the Serviceability Limit State, normal wave height conditions are analyzed. More specifically, it is assumed that mean daily wave heights follow a Rayleigh probability distribution function:

$$F_\theta(x) = 1 - \exp\left\{\frac{-(x - \mu)^2}{\sigma^2}\right\}$$

with parameter vector $\theta = (\mu, \sigma)$. L-moments estimation procedure is used to calculate the parameter vector $\theta_3 = (0.023, 0.677)$ of the Rayleigh distribution function.

Considering future climate conditions, because of the fact that no data for the future marine conditions are yet available at the site of interest, some general estimations are used in the present study in a rather primitive way. From ONERC (http://onerc.developpement-durable.gouv.fr/), three estimates of sea level rise due to global warming in 2100 are available: an optimistic scenario of +40cm, a pessimistic scenario of +60cm and an extreme scenario of +1m. Based on the shortage of information available, the distribution function for sea level for the future conditions is assumed similar to the one used for present conditions, but with an increased mean value based on the optimistic scenario of ONERC. For offshore wave conditions, CETMEF (http://www.cetmef.equipement.gouv.fr/) indicates an increase between 20 and 40cm of the offshore wave heights along the French coast up to 2100, according to emission scenario B1 and A2, respectively. In the present study an increase of 30cm in the mean extreme wave height conditions is assumed. Therefore, the extreme tail of wave heights is described by the parameter vector $\theta_4 = (4.49, 0.48, -0.078)$, retaining the scale and the shape parameters of the present climate distribution function unchanged.

Le Havre marina accepts boats without any tidal stress. Nowadays it offers around 1160 mooring rings, a wedge launch and a bunkering station. Several services such as electricity and water supplies are available for the users. The fictitious rubble mound breakwater, with a length of approximately 1 km, protects the marina against undesirable wave and water level action. Before commencing with a reliability-based optimization of the defence, a deterministic estimation of its main geometrical features should be performed for illustrative purposes.

In a deterministic approach, input conditions are given by discrete values and the breakwater’s geometry usually follows common design guidelines. The studied rubble mound breakwater is of a similar design concept with the one present in the port of Deauville, situated on the opposite bank of the Seine estuary. The typical cross-section of the breakwater under study includes a core made of limestones, a secondary armour layer made of quarry rock (rip-rap) a primary armour layer of concrete blocks and a wave wall made of concrete. The slope of the seaward primary armour layer is 1/2. The land side slope of the breakwater is 2/3. The Hudson formula is used to estimate the weight of the primary armour layer blocks (CEM, 2006):

$$W = \frac{\rho_c H^3}{K_D (\rho_c/\rho_w - 1)^3 \cot a}$$

with $\rho_c$ and $\rho_w$ as the densities of concrete and water, respectively, and $H$ the wave height.
where $H$ = the characteristic wave height; $\rho_c$ = the density of concrete blocks; $\rho_w$ = the water density; $K_d$ = the stability coefficient; and $a$ = the sea side slope angle. The characteristic wave height in this case is defined as the return level corresponding to a return period of 100 years, the reference period for both the ULS and SLS functions. Equation 1 and the estimated parameter vector $\theta_1$ are therefore utilized to estimate the 100-years significant wave height, $H_{100\text{years}} \approx 6.05\text{m}$. Tetrapod blocks are selected for the 2-layer primary armour layer, each one weighting 21.5t (Equation 3). The secondary armour layer is made of 2-layer quarry rocks weighting about one tenth of the armour unit in the primary cover layer (CEM, 2006).

The selected crest elevation should be the lowest that provides the protection required. For crest elevation the formula of Eurotop (2008) is used in the present work:

$$ q = 0.2C_r \exp(-2.6 \frac{R_c}{H_{s\gamma_f}}) $$

where $q$ = the overtopping discharge; $C_r$ = the reduction factor due to the effect of armoured crest berm, defined as a function of the crest berm width ($B$) and the wave height ($H_{mo}$); $\gamma_f$ = the influence factor for roughness and is set equal to 0.38; and $R_c$ = the crest freeboard. The crest berm, $B$, used in the definition of the reduction factor $C_r$ is considered equal to the combined width of three armour units (Equation 5). The maximum tolerable overtopping discharge in the marina area is set equal to 0.005m$^3$/s. The resulting deterministic crest freeboard is estimated $R_c \approx 6.2\text{m}$ using the 100-years significant wave height, $H_{100\text{years}} \approx 6.05\text{m}$. The structure under study also includes a wave wall, which ends 2m above the crest berm. A typical cross-section of the breakwater studied in the present paper is presented in Figure 1.

3 RELIABILITY-BASED OPTIMIZATION

3.1 Reliability analysis

Reliability-based design is a design approach where the probability of failure is used as a measure of the performance of the structure. A reliability analysis should start with a definition of the main function that is considered for the studied flood defence system. In the present work, failure of the breakwater to fulfill its requirements, leads to an unwanted “top event” of water entering the marina area uncontrolled. Therefore, a fault tree that gives a logical succession of all events leading to this unwanted “top event” can be constructed. This fault tree can contain both the Ultimate and the Serviceability Limit State of the breakwater. In the first subtree, which refers to the ULS of the breakwater, the objective of the structure is the protection of the marina against flooding. Within this objective, the “top event” of water entering the marina area uncontrolled. Therefore, a fault tree that gives a logical succession of all events leading to this unwanted “top event” can be constructed. In the present work, failure of the breakwater to fulfill its requirements, leads to an unwanted “top event” of water entering the marina area uncontrolled. Therefore, a fault tree that gives a logical succession of all events leading to this unwanted “top event” can be constructed. In the present work, failure of the breakwater to fulfill its requirements, leads to an unwanted “top event” of water entering the marina area uncontrolled. Therefore, a fault tree that gives a logical succession of all events leading to this unwanted “top event” can be constructed. In the present work, failure of the breakwater to

![Figure 1. Typical cross-section of the proposed breakwater](image)
each one of them under extreme marine conditions can cause malfunction of the whole breakwater system. Three main failure modes are considered here as the main types of instability under extreme conditions of the marine climate, namely the instability of the primary armour layer, the excessive wave overtopping and the erosion of the toe of the breakwater. The reliability function constructed for primary armour layer stability is based on the van der Meer (1988) formula for two layer armoured non-overtopped slopes covered with Tetrapods:

$$\frac{H_s}{\Delta D_n} = f_i (3.75 \frac{N_{od}}{N^{0.25}} + 0.85) s_{om}^{-0.2}$$  \(8\)

where $H_s$ is the significant wave height in front of the breakwater; $D_n$ is the length of cube with the same volume as Tetrapods defined as $D_n = V^{1/3} = \left(M/\rho_c\right)^{1/3}$, where $V$, $M$ and $\rho_c$ represent the volume, the mass and the density of blocks, respectively, $f_i$ is a coefficient denoting the difference in the slope angle of the tested model and the real design; $N_{od}$ is the number of units displaced out of the armour layer within a strip width of one cube length $D_n$; $N$ is the number of waves; $s_{om}$ is the wave steepness; and $\Delta = (\rho_v/\rho_w) - 1$ where $\rho_v$ is the water density. Apart from the probability distribution functions of the loading variables of Equation 8, defined in Section 2, the number of waves, $N$, is considered to follow a Normal distribution function with $\mu = 3000$ and $\sigma = 200$, while $N_{od} = 0.5$ to correspond to the no damage level in relation to the Hudson formula stability coefficient (CEM, 2006). Therefore, the reliability function for primary armour stability is:

$$Z = f_i (3.75 \frac{N_{od}}{N^{0.25}} + 0.85) s_{om}^{-0.2} \Delta D_n - H_s$$  \(9\)

The reliability function constructed for the erosion of the toe of the structure is based on the following Burcharth et al. (1995) formula:

$$\frac{H_s}{\Delta D_{n_{50}}} = \left(0.4 \frac{h_b}{\Delta D_{n_{50}}} + 1.6\right) N_{odoc}^{0.15}$$  \(10\)

used for the stability of the toe formed by two layers of parallelepiped concrete blocks. In Equation 10, $D_{n_{50}}$ is the equivalent cube length of median block, $h_b$ is the water depth at the toe defined as the sum of high design water level and storm surge, while $N_{odoc} = 2$ to allow some flattening out of the toe. The reliability function used for toe stability of the breakwater is:

$$Z = \left(0.4 \frac{h_b}{\Delta D_{n_{50}}} + 1.6\right) N_{odoc}^{0.15} \Delta D_{n_{50}} - H_s$$  \(11\)

For excessive wave overtopping, the formula of Eurotop (2008) (Equation 6) is used in the present work to derive the reliability function for overtopping. The reliability function considered for the above mentioned failure mode is:

$$Z = q_l - 0.2C_e \exp(-2.6 \frac{R_c}{H_{sp}^{0.9}}) \sqrt{gH_s^2}$$  \(12\)

where $q_l$ is the tolerable overtopping discharge, which for the marina area is set equal to 0.005$m^3$/s.

Excessive wave height in the marina basin is considered as a Serviceability Limit State (SLS) of the breakwater under study. The maximum tolerable wave height in the marina basin during normal weather conditions is set at $H_t = 0.5m$. The wave height inside the marina basin is considered a combination of wave refraction - diffraction via the entrance of the marina and wave transmission through and overtopping the breakwater (Dai Viet et al. 2008). The reliability function constructed to represent the SLS is:

$$Z = H_t - (K_{diff} + K_{trans}) H_s$$  \(13\)

The transmission coefficient, $K_{trans}$, is substituted in Equation 13 by the formula used in “The Rock Manual” (CIRIA, CUR, CETMEF, 2007) $K_{trans} = 0.46 - 0.3R/H_t$. The diffraction coefficient, $K_{diff}$, is approximated using the methodology presented in Goda (2010), assuming an angle of wave crest with respect to bottom contours where the wave enters shallow water equal to 30°. Its resulting estimate is $K_{diff} = 0.245$.

After defining the ULS and SLS reliability functions, as well as the probability distribution functions of the input variables, level III reliability methods are used to estimate the failure probabilities corresponding to each failure mode. Level III methods calculate the probability of failure, by considering the probability density functions of all strength and load variables and therefore the reliability of an element is linked directly to the failure probability (CUR, 2002). More specifically, Monte Carlo simulation techniques are used to extract the failure probabilities. The number of simulations for each failure mode is 10000, in order to estimate the failure probability with good confidence. The probability of failure is approximated for each failure mode by $P_f = n_f/n$, where $n$ is the total number of simulations; and $n_f = \#$ of simulations for which $Z < 0$. After defining the failure probability for each individual failure mode, fault-tree analysis is utilized to extract the total failure probability of the structure.

3.2 Economic optimisation

The boundary conditions for breakwater design include hydraulic boundary conditions (wave height, water level, storm surge and tide), as well as geometrical and geometrical conditions. Considering the stochastic variables of the marine climate, the distribution functions derived earlier (Section 2) are
used for the variables entering the limit state functions. Boundary conditions of both present climate and future climate conditions are used as input in the present work. A fault tree is then constructed including both an ULS and a SLS for the breakwater under study. This fault tree comprises all the possible limit states of the studied structure (Section 3). A large number of possible alternative geometries can then be tested, according to their performance for the limit state functions. For each such geometry, a failure probability is defined using probabilistic and numerical methods (Section 3). To decide whether a geometry is applicable in one structural concept, an acceptable value of the failure probability should be prescribed. The combination of the calculated failure probabilities and the probability constraint define the set of acceptable geometries as (Voortman, 2003):

\[ D = \{ z | P_f (z) \leq P_{f,\text{max}} \} \]  

(14)

where \( z \) is the vector of the design variables; and \( P_{f,\text{max}} \) = the maximum acceptable failure probability of the breakwater. In the present study, the design variables that differentiate between various alternatives are the weight of the primary armour layer blocks, \( W \), and the freeboard height, \( R_c \), while the maximum acceptable failure probability is \( P_{f,\text{max}} = 10^{-2} \) per year.

Equation 14 provides a large number of alternative geometries, which fulfill the probabilistic constraint. To select among these solutions, it would be really useful to estimate the costs of every alternative geometry. Therefore, a geometry is selected which satisfies the probabilistic constraint, while minimizing the economic expenses. In the present study only the costs of construction and the expected costs of failure (Dai Viet et al. 2008) are taken into account in the optimization process. The direct costs of construction can be written as a function of the design variables:

\[ I_{\text{cons}} = I(z) = L(A_{\text{armour}}(z)I_{\text{armour}}(z) + A_{\text{core}}(z)I_{\text{core}}(z)) \]

(15)

where \( z \) = the vector of design variables; \( L \) = the length of the structure; \( A_{\text{armour}}, A_{\text{armour}}, A_{\text{core}} \) = the areas of the cross sections of the primary armour layer, the secondary armour layer and the core of the breakwater, respectively, while \( I_{\text{armour}}, I_{\text{armour}}, I_{\text{core}} \) = the respective costs by volume.

The expected costs of failure are calculated for the total lifetime of the breakwater, which in the present work is set to 100 years and represent economic damage in case of failure, as well as costs of repair. The formula to extract the costs of failure is (Dai Viet et al. 2008):

\[ I_{\text{failure}} = \sum_{i=1}^{M} C_{\text{ULS}} P_{f,\text{ULS}}(z) \left( \frac{365}{(1+r)^i} \right) + \sum_{i=1}^{M} C_{\text{SLS}} P_{f,\text{SLS}}(z) \left( \frac{365}{(1+r)^i} \right) \]

(16)

where \( C_{\text{ULS}} \) and \( C_{\text{SLS}} \) = damage costs and \( P_{f,\text{ULS}} \) and \( P_{f,\text{SLS}} \) = probabilities of failure in case of ULS and SLS, respectively; \( r \) = the interest rate; and \( M \) = the reference period for ULS and SLS failure. The probability of failure for the ULS, \( P_{f,\text{ULS}} \), is expressed in an annual scale, while the probability of failure for the SLS, \( P_{f,\text{SLS}} \), is expressed in a daily scale. To estimate the ULS damage cost, \( C_{\text{ULS}} \), the cost of repairing the breakwater is assumed to reach up to 20% of the initial construction cost, while economic damage costs include loss of direct income from the operations of the marina, as well as loss of indirect income, caused by bad reputation and competitive marinas. To estimate direct economic damage it is assumed that when a collapse by an extreme storm occurs, the marina has to cut its capacity by half and this can last for up to half a year. Serviceability Limit State costs, \( C_{\text{SLS}} \), are estimated per day of downtime of the operations in the marina basin. Daily losses of direct and indirect income are also estimated for each such day. The reference time, \( M \), for the ULS and the SLS in the present work is set to \( M = 100 \) years. The interest rate is considered 5%, for both the present as well as the future conditions. The structure of the optimization process is presented in Figure 2.

Before minimizing the total lifetime costs, a more simple minimization is performed. The failure probability is \( P_f = 10^{-2} \) per year and the space of acceptable geometries is defined as:

\[ D = \{ z | P_f(z) = 10^{-2} \text{ and } \min I_{\text{cons}}(z) \} \]

(16)

Optimization is performed in this case for the weight of the Tetrapod, or the equivalent length of cube with the same volume as the Tetrapod, \( D_n \), and the freeboard height, \( R_c \), by a minimization of the construction cost of the breakwater (Equation 15). Results of the optimization process for \( D_n \) and \( R_c \), for
both present and future climate conditions are given in Figures 3-4. Points correspond to estimated costs for selected pairs of design variables, while lines result from the fitting of polynomial functions to the estimated costs.

The optimum solution is found at $D_n = 2.08 \text{m}$ and $R_c = 6.5 \text{m}$ for present climate conditions and at $D_n = 2.21 \text{m}$ and $R_c = 7 \text{m}$ for future climate conditions. The equivalent weights of the Tetrapod blocks are $W \approx 19.8 \text{ t}$ and $W \approx 23.8 \text{ t}$, for present climate and future climate conditions, respectively. Tetrapod units are heavier for future conditions up to 20%, while the crest level of the breakwater is estimated about 7.7% higher compared to the solution for the present climate. Comparing the extracted results for present climate data with those of the deterministic design of Section 2, it can be noted that the latter overestimates the Tetrapod weight up to almost 9%, while underestimates crest height up to almost 5%.

When failure costs are also considered and the set of acceptable geometries is defined by Equation 14, where $P_{f, \text{max}} = 10^{-2}$ per year, results of the optimization process with regard to the block equivalent cube length $D_n$, and the crest height, $R_c$, for present and future climate conditions are presented in Figures 5-6, respectively.
The total cost optimization procedure results in $D_n = 2.1 m$ and $R_c = 6.9 m$ for present climate conditions and $D_n = 2.26 m$ and $R_c = 7 m$ for future conditions.

The equivalent weights of the Tetrapod blocks are $W \approx 20.4 t$ and $W \approx 25.4 t$, for present climate and future climate conditions, respectively. Tetrapod units are heavier for future conditions up to 24.5%, while the crest level of the breakwater is estimated only 1.5% higher compared to the solution for the present climate. Comparing the extracted results for present climate data with those of the deterministic design of Section 2, it can be noted that reliability-based optimization presents lighter Tetrapod blocks (up to 5.5%), but higher crest heights (up to 11.5%). The main differences between design techniques are attributed to the use of an economic optimization procedure. When deterministic methods are implemented, the structure is designed to satisfy the hydraulic requirements, but the costs of protection can not be considered. Reliability-based optimization helps in obtaining optimal designs characterized by low costs, which can withstand a combination of different limit states.

4 CONCLUSIONS

In the present study reliability-based design for a breakwater structure in the marine area of the French city Le Havre is implemented both for present and future climate conditions. The breakwater system protects the marina of Le Havre against undesirable wave and sea level action. A maximum failure probability equal to $10^{-2}$ per year is defined and the structure should meet the requirements. In order to make a probabilistic optimization, the mechanisms that lead to structural instability or losing of a specific level of functionality are considered. Therefore, different failure modes and their relation to overall failure of the structure are represented in a fault tree. This fault tree can contain both the Ultimate and the Serviceability Limit State of the breakwater. Three main failure modes are considered here as the main types of instability under extreme conditions of the marine climate, namely the instability of the primary armour layer, the excessive wave overtopping and the erosion of the toe of the breakwater. Excessive wave height in the marina basin is considered as a Serviceability Limit State (SLS) of the breakwater under study. The set of acceptable geometries for the system is defined based on the probabilistic constraint of the maximum failure probability and an economic optimization of the total lifetime costs, namely the costs of construction and failure of the breakwater.

The optimization procedure for the total lifetime costs results in a rubble mound breakwater with lighter Tetrapod blocks (up to 5.5%) and higher freeboard height (up to 11.5%) compared to the ones estimated following a deterministic procedure. Comparison between optimized geometries for present and future climate conditions shows an increase of the weight of Tetrapod blocks and of the crest height up to 24.5% and 1.5%, respectively. When a constant failure probability $P_f = 10^{-2}$ is used and only the construction cost is minimized, Tetrapod units and crest level of the breakwater for future climate conditions are up to 20% and 7.7% higher, respectively, compared to the estimates for present marine conditions. The deterministic approach, as in the case of total lifetime cost minimization, seems to overestimate the Tetrapod weight and to underestimate the crest height.

In a general design problem, not only the armour unit weight and the crest level can be considered as design variables to be estimated by an optimization procedure. Other variables describing the geometry of the structure and also alternative choices for the materials of the structure can be included in the procedure. A final choice between alternatives can be based on a comparison of the properties of the resulting optimal design within each structural concept. Other failure mechanisms, such as geotechnical instability and excessive settlement or other limit states, such as Fatigue Limit States, can also be incorporated in the analysis to improve the calculations and provide more accurate results.

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6 REFERENCES


