MSc THESIS

Probabilistic design of breakwaters in shallow, hurricane-prone areas

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PREFACE

This report is an overview of the study conducted in the framework of my thesis for the title of Master of Science in Hydraulic Engineering of Delft University of Technology. The project has been supported by ARCADIS.

The topic has been chosen from the field of Hydraulic Structures and Probabilistic Design, and is the probabilistic design of breakwaters if shallow, hurricane-prone areas. The main objective is a critical assessment of the methods used for the design of an armour layer and the introduction of a new design approach.

I would like to thank all members of my graduation committee for their thoughtful supervision and critical comments throughout realisation of this work and especially ir. H.J. Verhagen for his essential assistance in all stages of this project. Then I would like to express my heartfelt thanks to ir. W. Kanning for his extensive supervision and moral support in the final project stages, as well as dr. ir. H.G. Voortman from ARCADIS who has introduced me to a more professional thinking. Furthermore I would like to thank ir. K. den Heijer for introducing me to MATLAB and Open Earth network, and dr. ir. P.J.H.J.M. van Gelder for his scientific advice on probabilistic calculations.

V. Tsimopoulou
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ABSTRACT

One of the failure mechanisms of a rubble mound breakwater is the failure of its armour layer. In order to determine the stability of an armour layer, the design load has to be defined, which is in fact the wave that attacks the structure. Being a highly stochastic phenomenon, the wave action is not easily defined, while there is always some uncertainty inherent to its definition. In a deterministic calculation this uncertainty is totally overlooked, as the possible variations of the design wave height are not taken into account. In order to incorporate uncertainties into the design process, and therefore increase its reliability, probabilistic design methods should be applied. A commonly used approach is a semi-probabilistic computation on level 1, which introduces the application of partial safety coefficients, yet the indicated methods to derive and apply them do not clarify the uncertainties incorporated, adding an undefined degree of safety in the process, or end up with incorrect results under certain conditions. Another approach is a fully probabilistic computation on level 2 or 3. This type of design tackles explicitly a great deal of uncertainties, hence its results can be considered much more accurate. However it is not commonly used, due to the fact that there are not straight forward guidelines to support it, and therefore a number of critical decisions by the designers are required.

The main objective of this study is to indicate the weaknesses of the existing design methods, and to suggest a design approach that is both attractive to designers and sufficiently reliable. This is achieved through application of the existing methods in an example case, whose features facilitates a critical assessment, and enables formulation of an improved approach. The chosen case is the jetties at the entrance of Galveston port, in the Gulf of Mexico, and the features of interest are the hurricane-dominated hydraulic climate and the fact that the structure is located in shallow water, meaning that the design load is determined by depth-limited waves. The design methods that are demonstrated are a classical deterministic design, a semi-probabilistic calculation on level 1 as proposed by PIANC in 1992, and a fully probabilistic calculation on level 3 with a Monte Carlo simulation. Based on the evaluation of the three design processes and the results, the new approach can be developed, which suggests a rational framework for deriving safety factors. According to it, a set of safety factors is generated which incorporate the same uncertainties as a fully probabilistic design; hence an equally reliable result is extracted.

The final product is a guideline for code makers indicating the procedure to derive the safety factors and a guideline for future designers indicating the analytic steps for a proper use of the safety factors. In addition a large number of concluding remarks are summarized, which can contribute in optimizing the performed analysis. The concluding remarks refer in particular to the determination of hydraulic boundary conditions, the application of the design methods, the probabilistic model used for Monte Carlo simulation, the proposed design approach, and the safety factors derived with this approach.

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1. INTRODUCTION

1.1 Problem definition

One of the failure mechanisms of a rubble mound breakwater is the failure of its armour layer. In order to determine the stability of an armour layer, the design load has to be defined, which is in fact the wave that attacks the structure. Being a highly stochastic phenomenon, the wave action is not easily defined, while there is always some uncertainty inherent to its definition. In a deterministic calculation this uncertainty is totally overlooked, as the possible variations of the design wave height are not taken into account. In order to incorporate uncertainties into the design process, and therefore increase its reliability, probabilistic design methods should be applied.

A commonly used approach is a semi-probabilistic computation on level 1, which introduces the application of partial safety coefficients in the stability formula. A standard procedure followed by the majority of designers is to apply a constant safety factor on the load. If for example a breakwater with a lifetime of 100 years needs to be designed, a design wave that occurs with a probability 1/100 is considered. In the end, for safety reasons, this value is about 30% increased, and therefore the final wave height that is used in all tests and calculations is about 130% of the wave with probability 1/100. This standard practice, although used by the majority of designers, adds an undefined degree of safety in the design process. For a certain area with certain wave data a 30% increase of the wave height may decrease the probability of failure 10 times, meaning that a probability 1/100 can become 1/1000. In cases that the standard deviation of the wave height decreases though, meaning that exceedance curve becomes steeper, the decrease of the failure probability can turn to be minor and might not satisfy the target safety level. This problem is depicted in the figure 1.1, while an indication of the corresponding probability density functions is shown in figure 1.2.

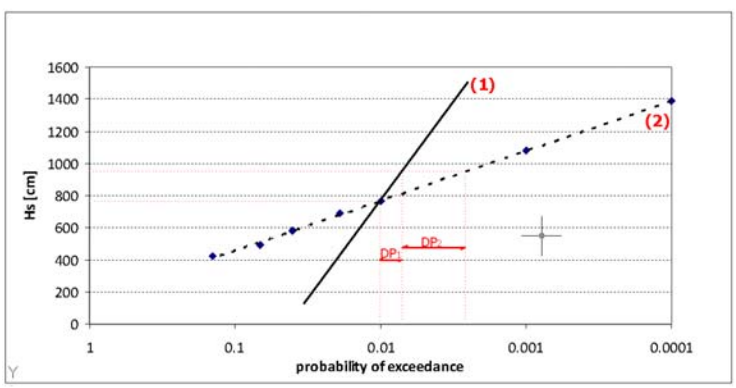

Figure 1.1: Decrease of exceedance probability in different curves

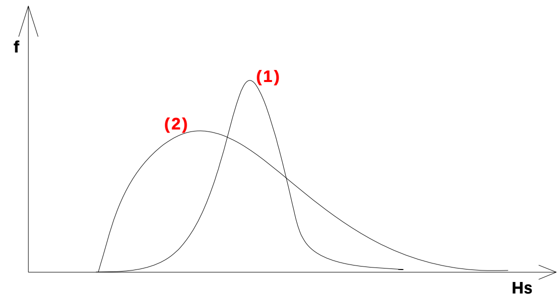

Figure 1.2: Corresponding probability density functions

An attempt to resolve the problem of undefined safety has been made by PIANC in 1992 with another semi-probabilistic method, which suggests the use of two partial safety coefficients, one for strength and one for load. This method has an important disadvantage though, which is the fact that it takes into account an infinite correspondence between the decrease of the exceedance probability and the increase of the wave height, as shown in figure 1.3.

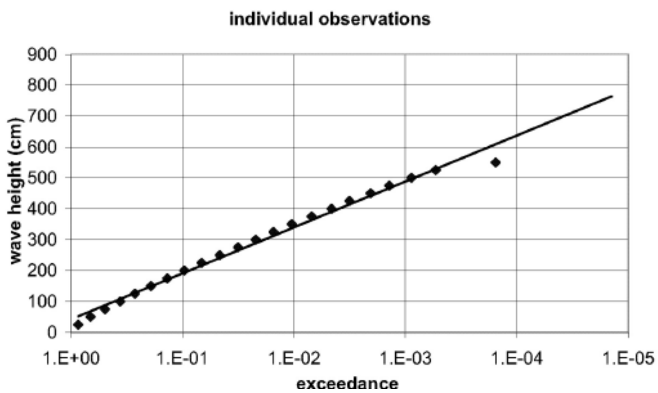

Figure 1.3: Example of exceedance graph

This graph is correct in deep water conditions, but not in shallow water where wave breaking takes place. The process of breaking dictates a threshold in the infinite increase of the wave height, which cannot get higher than a certain value. The exceedance graph then is the following.

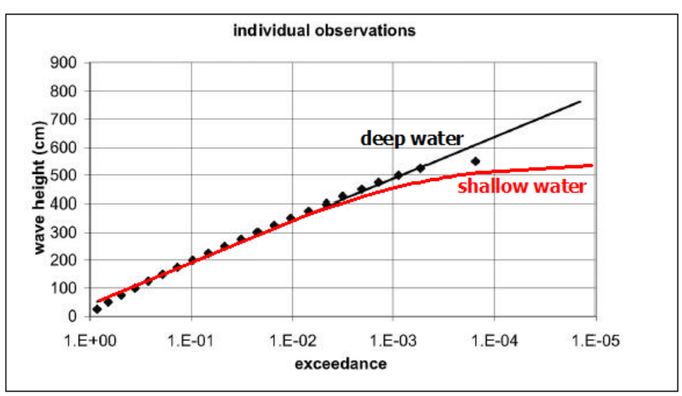

Figure 1.4: Exceedance graph for deep and shallow water

According to the shallow water curve and for wave height values that are close to the threshold, a minor increase in the design wave height can lead to a much lower probability of failure, while according to the deep water curve the achievement of the same failure probability demands the consideration of a much higher design wave height. As a consequence the method of PIANC leads to incorrect results when applied in areas where the design load is determined by depth-limited waves.

Another design approach is a fully probabilistic computation on level 2 or 3. This type of design tackles explicitly a great deal of uncertainties, hence its results can be considered much more accurate than the previously mentioned methods. Nevertheless it is not commonly used, due to the fact that there are not straight forward guidelines to support it. As a consequence a number of critical decisions are required by the designers, which makes it less attractive for use.

1.2 Project objective

According to the above it becomes clear that in practice the majority of the applied methods do not give fully acceptable results, while the most reliable solution, which is a fully probabilistic design is not broadly used. It is therefore interesting to search for a practical solution that will lead to more reliable results than the previously mentioned quasi-probabilistic approaches. This solution can be either a new method or a modification of the existing methods.

The core objective of this project can be summarized as follows:
1. Indication of the weaknesses of the existing methods for armour layer design of rubble mound breakwaters,
2. Suggestion of a new design approach that tackles the indicated weaknesses in a satisfactory way.

For demonstration to users, it is necessary to show the differences and similarities of all the previously mentioned methods in an elaborated example. The new solution can be developed through a prototype application in the example case. The chosen case needs to concentrate some particular features which will facilitate a critical evaluation of the design methods, and enables formulation of an improved design approach. An interesting case for demonstration can be found in the Gulf of Mexico, where extreme weather conditions such as a hurricane are very likely to occur, while the coastal waters are relatively shallow. The case that is proposed is the jetties at the entrance of Galveston Bay, which is a large estuary located along the upper coast of Texas (figure 1.5). The main function of the structures is to stop siltation at the entrance of the estuary, but they must also be able to resist the occurring waves. They are part of the network of structures that protect not only the port but also the vital industrial shipping facilities in and around Galveston Bay, where a significant amount of the America’s oil and chemicals are produced.

The maximum depth along the structures occurs at their toe and it is about 9m. Their total length is about 8000m and they are armoured with rock.

Figure 1.5: Location of Galveston Jetties

1.3 Report overview

This study consists of three parts, which are presented in the following chapters. The first part is a design-oriented description of the hydraulic climate in Galveston, and it is presented in Chapter 2. In Chapter 3 the design of Galveston jetties is elaborated with three different approaches, a classical deterministic, a quasi-probabilistic as indicated by PIANC in 1992, and a fully probabilistic with a Monte Carlo simulation. This chapter ends with a critical evaluation of the design results. The last part of the analysis is a rationalisation of the proposed design approach, and it is presented in Chapter 4. Chapter 5 contains a summary of the conclusions of this analysis, while in Chapter 6 some recommendations for further research are presented. The report ends with the list of references.

2. HYDRAULIC CLIMATE OF GALVESTON

2.1 Conceptual framework of hydraulic climate

The hydraulic processes that take place along the coasts of the Gulf of Mexico are affected in a high rate by the occurrence of hurricanes. Depending on the bathymetry, hurricanes contribute in the occurrence of extreme storm surges and waves, which, in combination with other unfavourable conditions, can determine the design conditions in a particular area. The general hydraulic climate can be described with the following conceptual framework of hydraulic processes.

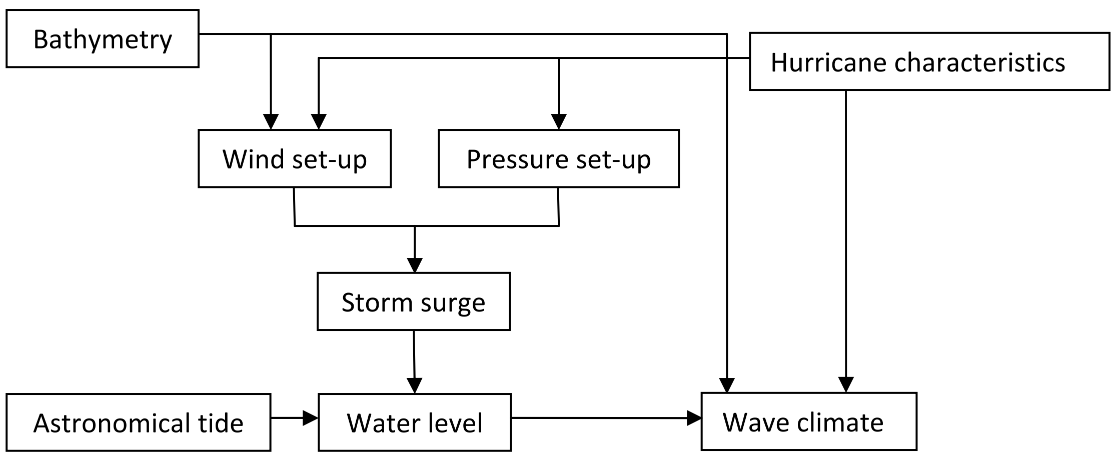

Figure 2.1: Conceptual framework for the description of hydraulic climate (Maaskant, Van Vuren, Kallen, 2009)

Based on this framework, the design conditions can be derived, and in particular the design wave height and water level. For both parameters the bathymetry and the characteristics of the passing hurricanes need to be known, and therefore they are defined first.

2.2 Bathymetry

The main source for defining the bathymetry at the project site is the bathymetric maps of TU Delft library (figure 2.2). Since those maps are not sufficiently detailed, some extra water depth information was acquired by Google Earth. This information refers to particular spots of interest for the design, such as spots along the existing jetties, and spots in the channel between the jetties.

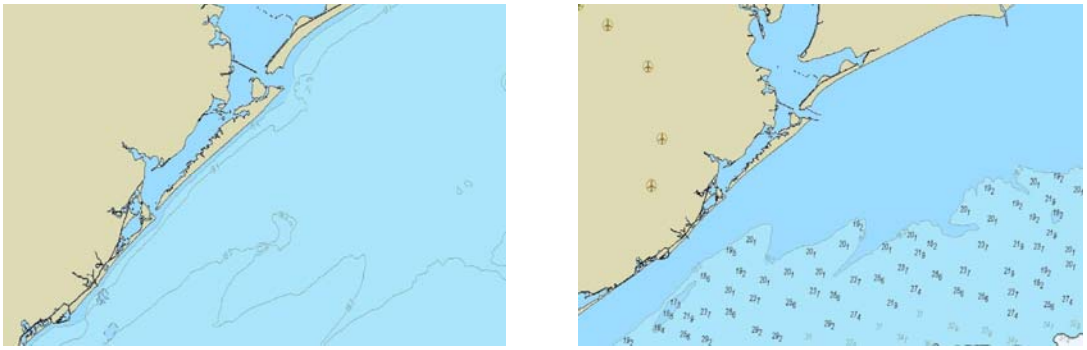

Figure 2.2: Bathymetric maps of the design site (copyright Chartworx Holland 1993-2005)

In order to come up with a typical bed profile which can be used in the design, a digital map of the area has been created in AutoCAD with bottom contour lines that approximate the information given in the above maps. The created typical bottom section has an average slope equal to $i=2.53 \cdot 10^{-4}$. This profile is used in later steps, and particularly in the definition of shallow water wave heights, which can be used directly in the breakwater design. A sketch of the bottom section, as created by SwanOne software, can be seen in the following figure.

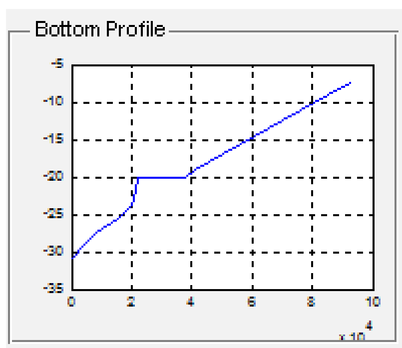

Figure 2.3: Typical bottom profile (SwanOne graph)

2.3 Hurricane characteristics

2.3.1 Hurricane records at Galveston

Galveston is an area with high frequency of passing hurricanes. Since 1850, the hurricanes crossing this area have been being recorded. The records contain information about the starting point of each hurricane, its intensity and its direction. The totality of recorded hurricane tracks in Galveston from 1850 through 2009 can be seen in figure 2.4.

Figure 2.4: Recorded hurricane tracks in Galveston (NOAA)

2.3.2 Return period

Based on these records, the National Oceanic and Atmospheric Administration of the United States (NOAA) have derived return periods of hurricanes which have a landfall in certain areas along the Gulf of Mexico, including Galveston. Those return periods are depicted in figure 2.5 for the various hurricane categories. The classification of hurricanes is made according to the Saffir-Simpson scale, which classifies the hurricanes with respect to the occurring wind speeds and the level of damage in the attacked regions.

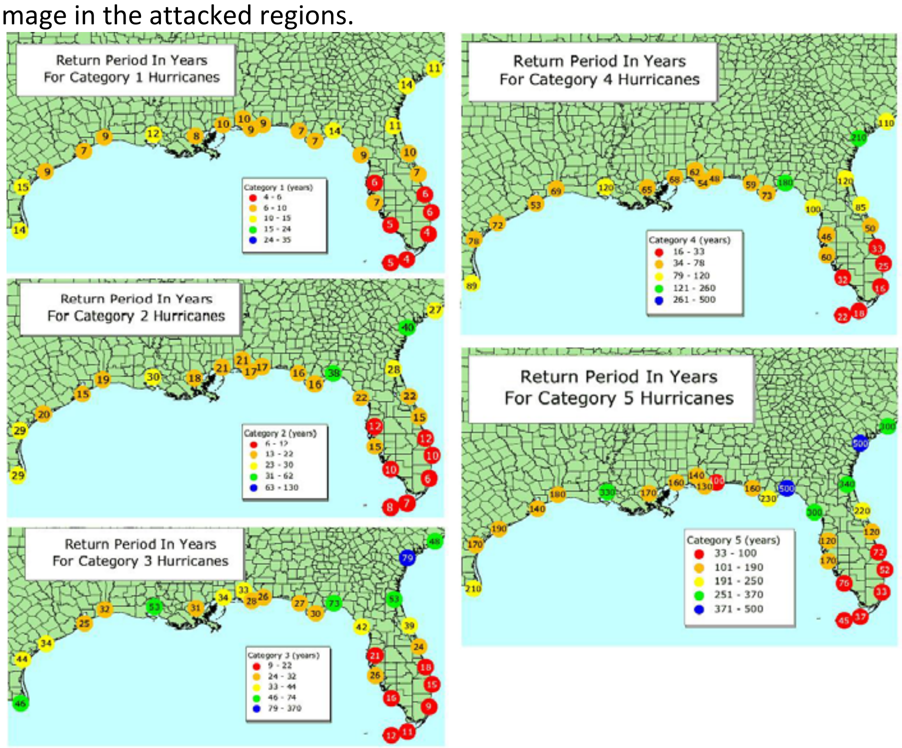

Figure 2.5: Return periods of hurricanes along the Gulf of Mexico (NOAA)

The available data allow for the creation of hurricane exceedance curves, which can be used in the design of marine structures. Due to the stochastic character of a hurricane landfall and its dependency on many parameters, the reliability of return periods determined through a pure statistical analysis of 160-year records is questionable. Another reason for this is the fact that the frequency of tropical storms and hurricanes have been increased the last three decades, which is not taken into account in this kind of analysis. A more reliable result could be derived by the use of a simulation model in combination with probabilistic methods. Such a simulation is not going to be performed, since it is out of the scope of this project. Instead, the assumption is made that the return periods of NOAA are reliable enough, and therefore they are extrapolated and used in the design of Galveston jetties.

The information given in the hurricane classification which is relevant with the design is the occurring wind speed. Another parameter that is relevant but is not included in the Saffir-Simpson scale is the minimum pressure in the eye of the hurricane. According to the hurricane records, there is a correlation between the eye pressure and the wind speed, which is indicated in the following graph by NOAA.

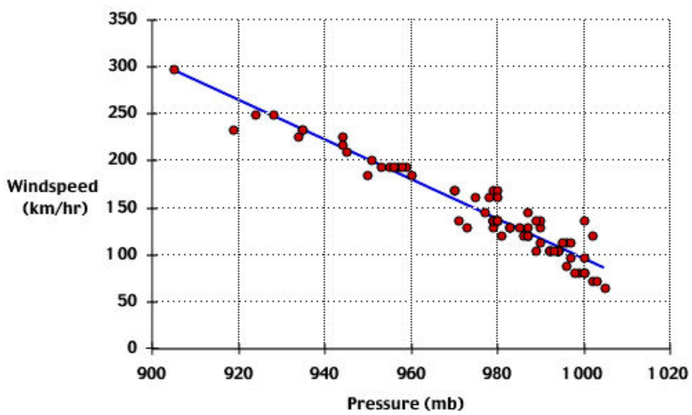

Figure 2.6: Correlation between hurricane speed and eye pressure (NOAA)

The wind speed values that correspond to the five hurricane categories, together with the respective eye pressure values are presented in the following table.

Category	Wind speed u_{10} (m/s)	Eye pressure (mb)
1	33-42	>980
2	43-49	979-965
3	50-58	964-945
4	59-69	944-920
5	>70	<920

Table 2.1: Hurricane categories (Saffir-Simpson scale)

Being a damage-based scale, Saffir-Simpson scale indicates only the minimum wind speeds that can cause a certain degree of damage, while no indication is included for the maximum physically possible wind speed. The maximum possible wind speed can be considered as an upper limit value of the most severe hurricane class, which is category 5 of Saffir-Simpson scale. This upper limit is assumed to be 20% higher than the highest ever recorded hurricane speed. The highest recorded speed is the one of Hurricane Camille in 1969 that reached 94m/s. Thus the maximum physically possible hurricane speed is supposed to be:

$$u_{\max} = 94 + 0.2 \cdot 94 = 113 m/s$$

According to the return periods of figure 2.5, and assuming that all hurricanes of a certain class have the most unfavourable wind speed and eye pressure of table 2.1, the following exceedance curves have been derived. Particularly for the wind speed exceedance curve, the above assumed upper limit is also taken into consideration. The final exceedance line is the red one of figure 2.7, while the dotted black line represents the mathematically derived curve.

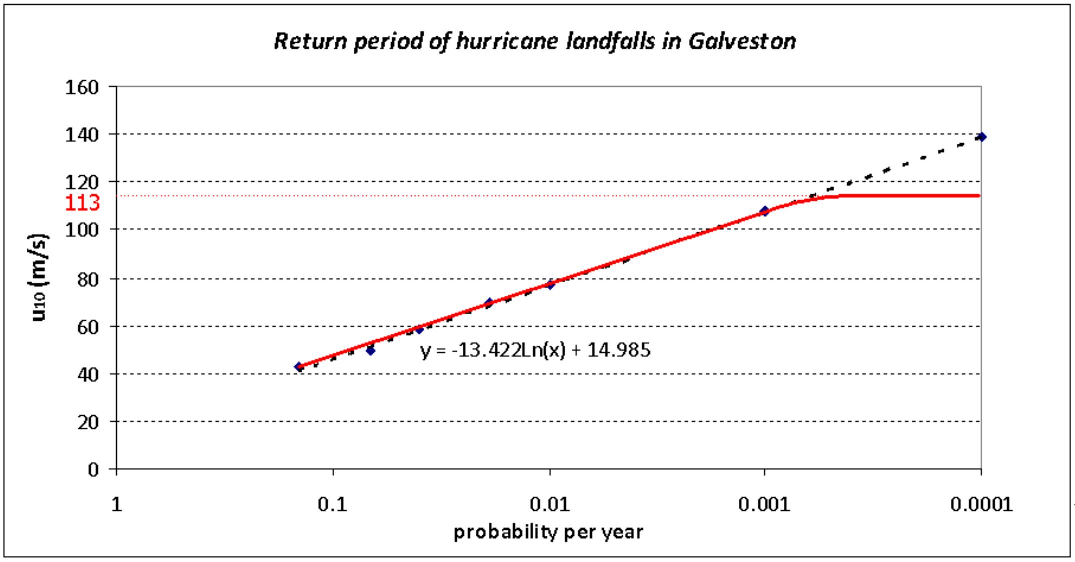

Figure 2.7: Exceedance curve of wind speed

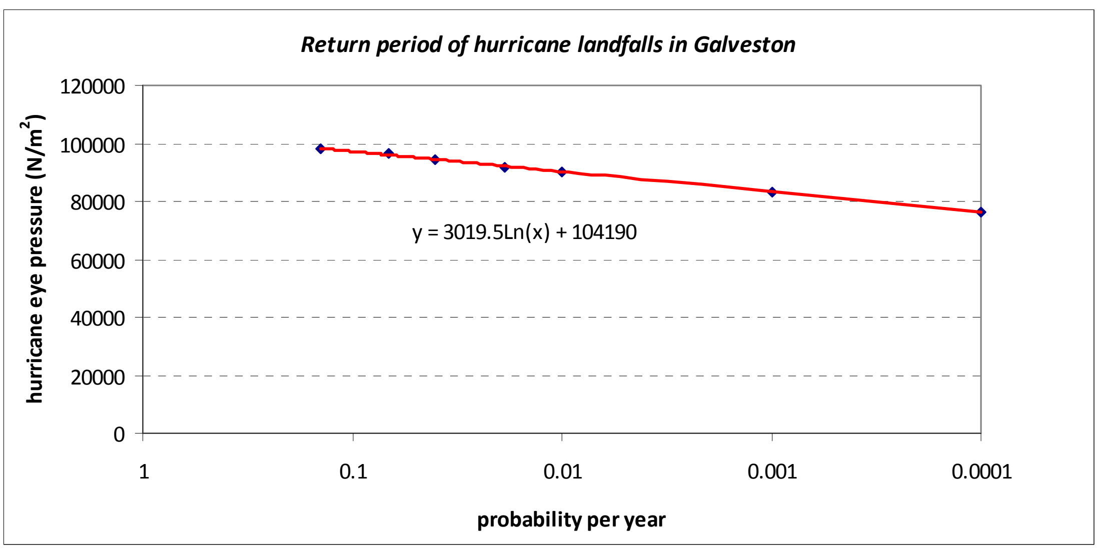

Figure 2.8: Exceedance curve of hurricane eye pressure

The above fitted curves determine the design conditions when different failure probabilities are considered. The design conditions for some standard failure probabilities can be seen in table 2.2.

Return period (yrs)	u_{10} (m/s)	p_0 (kN/m^2)
10	45.89	97
100	76.80	90
225	87.68	88
1000	107.70	83
10000	113.00	76

Table 2.2: Hurricane design conditions for standard probabilities per year

At this point it is important to note that the wind speed values indicated in the Saffir-Simpson scale, and consequently also the design values are the one-minute-sustained wind speeds. Although this wind speed is a good measure when material damage of conventional earth structures needs to be quantified, which is the case for Saffir-Simpson scale, it is not really representative of the design load on a marine structure. The one-minute-sustained wind speed is an instant extreme value and cannot be used for deriving a significant wave height or storm surge level at the site of interest. This would only be possible if the wind speed during a hurricane pass was a stationary process, meaning that the instant extreme values would be closer to average values of longer periods. In any other case a wind speed averaged over a longer period would give more realistic values for the wave height and storm surge, and certainly less conservative (figure 2.9).

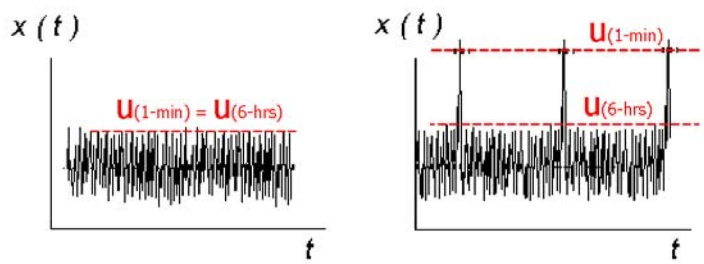

Figure 2.9: One-minute and six-hours-sustained wind speeds in a stationary (left) and non-stationary (right) process

Since it is too simplistic to assume that the wind speed during a hurricane pass is stationary, the consideration of a three or six-hour-sustained wind speed would be more correct than a one-minute-sustained speed. For this reason it is expected that the final design will be more conservative than in real design practice.

2.3.3 Hurricane tracks

The available hurricane records reveal that Galveston experiences hurricanes of various directions (figure 2.4), which cannot be easily classified. It is therefore difficult to come up with any conclusions about the most dominant tracks in this particular area. What is though feasible, and has been already accomplished by NOAA, is the determination of the typical hurricane tracks in the whole Gulf of Mexico. Those tracks can be seen in the figure below.

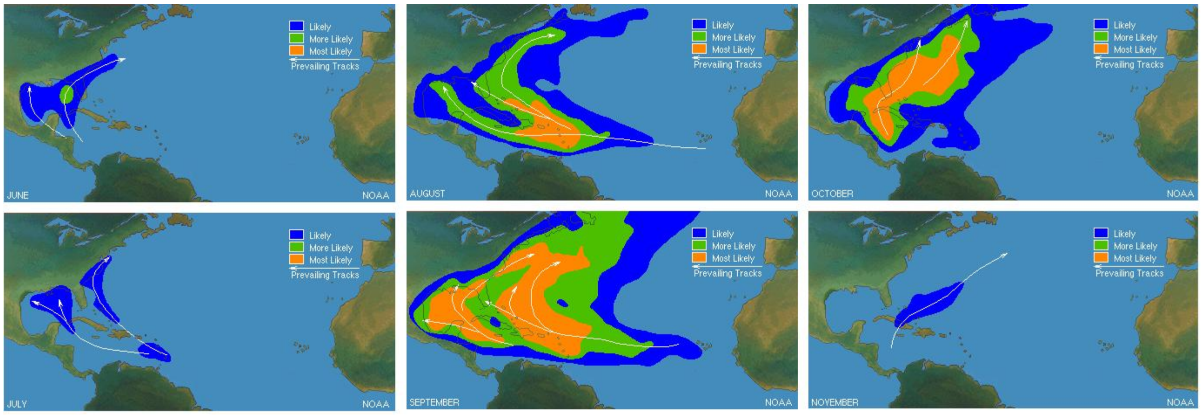

Figure 2.10: Typical hurricane tracks per month (NOAA)

Based on the tracks, it is safe to assume that the dominant directions of hurricanes striking Galveston are S, SE and E. However this information is not as relevant to the design process as the hurricane direction that causes the most unfavourable design conditions at the project site. The design boundary condition associated with the hurricane track is the maximum water level. This parameter is affected by the wind set-up and pressure set-up, whose values are related to the hurricane track (see figure 2.1). The design hurricane track could be safely extracted with a simulation model and the use of probabilistic methods. In accordance to the determination of hurricane return periods, the simulation is skipped, and an assumption is made instead. It is assumed that the most unfavourable conditions occur when the hurricane comes from SE, and particularly when it attacks in direction perpendicular to the coastline. The coastline has an angle $\alpha=52^o$ with respect to the coast, and therefore the angle of the design hurricane track is assumed equal to $52^o+90^o=142^o$.

2.4 Wave climate

The wave climate at the area of interest is determined by normal storm waves and waves induced by hurricanes. Both types of waves are examined. The design conditions are based on the most unfavourable values of the two analyses.

2.4.1 Storm waves

2.4.1.1 Significant wave height

The determination of significant wave heights of storm waves is based on two series of data, Global Wave Statistics and Argoss data. These two sources refer to different kinds of data. The Global Wave Statistics are visual observations collected by ships at given times but random locations. The Argoss data are satellite observations. Another difference is that Global Wave Statistics refer to a much larger area than Argoss. While the first refers to the whole Gulf of Mexico, the latter refers only to an area 500x500 kilometres. The features of each of the two sets are considered to impose the same degree of uncertainty in the further steps of analysis, and therefore they are considered equally reliable. Furthermore both of the sources contain completely uncorrelated data, which means that for both sets the same analysis can be performed. For this reason the performed statistical analysis is based on average values of the two sources.

The available data are both annual and seasonal. The seasonal data refer to periods of three months (March-May, June-August, September-November, and December-February). In order more reliable results to be extracted, the statistical analysis is performed five times, once for the annual records, and once for every set of seasonal records. In the end, the most unfavourable wave conditions are chosen for the design. There is also directional information contained. Since not all directions are relevant for the design, only the ones that attack the structure are selected. Since the orientation of the jetties is NW-SE, the relevant directions are N-NE-E and SE-S-SW for the north and south jetty respectively. For the statistical analysis, an extreme value distribution is used, since it gives the most reliable results. The present analysis is making use of a Gumbel distribution, and assumes storm duration equal to 12 hours.

The extract of the analysis is storm exceedance curves for the north and south jetty separately, and for every set of data (annual and seasonal). For both of the jetties, the most unfavourable wave heights come from the winter data (December-February). The corresponding exceedance curves can be seen in figures 2.11 and 2.12, while a summary of the analysis is presented in table 2.3.

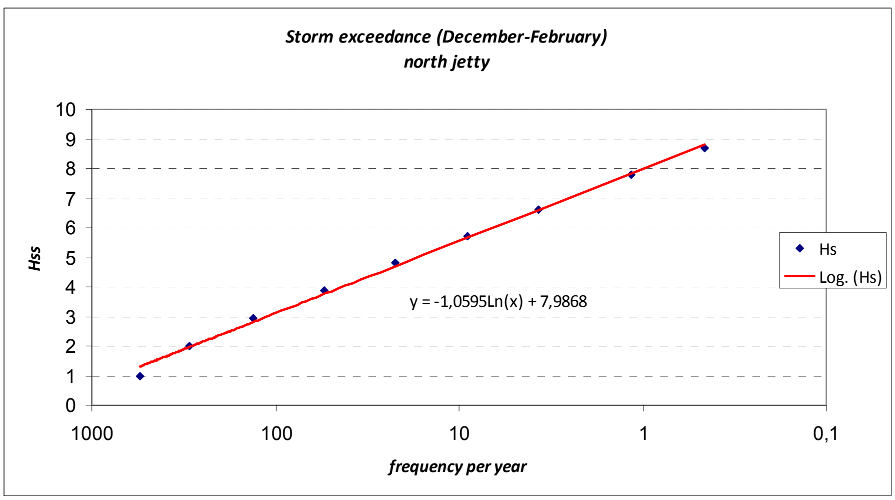

Figure 2.11: Storm exceedance curve for north jetty (based on GWS and Argoss observations)

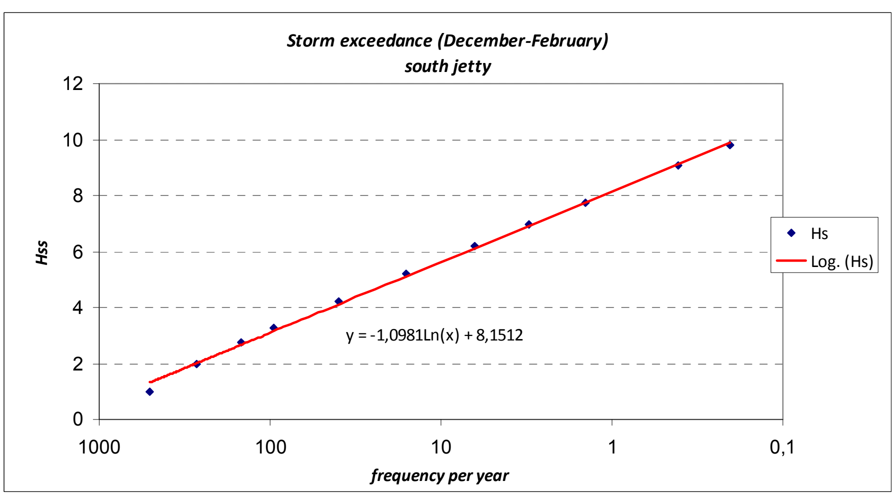

Figure 2.12: Storm exceedance curve for south jetty (based on GWS and Argoss observations

	Frequency per year	0,1	0,01	0,004	0,001	0,0001
North jetty	Annual	8,91	11,01	11,75	13,12	15,22
	March-May	8,43	10,39	11,08	12,35	14,32
	June-August	6,16	7,62	8,13	9,08	10,54
	September-November	9,08	11,21	11,95	13,33	15,45
	December-February	10,43	12,87	13,73	15,31	17,75
South jetty	Annual	9,59	11,89	12,70	14,19	16,48
	March-May	7,97	9,84	10,50	11,71	13,58
	June-August	6,70	8,30	8,86	9,89	11,48
	September-November	8,86	10,95	11,69	13,04	15,13
	December-February	10,68	13,21	14,10	15,74	18,27

Table 2.3: Summary of wave height statistical analysis

2.4.1.3 Wave period

Except for the significant wave height, the mean wave period has to be determined. The information about wave period is based on Argoss data.

The wave period is determined by both short waves and swells. Although the Gulf of Mexico is a closed sea, neglecting the action of swells can lead to significant errors in the design process. The longest fetch of a wave reaching Galveston is shown in the figure below, and equals to 2900 kilometres.

Figure 2.13: Longest possible fetch of waves at Galveston (Google Earth picture)

This fetch is not realistic because diffraction occurring in the narrow pass between Mexico and Cuba obstructs the development of wave heights along this line. The wave period though can be affected by storms occurring in the Caribbean Sea, and this can be confirmed by the wave data of Argoss. The two-dimensional scatter table of Argoss that correlates the percentage of occurrence of wave heights (rows) with mean wave periods (columns) for short waves is shown in table 2.4. The corresponding table that correlates both short and long waves with significant wave heights is shown in table 5. It is obvious that in the set of mixed data, there is a range of period values that does not follow the trendline of short waves. This range of period values is found on the right side of the trendline, and represents swells. The action of swells imports a considerable amount of energy in the wave spectrum which in many cases can reach 50% of the energy of short waves.

Percentage of occurrence of height of wind sea (m) in rows versus mean period of wind sea (s) in columns

	lower	01	02	03	04	05	06	07	08	09	10	11	12	13	14	
lower	upper	02	03	04	05	06	07	08	09	10	11	12	13	14	15	total
00	01	0	6.5	12.4	6.6	1.9	0.1	0	0	0	0	0	0	0	0	27.6
01	02	0	0	0	10.6	12.5	0.7	0	0	0	0	0	0	0	0	23.8
02	03	0	0	0	0	2.8	16.5	2.3	0	0	0	0	0	0	0	21.7
03	04	0	0	0	0	0	1.6	9.2	1.5	0.0	0	0	0	0	0	12.3
04	05	0	0	0	0	0	0	0.3	4.7	0.7	0	0	0	0	0	5.8
05	06	0	0	0	0	0	0	0	0.4	3.3	0.1	0	0	0	0	3.9
06	07	0	0	0	0	0	0	0	0	1.4	1.5	0.0	0	0	0	2.9
07	08	0	0	0	0	0	0	0	0	0	0.7	0.4	0	0	0	1.1
08	09	0	0	0	0	0	0	0	0	0	0.1	0.4	0.0	0	0	0.6
09	10	0	0	0	0	0	0	0	0	0	0	0.1	0.2	0	0	0.3
10	11	0	0	0	0	0	0	0	0	0	0	0	0.1	0	0	0.1
11	12	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0	0.0
12	13	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0
13	14	0	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.0
14	15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
total		0.0	6.5	12.4	17.2	17.2	18.9	11.9	6.6	5.4	2.5	0.9	0.4	0.0	0.0	100.0

Table 2.4: Argoss scatter table for mean wave period of short waves (Hs in rows/Tm in columns)

Percentage of occurrence of wave height (m) in rows versus mean wave period (s) in columns

	lower	03	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	
lower	upper	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	total
00	01	0	0.1	0.5	1.2	0.3	0	0	0	0	0	0	0	0	0	0	0	2.1
01	02	0	0.0	1.5	5.5	6.0	3.2	0.7	0.1	0.0	0	0.0	0	0	0	0	0	17.0
02	03	0	0	0.0	4.2	7.0	6.8	4.9	1.6	0.3	0.0	0	0	0	0	0	0	24.9
03	04	0	0	0	0.1	4.7	7.1	4.8	3.5	1.3	0.2	0	0	0	0	0	0	21.6
04	05	0	0	0	0.0	0.2	3.8	4.5	2.8	1.8	0.5	0.1	0	0.0	0.0	0	0	13.7
05	06	0	0	0	0	0	0.3	3.3	2.7	1.6	0.7	0.2	0.0	0	0	0.0	0	8.8
06	07	0	0	0	0	0	0	0.8	2.4	1.3	0.8	0.1	0	0	0	0	0	5.4
07	08	0	0	0	0	0	0	0.0	0.9	1.5	0.9	0.1	0.0	0	0	0	0	3.5
08	09	0	0	0	0	0	0	0	0.1	0.7	0.5	0.2	0.1	0	0	0	0	1.7
09	10	0	0	0	0	0	0	0	0.0	0.1	0.4	0.0	0.0	0	0	0	0	0.6
10	11	0	0	0	0	0	0	0	0	0.0	0.2	0.1	0.0	0	0	0	0	0.5
11	12	0	0	0	0	0	0	0	0	0	0	0.0	0.0	0	0	0	0	0.1
12	13	0	0	0	0	0	0	0	0	0	0	0.0	0.0	0	0	0	0	0.1
13	14	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0	0	0	0.0
14	15	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
total		0.0	0.1	2.0	10.9	18.2	21.1	19.1	14.1	8.7	4.3	1.1	0.3	0.0	0.0	0.0	0.0	100.0

Table 2.5: Argoss scatter table for mean wave period of total waves (Hs in rows/Tm in columns)

The combined action of short and long waves is described by double-peak spectra, like the one presented in the figure below. When this type of wave fields exists, the peak period can be more clearly recognised in the spectrum than the mean period, because the latter is very much dependent on the shape of the spectrum which is rather irregular. For this reason the use of peak period is usually preferable for the description of hydraulic boundaries. Nevertheless in the examined case the mean periods are also used for the description of the hydraulic climate, since their correlation with the significant wave height is indicated in the tables of Argoss, which are considered sufficiently reliable.

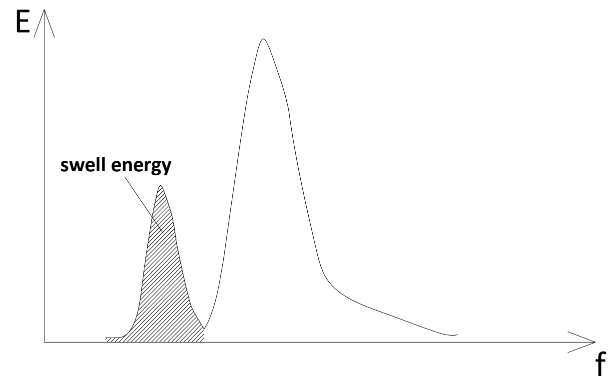

Figure 2.14: Spectrum for combined action of short waves and swells

Short waves
The correlation of mean wave periods and significant wave heights of short waves that is shown in table 2.4 indicates that there is a dependency of the two parameters, which can be expressed with a trendline. By inserting the most frequent period values for a certain significant wave height in a chart, the optimum fitted curve can be estimated, and based on this the mean periods of the most unfavourable wave heights can be calculated. The optimum fitted curve is presented in the following chart.

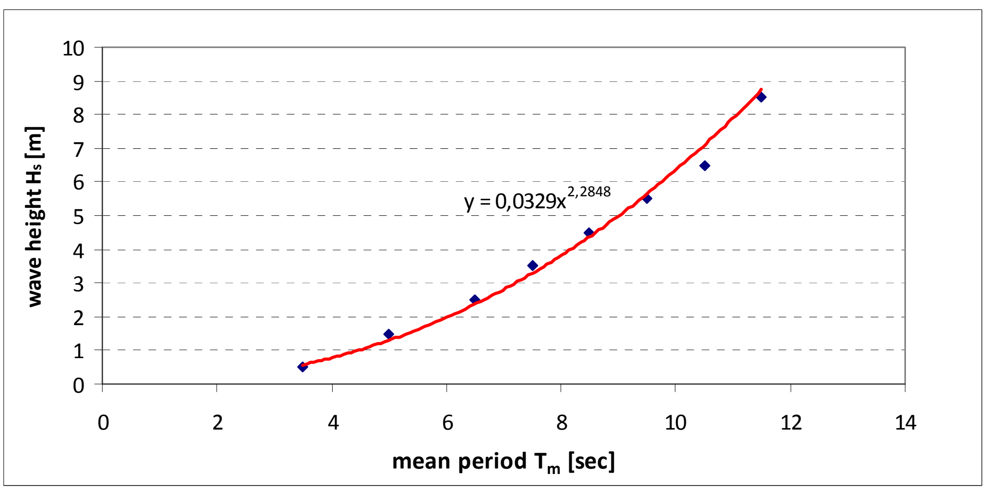

Figure 2.15: Correlation of mean period and significant wave height of short waves

The validity of the wave periods that are calculated with the above chart can be checked with the wave steepness, which should take values in the range 3-5%. The wave steepness is calculated with the following formula:

$$s_o = \frac{2\pi H_s}{g T_m^2} \tag{2.1}$$

A summary of the resulted values of mean period and corresponding wave steepness for the worst occurring significant wave heights is presented in the table below. For all values the wave steepness is in the range 3-5%, and therefore acceptable.

	Prob. per year	H_s (m)	T_m (s)	s_o
north jetty	0,1	10,43	12,44	0,043
	0,01	12,87	13,63	0,044
	0,004	13,73	14,03	0,045
	0,001	15,31	14,71	0,045
	0,0001	17,75	15,69	0,046
south jetty	0,1	10,68	12,57	0,043
	0,01	13,21	13,79	0,044
	0,004	14,1	14,19	0,045
	0,001	15,74	14,89	0,045
	0,0001	18,27	15,89	0,046

Table 2.6: Mean wave period and wave steepness for maximum significant wave heights

Swells

The tables with swell data provided by Argoss, reveal that for swells there is no correlation between wave height and period like for short waves, since the data are scattered in a way that no curve can be fitted (table 2.7). It is therefore clear that the wave steepness criterion cannot be used in this case.

According to table 2.7, a cumulative probability curve for wave height can be derived. The curve is shown in figure 2.16. The design wave height of swells will be assumed equal to the 90% wave height according to the curve, thus 4.5m. According to table 2.7, the most probable period for wave height of 4.5m is 13sec. These values are assumed good for all standard return periods.

Percentage of occurrence of height of swell (m) in rows versus mean period of swell (s) in columns

	lower	04	05	06	07	08	09	10	11	12	13	14	15	16	17	18	19	20	21	22	23	
lower	**upper**	**05**	**06**	**07**	**08**	**09**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**total**
00	**01**	0	0.3	1.6	1.3	1.3	1.0	1.1	0.9	0.7	0.6	0.7	0.1	0.3	0.1	0.1	0.0	0.0	0.0	0.0	0	10.4
01	**02**	0	0.0	0.9	5.4	8.6	7.4	4.5	2.0	1.0	0.4	0.3	0.1	0	0.0	0.0	0	0	0	0	0	30.8
02	**03**	0	0	0	0.1	1.3	7.4	8.6	5.9	2.8	0.9	0.5	0.2	0.1	0.0	0	0	0	0	0	0	27.7
03	**04**	0	0	0	0	0.0	1.2	3.5	5.7	3.5	1.8	0.7	0.4	0.1	0	0	0	0	0	0	0	16.8
04	**05**	0	0	0	0	0	0.2	0.9	1.3	3.0	1.8	1.0	0.4	0.3	0	0	0	0	0	0	0	8.9
05	**06**	0	0	0	0	0	0	0.2	0.3	0.7	1.2	0.7	0.3	0.1	0.0	0	0	0	0	0	0	3.5
06	**07**	0	0	0	0	0	0	0	0.0	0.1	0.5	0.4	0.2	0.1	0	0	0	0	0	0	0	1.3
07	**08**	0	0	0	0	0	0	0	0	0	0.0	0.2	0.1	0	0.0	0.0	0	0	0	0	0	0.4
08	**09**	0	0	0	0	0	0	0	0	0.0	0	0.0	0.1	0.0	0.0	0	0	0	0	0	0	0.2
09	**10**	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0	0	0	0	0	0	0	0.0
10	**11**	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0
total		0.0	0.3	2.5	6.8	11.3	17.2	18.8	16.1	11.8	7.3	4.3	2.0	0.9	0.3	0.2	0.0	0.0	0.0	0.0	0.0	100.0

Table 2.7: Argoss scatter table for mean wave period of swells (Hs in rows/Tm in columns)

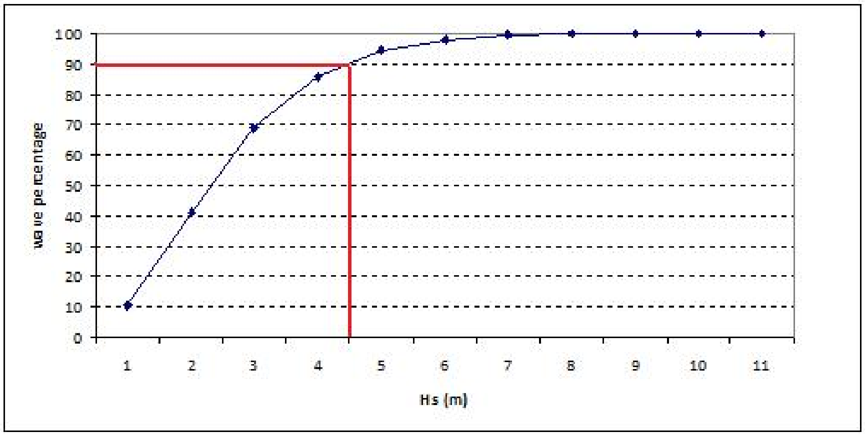

Figure 2.16: Cumulative wave height distribution of swells

2.4.2 Hurricane-induced waves

The waves caused by a hurricane can be estimated through the hurricane wind speeds u_{10}. Depending in the track of the hurricane, there can be either short waves or swells occurring. When a hurricane generates waves in the Caribbean Sea, without crossing the Gulf of Mexico, there are only swells arriving to Galveston. When it hits the Gulf, it is most probable that only short waves will arrive.

2.4.2.1 Short waves

The maximum fetch in the Gulf equals to 1295km, and is presented in the following figure.

Figure 2.17: Maximum fetch in the Gulf of Mexico (Google Earth picture)

The calculation procedure of the wave characteristics developing along this fetch is presented below.

The wind speed of a hurricane with return period of 10 years is u_{10}=45.89 m/s (see table 2.2).

Dimensionless fetch: $\tilde{F} = \frac{gF}{u_{10}^2} = \frac{9.81 \cdot 1295 \cdot 10^3}{45.89^2} \approx 6032$

Water depth (deep water): d=2500m (this is a representative value for the Gulf of Mexico)

Dimensionless water depth: $\tilde{d} = \frac{gd}{u_{10}^{2}} = \frac{9.81 \cdot 2500}{45.89^{2}} \to \infty$

From graph 8.2 of Waves in coastal and oceanic waters, L. Holthuijsen, the values for dimensionless wave height and period are extracted.
$\tilde{H} = 0.18$, $\tilde{T}_p = 5$

Significant wave height: $H = \frac{\tilde{H} \cdot u_{10}^{2}}{g} = \frac{0.18 \cdot 45.89^{2}}{9.81} = 38.64m$

Peak period: $T_p = \frac{\tilde{T} \cdot u_{10}}{g} = \frac{5 \cdot 45.89}{9.81} = 23.38 sec$

A summary of the short wave characteristics for hurricanes with some standard return periods is presented in the table below. It can be perceived that for large return periods some extreme wave features are extracted, whose possibility of existence can be questioned. An explanation for this result is the use of one-minute-sustained wind speeds in the above calculations. As mentioned before, the use of wind speeds averaged over three to six hours, which are expected to be considerably lower than the one-minute-sustained speeds, might be more appropriate for the determination of wave climate, and would certainly result in lower values of wave height and period. The sensitivity of the results on variations of wind speed is not further elaborated in this study, as the values of the table below are not really critical for the upcoming design. The reason is that these values constitute deep water conditions, while the design will take place in shallow water, where depth-limited waves determine the design conditions.

Return period (yrs)	u_{10} (m/s)	$F_{(d/less)}$	$d_{(d/less)}$	$H_{(d/less)}$	$T_{(d/less)}$	H (m)	T_p (sec)
10	45,89	6032,23	∞	0,18	5	38,64	23,39
100	76,80	2154,05	∞	0,1	3,8	60,12	29,75
225	87,68	1652,49	∞	0,085	3,4	66,61	30,39
1000	107,70	1095,20	2,114	0,09	3,6	106,42	39,52
10000	113,00	994,91	1,921	0,075	3,2	97,62	36,86

Table 2.8: Short wave characteristics for hurricanes with standard return periods

2.4.2.2 Swells

As explained before, the swells arriving from the Caribbean Sea are diffracted at the narrow pass between Cuba and Mexico. For this reason, following the same procedure like for waves generated in the Gulf for the calculation of swell heights cannot lead to correct results. This procedure can only be followed for estimating the swell periods at Galveston. Reasonable values for the wave heights at Galveston could be derived if the energy conservation principle is used. The amount of wave energy arriving at the pass of 200km between Cuba and Mexico is distributed along the coastline of 4800km of the Gulf. A reasonable correlation between the wave heights at the pass and at Galveston is therefore the following:

$$200H_{s(pass)}^{2} = 4800H_{s(Galveston)}^{2} \quad (2.2)$$

Significant wave height

In order to come up with a wave height at Galveston, the wave height at the narrow pass between Mexico and Cuba needs to be estimated first. For this estimation a fetch of 1700 kilometres is taken into account (figure 14).

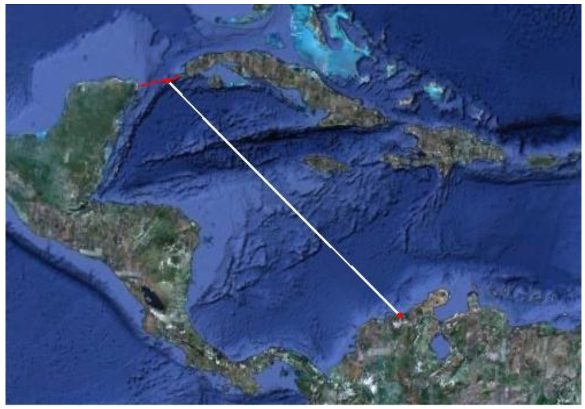

Figure 2.18: Maximum fetch of waves reaching the pass between Cuba and Mexico (Google Earth picture)

Following the same procedure as for the estimation of short waves, the following results for wave heights with standard return periods are extracted.

Return period (yrs)	u_{10} (m/s)	$F_{(d/less)}$	$d_{(d/less)}$	$H_{(d/less)}$	H (m)
10	45,89	7918,76	∞	0,17	36,50
100	76,80	2827,70	∞	0,12	72,14
225	87,68	2169,29	∞	0,1	78,37
1000	107,70	1437,71	2,114	0,07	82,77
10000	113,00	1306,05	1,921	0,07	91,11

Table 2.9: Estimated wave heights with standard return periods at the narrow pass

For the above estimation, an average water depth of 2500 meters is used. Such an approximation cannot give reasonable results if there are shoals on the sea bottom, which are not taken into account. The bathymetry of the narrow pass as given in Google Earth seems to be such a case. The average water depth at that area can be approximated to 1500 meters, which is much smaller than 2500 meters, and is considered shallow water for the wave with return period 10000 years. It is therefore clear that the wave of 117.5m will break as soon as it reaches the pass. At this stage it can be roughly assumed that after breaking, the wave height becomes equal to the wave height with 1000 years return period. The final wave heights at the narrow pass, as well as the resulting wave heights at Galveston, as calculated with equation 2.2 are presented in the table below.

Return period (yrs)	$H_{s(pass)}$ (m)	$H_{(galveston)}$ (m)
10	36.50	7.45
100	72.14	14.72
225	78.37	15.99
1000	82.77	16.89
10000	82.77	16.89

Table 2.10: Wave heights by hurricane-induced swells at the entrance of Mexico Gulf and at Galveston

Wave period

Based on the principles of wave theory, the swell peak periods arriving at Galveston can be estimated for standard return periods (table 2.11).

Return period (yrs)	u_{10} (m/s)	$F_{(d/less)}$	$d_{(d/less)}$	$T_{(d/less)}$	T_p (sec)
10	45,89	13950,99	∞	7,69	35,97
100	76,80	4981,75	∞	5	39,14
225	87,68	3821,77	∞	4	35,75
1000	107,70	2532,91	2,114	3,8	41,72
10000	113,00	2300,96	1,921	3,1	35,71

Table 2.11: Peak periods of hurricane-induced swells at Galveston

2.5 Water levels

The hurricane records at the area of Galveston indicate that there is a correlation between the occurrence of hurricanes and storm surges. This correlation was expected as already shown in the conceptual framework of hydraulic climate (figure 2.1). It is assumed that the highest water levels at Galveston occur when a hurricane pass from the area is combined with a high tide. The maximum water level at the design point can be then calculated by the following formula.

$$WL = h_d + h_t + h_p + h_w \tag{2.3}$$

where: h_d = depth below mean sea level
h_t = tidal water level
h_p = hurricane pressure set-up
h_w = hurricane wind set-up.

2.5.1 Tidal water level

The tidal effect at the area of Galveston is constantly recorded by NOAA. Two indicative record charts are presented below.

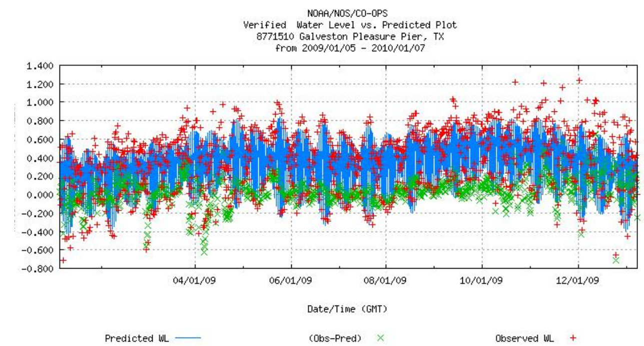

Figure 2.19: Tidal records of 2009 (NOAA)

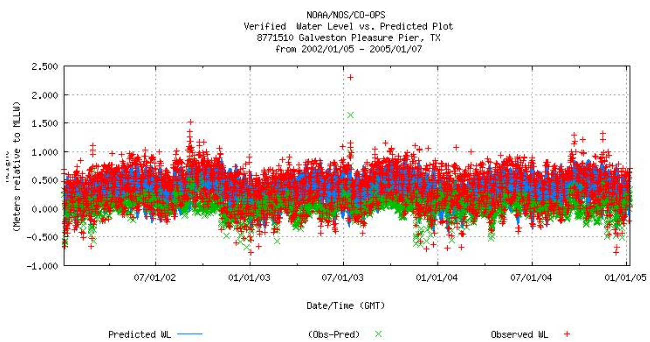

Figure 2.20: Tidal records of period 2002-2005 (NOAA)

According to the charts, a reasonable average maximum tidal level in the area is 0.85m.

2.5.2 Pressure set-up

The occurrence of a hurricane, which constitutes a low pressure weather system, is always accompanied by a pressure set-up in the surrounding water body, which can be calculated with the following formula:

$$h_p = \frac{p_n - p_\alpha}{10\gamma_w} \tag{2.4}$$

In the above equation the following variables are present:

p_n = atmospheric pressure: $p_n = 101300 N/m^2$

γ_w = specific weight of salt water: $\gamma_w = 10200 N/m^2$

p_α = pressure as a function of the distance from the hurricane eye:

$$p_\alpha(r_s) = p_n - \frac{p_n - p_o}{\sqrt{1+\left(\frac{r_s}{R}\right)^2}} \tag{2.5}$$

In equation 2.5 the following additional variables are present:

p_o = minimum pressure in the eye of the hurricane

r_s = shortest distance between the eye of the hurricane and the spot of interest

R = radius associated with the maximum wind speed (in km):

$$R = 5.4436(p_n - p_o)^{0.5034} \tag{2.6}$$

In order to come up with the maximum possible pressure set-up, the most unfavourable distance between the structure and the hurricane eye has to be determined first, together with the most unfavourable hurricane direction. The most unfavourable values are the ones that their combination maximizes the water level at Galveston. Both parameters could be safely estimated with probabilistic simulations. This analysis is skipped at this stage because it is out of the scope of this chapter. A further elaboration of the probabilistic estimation of the hydraulic load will be presented in a later stage of this study. For the time being, certain deterministic values of the hurricane parameters are assumed instead. As worst hurricane direction has already been assumed the one perpendicular to the coastline. This has an angle equal to 141^o to the north. The most unfavourable distance is assumed equal to 2000m.

Based on the assumptions the pressure set-up is calculated.

The eye pressure of a hurricane with return period of 10 years is p_o=97kN/m^2 (see table 2.2).

$$(6) \Rightarrow R = 5.4436(101.3-97)^{0.5034} = 11.34\text{km}$$

$$(5) \Rightarrow p_\alpha(2) = 101.3 - \frac{101.3-97}{\sqrt{1+\left(\frac{2}{11.34}\right)^2}} == 97.07\text{kN/m}^2$$

$$(4) \Rightarrow h_p = \frac{101.3-97.07}{10 \cdot 10.2} = 0.041m$$

2.5.3 Wind set-up

The wind set-up that is induced by a hurricane is calculated with the following equation:

$$h_w = \frac{c_1 u^2 F}{gd} \tag{2.7}$$

In the above equation the following variables are present:
c_1 = calibration parameter for the friction between wind speed and water surface. A value of $4 \cdot 10^6$ is assumed to be a good estimate, because it is in the order of magnitude of corresponding values used for determination of typhoon-induced conditions at the Vietnamese coast (*Maaskant, van Vuren, Kallen, [2009], Typhoon-induced hydraulic boundary conditions, Northern part of Vietnam Coast*).
u = wind speed along fetch F
F = circular fetch (see figure 2.21)
d = water depth along fetch
g = gravitational constant.

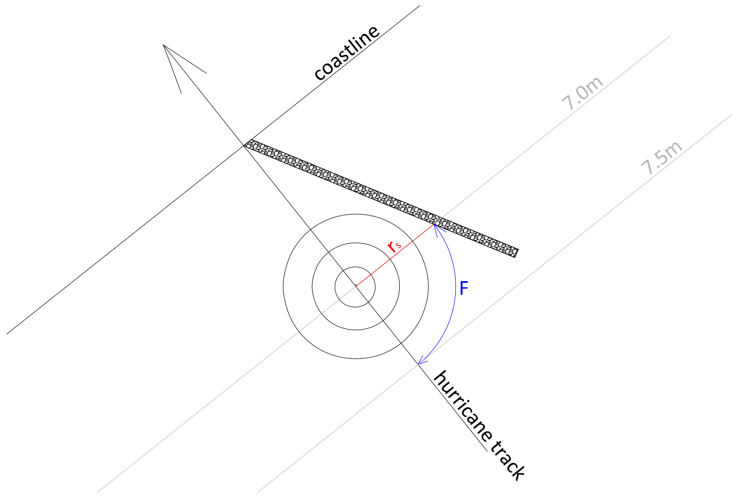

Figure 2.21: Fetch for determining the wind set-up

The maximum water level needs to be calculated at a typical cross section of the jetty, with water depth equal to 7m. The bottom slope close to structure is very small, and therefore for the most unfavourable hurricane pass (r_s=2km and α=141°) the water depth along the fetch can be considered stable, and equal to 7.3m. This simplification allows for a straight forward calculation of the wind set-up through equation 2.7. For a hurricane with return period 10 years, the wind speed is u_{10}=45.89 m/s.

fetch: $F = \frac{\pi r_s}{2} = \frac{3.14 \cdot 2000}{2} = 3140\text{m}$

$$(7) \Rightarrow h_w = \frac{4 \cdot 10^{-6} \cdot 45.89^2 \cdot 3140}{9.81 \cdot 7.3} = 0.37\text{m}$$

2.5.4 Design water levels

The results of the above calculations for the standard return periods of hurricanes together with the resulting water levels are presented in the table below. These water levels are the ones to be used for design of coastal structures in Galveston area. A different source of design conditions could be local water level observations. Although there are water level observations recorded in Galveston, they only refer to normal conditions, and there are not separate records for when hurricane conditions occur. It is therefore necessary that the hurricane water levels are calculated with the above procedure.

Return period (yrs)	u_{10} (m/s)	p_0 (kN/m^2)	R (km)	p_α (kn/m^2)	h_p (m)	h_w (m)	Water Level (m)
10	45,89	97	11,34	97,13	0,041	0,369	8,26
100	76,80	90	18,45	90,13	0,109	1,034	8,99
225	87,68	88	20,03	88,13	0,129	1,348	9,33
1000	107,70	83	23,52	83,13	0,178	2,034	10,06
10000	113,00	76	27,68	76,13	0,247	2,240	10,34

Table 2.12: Design water levels

2.6 Summary of results

The outcome of the above analysis is the boundary values in certain return periods that describe the hydraulic climate in Galveston. In particular the significant wave height and period in deep waters have been estimated for all the possible occurring wave types, as well as the maximum possible coastal water level. It should be noted that the wave characteristics derived in this chapter are not the design hydraulic boundaries for the coast of Galveston, because they constitute deep water conditions. The coastal waters of Galveston are shallow, and therefore depth-limited waves should be taken into account for a design. Yet the extracted water levels refer to coastal waters, and can be used for design purposes.

The results of the above analysis are summarised in the table below. They refer only to the north jetty for which the upcoming design will be elaborated. It is important to note that for all types of waves a correlation between the mean and peak period is considered that corresponds to a Pierson-Moskowtz spectrum (T_m=0.75T_p). This assumption for correlation of the two types of periods is necessary due to the fact that the hydraulic boundaries are derived with a different approach for normal storm waves and hurricane-induced waves. The approach for storm waves gives estimation of mean periods, while the one for hurricane waves estimates peak periods. In the end the same type of periods should be known for both wave types in order to decide for the most unfavourable conditions that will be used for the design.

Return period	Wave type	$H_{s(deep)}$ (m)	$T_{p(deep)}$ (s)	$T_{m(deep)}$ (s)	WL (m)
10	Storm short waves	10,43	16,59	12,44	7,85
	Storm swells	4,5	17,33	13	7,85
	Hurricane short waves	38,64	23,39	17,54	8,26
	Hurricane swells	7,45	35,97	26,98	8,26
100	Storm short waves	12,87	18,17	13,63	7,85
	Storm swells	4,5	17,33	13	7,85
	Hurricane short waves	60,12	29,75	22,31	8,99
	Hurricane swells	14,72	39,14	29,36	8,99
225	Storm short waves	13,73	18,71	14,03	7,85
	Storm swells	4,5	17,33	13	7,85
	Hurricane short waves	66,61	30,39	22,79	9,33
	Hurricane swells	15,99	35,75	26,81	9,33
1000	Storm short waves	15,31	19,61	14,71	7,85
	Storm swells	4,5	17,33	13	7,85
	Hurricane short waves	106,42	39,52	29,64	10,06
	Hurricane swells	16,89	41,72	31,29	10,06
10000	Storm short waves	17,75	20,92	15,69	7,85
	Storm swells	4,5	17,33	13	7,85
	Hurricane short waves	97,62	36,86	27,65	10,34
	Hurricane swells	16,89	35,71	26,78	10,34

Table 2.13: Deep water wave characteristics and coastal water levels at Galveston

3. DESIGN OF GALVESTON JETTIES

3.1 Functional requirements

The functional requirements on which the design of the jetties needs to be based are the following:

1. The structure has to stop siltation in the entrance of Galveston estuary, where also the port of Galveston is located. For this reason the jetties need to be long enough to prevent the accumulation of sediment at the port channel. It is assumed that the length of the existing jetties is sufficient, and therefore it will be kept as it is in the new design.
2. The structure has to provide safe manoeuvring at the entrance of Galveston port, which means to be able to reduce sufficiently the incoming wave height. It is assumed that safe manoeuvring can be achieved with a transmitted wave equal to 1m.

3.2 Selection of breakwater type

All breakwater types are feasible in the area of interest. There are some environmental conditions though which impose some advantages in the application of a classical rubble mound structure. The conditions are listed below.

1. The structure is located in relatively shallow water (depth at the roundhead equal to 9m). In shallow waters the construction of rubble mound structures is usually cheaper than caisson solutions.
2. The structure is located at the entrance of an estuary on very soft seabed. Some deformations are expected, which can be better received by a rubble mound structure.
3. The requirement for acceptable manoeuvring conditions introduces limitations in the wave reflection in the inner part of the jetties. A rubble mound structure is usually more favourable under this kind of limitations, since they usually meet the requirements with a lower cost than caisson structures.

3.3 Design storm and design lifetime

An economic lifetime for a breakwater is in the order of 50 years. For this reason a lifetime of 50 years is chosen. During lifetime, a probability of failure equal to 20% is assumed to be acceptable for a structure functioning as outer breakwater, like Galveston jetties. In case that the structure was meant to protect directly a harbour basin, then the target probability of failure should be lower than in case of outer breakwaters, because then the expected damage is higher, as it is connected with the port operations (figure 3.1). In reality the target probability of failure is chosen based on economic optimization and it is an issue that mostly decision-making bodies are concerned about and not designers.

The target probability of failure is Poisson distributed:

$$p = -\exp(-fT_L) \tag{3.1}$$

Where p = probability of serious damage during lifetime
T_L = breakwater lifetime
f = storm frequency.

Having the probability of failure as prerequisite, the required storm frequency can be derived from the above equation:

$$(3.1) \Rightarrow f = -\frac{1}{50}\ln(1\text{-}0.2) = 0.004 \rightarrow \text{ 1/225 per year}$$

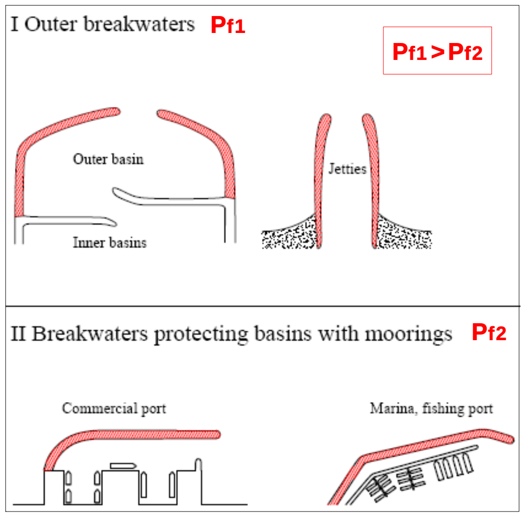

Figure 3.1: Target probabilities of failure of structures with different functions

3.4 Design conditions

3.4.1 Bed conditions

The structure is located at the entrance of an estuary. In this type of areas the seabed is usually very soft, and as a consequence some important deformations are expected, which have to be taken into account in the structural design. Since this research concerns the armour layer stability, which is irrelevant to bed stability, this issue is neglected in the upcoming analysis, by assuming that the structure is located on a steady sand bottom.

3.4.2 Available material

There are granite quarries in the area which have been used extensively in the previous century. It is assumed that they are still operational and can be used in the construction of the breakwaters. The granite density is in the range 2.5-2.8 kg/m^3 (Rock Manual 2007).

3.4.3 Hydraulic boundary conditions

The hydraulic climate of Galveston has been thoroughly presented in the previous chapter, in a firmly design oriented way. The design values that have been derived in the previous chapter can be used as hydraulic boundary conditions for a deterministic design in Galveston, they refer though to deep waters. In shallow waters the conditions change because wave breaking takes place. Hence the wave heights that are used for the design are smaller than those in deep water. The shallow water conditions can be calculated through the use of SwanOne software. A summary of the results of SwanOne that are relevant to the design are presented in the following table.

The design equation is applied for each set of wave conditions presented below. The largest resulting armour units are the ones to be chosen for the final design.

Wave type	H_{m0} (m)	$H_{2\%}$ (m)	T_p (s)	T_{m-10} (s)
Storm short waves	3,09	4,51	6,46	7,58
Storm swells	1,8	2,54	17,28	9,32
Hurricane short waves	4,3	6,26	8,96	9,69
Hurricane swells	2,69	3,88	29,87	11,11

Table 3.1: Summary of SwanOne results

3.5 Deterministic design

The classical method for designing a breakwater is application of all the common dimensioning rules with the use of deterministic values for all the parameters. The starting point is the dimensioning of the armour units. Next the determination of crest height and width takes place, and in the end the sub-layers are defined, together with the toe dimensions. Here only the dimensioning of the armour units is presented. The armour units need to be designed for the four different hydraulic conditions. In the end the heaviest resulting armour unit is going to be selected.

The armour units to be used are granite units of density ρ_s=2650kg/m^3. They will be applied in two layers, in a slope 1V:2H. The weight of the armour units can be reliably defined with application of the Van der Meer formula. This formula varies with respect to the site conditions, and particularly the steepness of the incoming wave.

The type of incoming waves is determined by the surf similarity parameter. The transition between surging and plunging waves can be calculated using the critical value for the surf similarity parameter:

$$\xi_{cr} = \left(\frac{c_{pl}}{c_s} P^{0.31} \sqrt{\tan\alpha} \right)^{\frac{1}{P+0.5}} \quad (3.2)$$

For $\xi_m<\xi_{cr}$ waves are plunging, while for $\xi_m>\xi_{cr}$ waves are surging.

In formula (2) the following parameters are present:
a = angle of the seaward slope of the structure. For slope 1V:2H⟶ tanα=0.5, cotα=2
P = notional permeability coefficients. For a structure with 1 sub-layer (filter) and core, P=0.4 (Rock Manual 2007).
c_{pl}, c_s = coefficients dependent on the wave spectrum. In the deterministic design, for deep water the values c_{pl}=5.5 and c_s=0.87 are used, while for shallow water 7.25 and 1.05.

According to the Rock Manual (2007) there is also variation in the Van der Meer formula form between deep and shallow water. It has been proven though that the form for shallow water conditions gives correct results in deep waters as well. This form is indicated in "Riprap stability for deep water, shallow water and steep foreshores" (Verhagen, Mertens, 2007) and is presented below:

In these formulae the following parameters are present:
$H_{2\%}$ = wave height exceeded by 2% of the waves
$T_{m-1,0}$ = wave period calculated from the first negative moment of the spectrum
$s_{m-1,0}$ = fictitious wave steepness calculated from the first negative moment of the spectrum: $s_{m-1,0} = 2\pi H_{2\%} / (g T_{m-1,0}^2)$
$\xi_{m-1,0}$ = surf similarity parameter based on the first negative moment of the spectrum: $\xi_{m-1,0} = \tan\alpha / \sqrt{s_{m-1,0}}$
N = number of incoming waves: N=(storm duration)/T_m
S = damage level. For slope 1V:2H and intermediate damage during lifetime, the damage level can take values in the range 4-6 (table 7-2, Breakwaters and closure dams, Verhagen, d' Angremond, van Roode). For the deterministic design the mean value is used, S=5.
Δ = relative mass density: $\Delta=(\rho_s-\rho_w)/\rho_w=(2650-1000)/1000=1.65$
d_{n50} = nominal median block diameter.

A summary of the results of application of the appropriate form of Van der Meer formula in each set of hydraulic boundary conditions is presented in table 3.2. For every set of hydraulic boundaries, a nominal median block diameter is extracted, from which the block weight can be approximated with the formula:

$$M_{50} = \rho_s \cdot d_{n50}^3 \tag{3.3}$$

The largest block size resulting from application of Van der Meer formulas indicates the design armour unit, and is marked with red in the following table.

Wave type	d_{n50} (m)	M_{50} (kg)
Storm short waves	1,20	4545
Storm swells	0,86	1686
Hurricane short waves	1,69	12855
Hurricane swells	1,27	5373

Table 3.2: Armour unit sizes for the different hydraulic boundary conditions

As shown in the table above, the critical design conditions are imposed by hurricane-induced short waves. The resulting nominal mass corresponds to rock class 10-15 tones.

At this point, it is important to note that the above distinction between short waves and swells is not an accurate representation of the real load conditions in the design area. In reality the action of short waves and swells is combined, and therefore the total wave energy that hits the structure is the combined energy of the two types of waves. This combined action is described by double-peak spectra, which have been presented in the previous chapter. This type of spectra is taken into account in the Van der Meer formula by making use of the first negative moment wave period, instead of the mean period. In the presented design the first negative moment period is calculated by SwanOne, yet not for a spectrum that represents the superposition of long and short waves, because also the input in SwanOne was distinct between short and long waves. For this reason, in case that a distinct consideration of short and long waves is made, like in this design, it is recommended that the design results are validated through model tests.

3.6 Quasi probabilistic design – PIANC method

An alternative method for design of a breakwater is the method of partial safety coefficients, which was worked out by PIANC (1992). This method introduces the use of safety coefficients for load and strength in the armour stability formula. The elaboration of this method in the case of Galveston jetties is presented below.

The probability of exceedance during service life of the design storm is the following:

$$P_{f-lifetime} = 1 - \left(1 - \frac{1}{t_L}\right)^{t_L} \qquad (3.4)$$

where t_L is the design lifetime, which has already been selected equal to 50 years. For t_L=50 years, $P_{f\text{-}lifetime}$=0.64

The return period of the design storm is:

$$t_{Pf} = \left(1 - (1 - P_f)^{1/t_L}\right)^{-1} \qquad (3.5)$$

where P_f is the probability of failure during lifetime. For P_f=5%, t_{Pf}=975 years.

The partial safety coefficient for load is calculated with the following formula:

$$\gamma_{Hss} = \frac{H_{ss}^{t_{Pf}}}{H_{ss}^{t_L}} + \sigma'_{Q_L}\left(1 + \left(\frac{H_{ss}^{3tL}}{H_{ss}^{tL}} - 1\right)k_\beta P_f\right) + \frac{0.05}{\sqrt{P_f N}} \qquad (3.6)$$

In this equation the following variables are present:

$H_{ss}^{t_L}$ = wave height for the chosen design lifetime t_L

$H_{ss}^{3t_L}$ = wave height for design lifetime $3t_L$

$H_{ss}^{t_{Pf}}$ = wave height with return period t_{Pf} based on the chosen probability of failure during lifetime t_L

k_β = coefficient determined by optimization (PIANC manual 1992)

P_f = probability of failure during lifetime, chosen by the designer

N = number of storms in a PoT-analysis

σ'_{QL} = standard deviation dependent on the type of wave measurements available (PIANC manual 1992).

If the wave statistics are not based on N wave data, but are based on other types of estimates, then the last term of equation 3 disappears, and a value for σ’ is chosen that accounts for the inherent uncertainty.

The partial safety factor for strength is calculated with the following formula:

$$\gamma_z = 1 - (k_\alpha \ln P_f) \tag{3.7}$$

where k_α = coefficient determined by optimization (PIANC manual 1992).

For the design of Galveston jetties the stability formulae of Van der Meer have to be applied four times, since there are four different types of waves that are taken into account. Hence the partial safety factors are also calculated four times, once for each set of wave data.

3.6.1 Storm-induced short waves

For the statistical analysis of storm-induced short waves a Gumbel distribution has been used:

$$Q = \exp\left(-\exp\left(-\exp\left(-\frac{H_{ss}-\gamma}{\beta}\right)\right)\right) \tag{3.8}$$

The non-exceedance probability for $N_s t_L$ events during lifetime is:

$$Q_{tL} = (1-Q)^{N_s t_L} \Rightarrow H_{ss} = \gamma - \beta \ln\left(\ln\left(1 - \frac{\ln Q_{tL}}{N_s t_L}\right)\right) \tag{3.9}$$

For the analysis of this kind of wave data average values of Global Wave Statistics and Argoss data have been used. The total observations are therefore the summation of GWS and Argoss observations, which equals to 1581 and 1831 for the north and south jetty respectively. Since the duration of observation period is not known, an assumption is made. It is assumed that the available data refer to 20-year observations. So the number of storms per year for the north and south jetty are the following:
north jetty: N_s=1581/20=79.05
south jetty: N_s=1831/20=91.55

The coefficients k_α and k_β take the values 0.027 and 38 respectively for use of Van der Meer formula in rock armor layer and plunging waves (PIANC manual 1992). The standard deviation is considered σ'=0.2. This is the most unfavourable value between the values for visual observations (σ'=0.2) like the Global Wave Statistics, and radar records (σ'=0.15) like Argoss data.

Based on the above equations and data, the partial safety coefficients are calculated for the design of north and south jetty. A summary of the calculation of H_{ss} values is presented in the table 5, while the calculated partial coefficients can be seen in table 6.

t (years)		$P_{f,lifetime}$	Q_{tL}	$H_{ss(north)}$ (m)	$H_{ss(south)}$ (m)
t_L	50	0,636	0,364	9,60	10,07
$3t_L$	150	0,633	0,367	10,70	11,24
$t_{Pf}=t_{5\%}$	975	0,632	0,368	12,57	13,22

Table 3.3: Summary of H_{ss} calculation

	γ_{Hss}	γ_z
north jetty	1,39	1,08
south jetty	1,56	1,08

Table 3.4: Partial safety coefficients for storm-induced short waves

3.6.2 Storm-induced swells

For storm-induced swells no statistical analysis has been performed. A simplified assumption has been made instead, that the design swell is H_s=4.5m, T_m=13s in all standard return periods from 10 to 10000 years. According to this assumption, the exceedance curve is a constant straight line (figure 3.20).

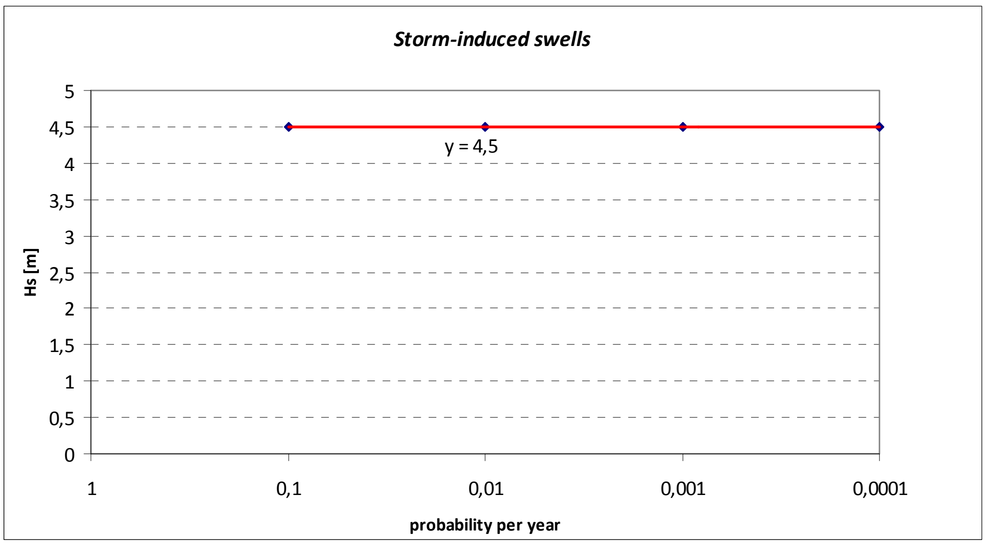

Figure 3.2: Exceedance curve of storm-induced swells

The coefficients k_α and k_β take in this case values 0.031 and 38 respectively, for use of the Van der Meer formula in rock armour layer, and plunging waves. Due to the rough estimation of this load case, it is considered safer to neglect the last term of equation 5, and to use a standard deviation σ'=0.35, which accounts for the inserted uncertainty.

In this particular case all storm wave heights take the standard value 4.5m. The calculated partial coefficients for strength and load are presented in the table below.

	γ_{Hss}	γ_z
north jetty	1,35	1,093
south jetty	1,35	1,093

Table 3.5: Partial safety coefficients for storm-induced swells

3.6.3 Hurricane-induced short waves

The determination of design hurricane-induced waves has been based on hurricane statistics. According to the exceedance curves of the occurring hurricane wind speeds and eye pressures, waves with standard return periods have been derived. Based on these wave data, a wave height exceedance curve is produced, which can be used for the calculation of partial safety coefficients. The produced exceedance fitted curve is exponential and can be seen in the following graph.

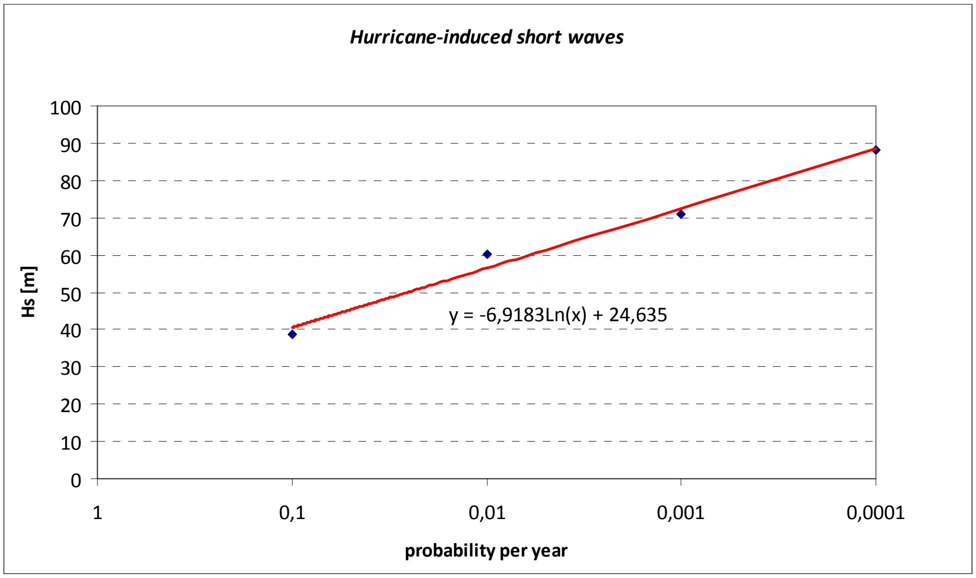

Figure 3.3: Exceedance curve of hurricane-induced short waves

The coefficients k_α and k_β take the values 0.027 and 38 respectively for use of Van der Meer formula in rock armour layer and plunging waves (PIANC manual 1992). Like in the case of storm-induced swells, here too the wave characteristics are not based on N wave data, but on estimates of the significant wave height. Hence the third term of equation 5 is again neglected, and a standard deviation σ'=0.35 is used. The calculation procedure of the different H_{ss} values and the derived partial coefficients for load and strength are presented in the tables below.

t		$P_{f,lifetime}$	P_f/year	H_{ss} (m)
t_L	50	0,636	0,013	54,83
$3t_L$	150	0,633	0,004	62,46
$t_{Pf}=t_{5\%}$	975	0,632	0,001	75,42

Table 3.6: Summary of H_{ss} calculation

	γ_{Hss}	γ_z
north jetty	1,641	1,081
south jetty	1,641	1,081

Table 3.7: Partial safety coefficients of hurricane-induced short waves

3.6.4 Hurricane-induced swells

The exceedance curve of this type of waves is derived in a way similar to the curve of hurricane-induced short waves, and is presented in the graph below.

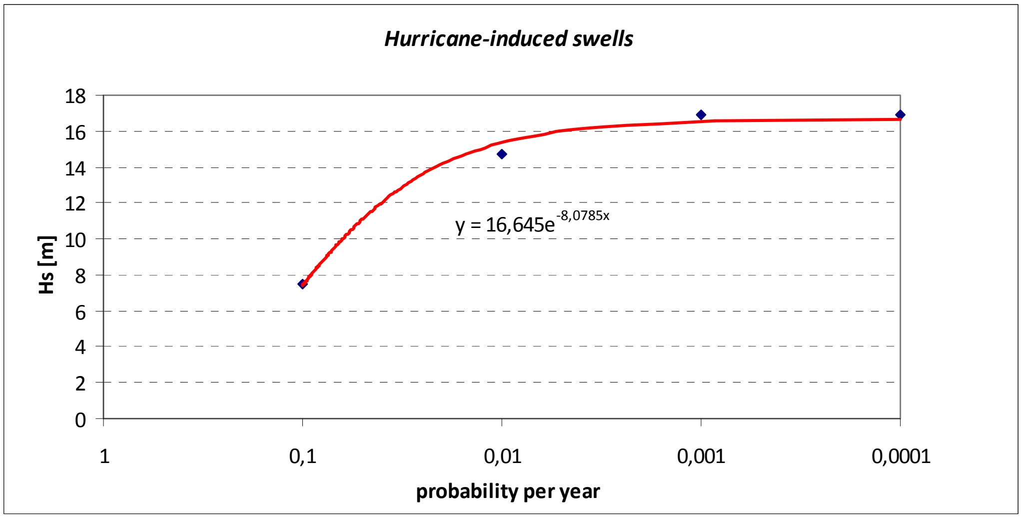

Figure 3.8: Exceedance curve of hurricane-induced swells

The coefficients k_α and k_β take the values 0.027 and 38 respectively for use of Van der Meer formula in rock armour layer and plunging waves (PIANC manual 1992). Similarly to the case of short waves the standard deviation is considered equal to 0.35, and last term of equation 5 is neglected. The summary of H_{ss} calculation and the derived partial coefficients are presented in the tables below.

t		$P_{f,lifetime}$	P_f/year	H_{ss} (m)
t_L	50	0,636	0,012717	15,02
$3t_L$	150	0,633	0,004222	16,09
$t_{Pf}=t_{5\%}$	975	0,632	0,000648	16,56

Table 3.8: Summary of H_{ss} calculation

	γ_{Hss}	γ_z
north jetty	1,406	1,081
south jetty	1,406	1,081

Table 3.9: Partial safety coefficients of hurricane-induced swells

3.6.5 Dimensioning of armour units

Equation 3.6, which calculates the partial safety coefficient for load, makes use of three storm wave heights with different return periods. Those parameters are wave conditions in deep waters. Since the calculation of the safety coefficient is based on data that refer to deep waters, the stability of the armour layer can only be determined with the Van der Meer formulae for deep waters, which are also indicated in the PIANC manual of 1992. For rock armour units the formulae take the following form:

For plunging waves:
$$\frac{1}{\gamma_z} c_{pl} S^{0.2} P^{0.18} \Delta d_{n50} (\cot\alpha)^{0.5} s_m{}^{0.25} N^{-0.1} \geq \gamma_{Hss} H_{ss}^{tL} \qquad (3.10)$$

For surging waves:
$$\frac{1}{\gamma_z} c_s S^{0.2} P^{-0.13} \Delta d_{n50} (\cot\alpha)^{0.5-P} s_m{}^{0.5P} N^{-0.1} \geq \gamma_{Hss} H_{ss}^{tL} \qquad (3.11)$$

For the coefficients c_{pl} and c_s the values 6.2 and 1.0 are recommended in the PIANC manual respectively.

The wave height values that have to be used in the Van der Meer formulae are the storm waves with a probability of exceedance of once in the lifetime of the structure (H_{ss}^{tL}). For those wave values the mean wave periods have not been estimated before. For storm short waves the mean periods can be directly derived from the correlation graph of significant wave height and mean period (figure 2.15). For storm induced swells the value of mean wave period is assumed to be constant and equal to 13 seconds. For hurricane-induced waves the period values cannot be extracted directly from the hydraulic boundary conditions. What can be easily derived though is an average wave steepness based on the wave characteristics with standard return periods, which can be used for the calculation of the mean wave periods that correspond to H_{ss}^{tL} wave values.

A summary of the storm wave heights that correspond to each type of waves, and the estimated mean wave periods is presented in the following table.

		H_{ss}^{tL} (m)	T_m (sec)
north jetty	Storm-induced short waves	9,60	5,77
	Storm-induced swells	4,50	13,00
	Hurricane-induced short waves	54,83	21,21
	Hurricane-induced swells	15,02	31,01
south jetty	Storm-induced short waves	10,07	6,45
	Storm-induced swells	4,50	13,00
	Hurricane-induced short waves	54,83	21,21
	Hurricane-induced swells	15,02	31,01

Table 3.10: Wave heights and mean wave periods used in the application of stability formula

According to the above given information the surf similarity parameters ξ_{cr} are calculated, and then the stability formulae 3.10 and 3.11 are applied for the dimensioning of the armour units. The calculated weights of the armour units are shown in the table below. The critical design values are marked red.

Wave type	d_{n50} (m)	M_{50} (kg)
Storm-induced short waves	1.54	9758
Storm-induced swells	1.06	3183
Hurricane-induced short waves	2.88	63143
Hurricane-induced swells	0.61	595

Table 3.11: Armour unit sizes for the different hydraulic boundary conditions

As expected the weight of armour units resulting from the PIANC design is extremely high. A rock armour unit of 63 tones can certainly not exist, and it is doubtful if there could be an artificial unit of this size constructed. If compared to the classical deterministic design, the results of PIANC design are approximately 5 times higher. This is due to the fact that depth-limitations in the wave height are not taken into account in the PIANC method, and therefore incorrect results are extracted in case of shallow water designs, like Galveston jetties.

3.7 Fully probabilistic design – Monte Carlo simulation

The application of a fully probabilistic computation is the design method that deals explicitly with all uncertainties. This computation can be performed on level 2 or 3 with a First Order Reliability method or Monte Carlo simulation respectively. All simulations can be elaborated with appropriate MATLAB routines. A level 3 calculation is more accurate than level 2, hence a level 3 design is chosen to be performed for Galveston jetties. According to this probabilistic calculation the results of the previously presented designs can be first controlled and evaluated. If it is necessary, a new probabilistic design can be then elaborated.

3.7.1 Limit state function

The first step for a probabilistic computation is to rewrite the design formula as a limit state function. According to previously presented calculations, for hurricane induced short waves, which constitute the design condition, the site is dominated by plunging waves, and therefore the design formula is the following:

$$\frac{H_{2\%}}{\Delta d_{n50}} = c_{pl} \cdot P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \left(s_{m-1,0} \right)^{0.25} \sqrt{\cot \alpha} \tag{3.12}$$

Accordingly, the limit state function is as follows:

$$Z = c_{pl} \cdot P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \left(s_{m-1.0} \right)^{0.25} \sqrt{\cot \alpha} \cdot \Delta d_{n50} - H_{2\%} \tag{3.13}$$

This equation contains nine parameters whose distribution needs to be determined. The determination of distributions is here mostly based on bibliographical review, and mainly on information contained in the book Breakwaters and Closure Dams (Verhagen, d' Angremond & van Roode).

3.7.2 Probabilistic determination of limit state function parameters

3.7.2.1 Coefficient c_{pl}

The coefficient c_{pl} is assumed to be normally distributed with a mean value μ=8.4 (Verhagen & Mertens, 2007). According to Van der Meer its standard deviation is approximately 10% of its value, thus σ=0.84.

3.7.2.2 Notional permeability P

The notional permeability is assumed normally distributed with a mean value μ=0.4. This is a parameter affected by the quality of construction. For all parameters that are connected to the construction process a standard deviation equal to 5% of their values is assumed, thus σ=0.02.

3.7.2.3 Damage level S

The acceptable level of damage is a target value, and therefore it should be deterministic, S=5.

3.7.2.4 Number of waves N

The number of incoming waves is affected by the storm duration. Although in this design storm duration of 12 hours has been taken into account, this value can vary considerably, meaning that also the standard deviation of the number of waves can be large. In this case, the use of a normal distribution can lead to negative number of waves. Since the effect of the number of waves is not so high, and in order to avoid the occurrence of negative number of waves, a lognormal distribution is assumed. The parameters of a lognormal distribution are the mean value and standard deviation of the parent distribution, μ_Y and σ_Y respectively.
For the x-distribution, the mean value is the average of the number of waves, which is assumed equal to the number for design conditions, μ_X=6042. The variation of this number depends on the possible occurring storm duration, which is assumed to vary from 1 to 24 hours. According to this assumption the maximum possible standard deviation of the x-distribution can be calculated as follows:

$$\sigma_x = \sqrt{\frac{(503-6042)^2 + (12084-6042)^2}{2}} = 5796$$

The parameters μ_Y and σ_Y are calculated with the following formulae:

$$\sigma_Y = \sqrt{\ln\left(1+\left(\frac{\sigma_x}{\mu_x}\right)^2\right)} \tag{3.14}$$

$$\mu_Y = \ln\left(\mu_X - \frac{1}{2}\sigma_Y{}^2\right) \tag{3.15}$$

(3.14) $\Rightarrow \sigma_Y = 0.833$, (3.15) $\Rightarrow \mu_Y = 8.71$

3.7.2.5 Wave steepness $s_{m-1.0}$

The wave steepness is assumed to be normally distributed. Its mean value and standard deviation can be calculated directly from the wave dataset with the use of basic statistical formulae. The calculated values are μ=0.041 and σ=0.012. It should also be noted that a zero correlation between steepness and wave height is assumed.

3.7.2.6 Structure slope cota

The slope of the structure is assumed normally distributed with mean value μ=2. Like notional permeability, the slope is a parameter dependent on the construction, and therefore its standard deviation is assumed equal to 5% of its average, σ=0.1.

3.7.2.7 Relative mass density Δ

The relative mass density is dependent on the water and rock density according to the formula:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{3.16}$$

It is therefore more accurate to consider the water and rock density as problem parameters, and to define their distributions instead of the relative mass density. Both variables are normally distributed. The salt water density can vary in the range 1025-1035kg/m^3. A representative mean value is therefore μ=1030kg/m^3, while its standard deviation is rather small. The larger the standard deviation, the higher the probability of failure becomes. For this reason the largest possible standard deviation is chosen for the simulation, which is the one resulting from the following calculation:

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \mu)^2}{n}} \Rightarrow \sigma = \sqrt{\frac{(1025-1030)^2 + (1035-1030)^2}{2}} = 5$$

The granite density varies in the range 2500-2800kg/m^3 (Rock Manual, 2007). A mean value μ=2650kg/m^3 is therefore considered. By making the same considerations for standard deviation as in the case of salt water density, for rock density the following value is resulting:

$$\sigma = \sqrt{\frac{(2500-2650)^2 + (2800-2650)^2}{2}} = 150$$

3.7.2.8 Nominal diameter d_{n50}

Like the relative mass density, the nominal stone diameter is not independent variable. It depends on the rock density and mass according to the following formula:

$$d_{n50} = \sqrt[3]{\frac{M_{50}}{\rho_s}} \qquad (3.17)$$

For this reason the nominal diameter is replaced by the two new variables in the limit state function. The distribution of the rock density together with its properties has already been determined. The nominal rock mass can be assumed normally distributed. Its mean value and standard deviation can be derived from the corresponding stone class.

The classical deterministic design resulted in the stone class 3-6 tones. The mean value of the nominal rock mass is assumed μ=4500kg. For armor units, the nominal mass of this stone class can vary in the range 4430-5060 kilograms (Table 3.6, Rock Manual 2007). For the probabilistic computation the most unfavorable standard deviation is used, which is calculated as follows:

$$\sigma = \sqrt{\frac{(4430-4500)^2 + (5060-4500)^2}{2}} \approx 399$$

The PIANC design resulted in a stone class 10-15 tones. In accordance to the classical design, the mean value of the nominal mass is assumed μ=12500kg, while the estimation of the standard deviation is based on limitations indicated in table 3.6 of the Rock Manual. The range of the nominal diameter of this stone class is 12000-13000kg, and therefore the standard deviation is:

$$\sigma = \sqrt{\frac{(12000-12500)^2+(13000-12500)^2}{2}} = 500$$

3.7.2.9 Wave height $H_{2\%}$

The wave height is the parameter with the highest degree of uncertainty in the limit state function. The main sources of uncertainty are the wave data used for the determination of design values, and the effect of wave breaking, which cannot be accurately defined. The latter uncertainty is basically an issue in shallow waters.

In deep waters the wave height follows an extreme value distribution. In the case of Galveston though the water is shallow, meaning that only depth-limited waves can exist, and therefore no extreme value wave heights occur. In this case it can be assumed that the wave height is a function of the water depth h:

$$H_{2\%} = \gamma_b h \tag{3.18}$$

In the above equation the parameter γ_b is the breaker index, which can be assumed normally distributed. For foreshores with a gentle slope its mean value and standard deviation can be considered μ=0.55 and σ=0.05 respectively (Verhagen, d' Angremond, Van Roode, Breakwaters and closure dams). As water depth h the total depth is considered, i.e. the depth below mean sea level, plus the rise of water level due to tide and storm surge. Based on this information equation 3.18 can be rewritten as follows:

$$H_{2\%} = \gamma_b (h_{depth} + h_{surge}) \tag{3.19}$$

The water depth below mean sea level has a normal distribution. However its standard deviation is usually so low that it can be considered as a deterministic value. In the case of Galveston it can be considered deterministic, h_{depth}=7m.

The surge in shallow water is a function of different parameters depending on the hydraulic conditions that are examined each time. In case that the analysis concerns a regular storm, the storm surge is the sum of wave set-up and wind set-up. These two parameters can be considered highly correlated since they are always induced by the same storm. Hence in the final probabilistic computation for the armor layer, a joint probability function of the two parameters has to be used. If a hurricane pass is the determining design condition, which is the case for Galveston, the surge is defined in a different way, which is explained in the following paragraph.

3.7.3 Probabilistic determination of hurricane-induced surge level

When a hurricane crosses the area of interest, the maximum surge is defined as the sum of high tide, wind and pressure set-up (see equation 2.3). The wind and pressure set-up are functions of many other parameters, whose distributions have to be derived. It is important to note that in order to run a Monte Carlo simulation, the problem variables have to be uncorrelated. The analysis of the surge parameters is presented below.

3.7.3.1 Tidal level

The distribution of the tidal water level raise can be derived from tidal datasets that are available by NOAA (figure 3.5). In particular high tide measurements of the period November 2009-May 2010 have been used.

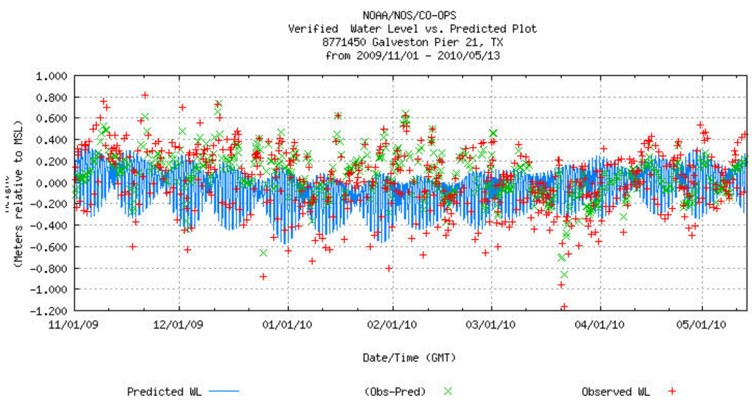

Figure 3.5: Galveston tidal level dataset (NOAA)

Those data have been inserted in BestFit software, which indicated a Weibull distribution with parameters α=1.46 and β=0.29 as the curve closest to the data series (figure 3.6).

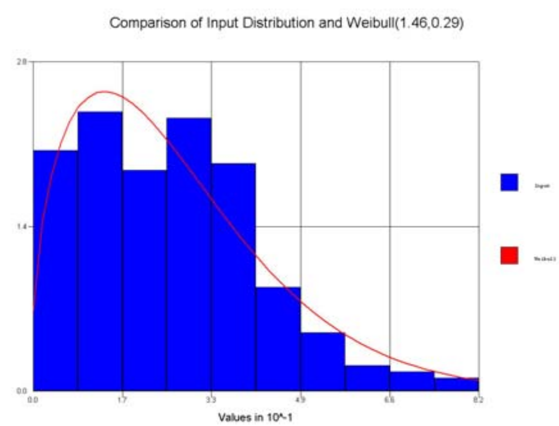

Figure 3.6: BestFit output for tidal level

3.7.3.2 Pressure setup

As indicated in the chapter of hydraulic boundary conditions, the pressure setup induced by a hurricane can be calculated with equation 2.4, which contains the following variables:
p_n = atmospheric pressure,
p_α = relative pressure as a function of distance between the hurricane eye and the design spot, and of the hurricane eye pressure,
γ_w = specific weight of salt water.

The analytic steps for choice of the appropriate distribution for each parameter are presented below.

Specific weight of salt water γ_w
A normal distribution can be assumed for the specific weight of salt water. Its mean value is μ=10250N/m^3, while its standard deviation is considered relatively small, σ=50.

Atmospheric pressure p_n
This parameter is measured regularly by NOAA in Galveston. Based on those measurements some monthly statistics can be extracted directly from the NOAA website, which is presented in figure 3.7. They concern the period 1993-2001.

It can be assumed that the atmospheric pressure follows a normal distribution. Its parameter values can be estimated based on the above plot. Since the hurricane season lasts from July to October, only data of these four months are relevant, which give an average mean value μ=101552.5 N/m^2 and standard deviation σ=3.94.

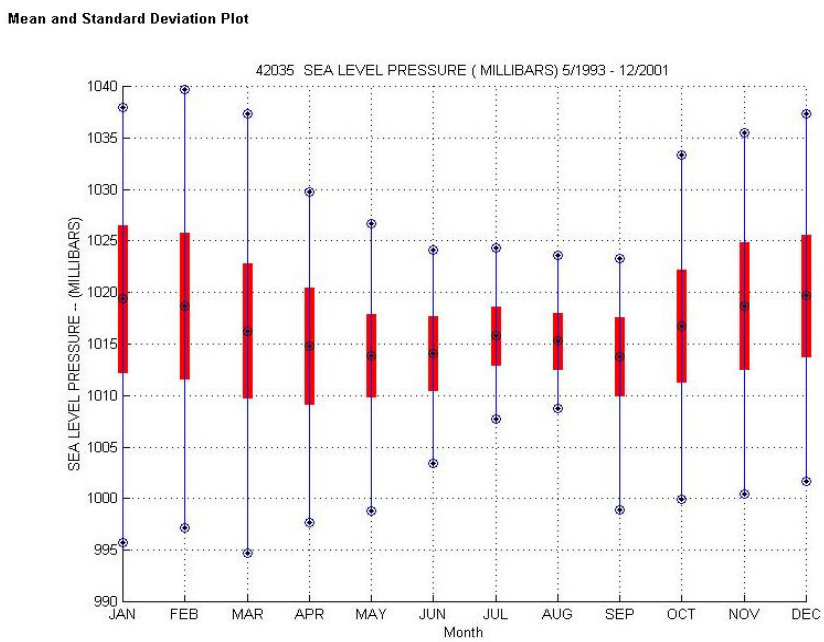

Figure 3.7: Atmospheric pressure statistics (NOAA)

Relative pressure p_α
This pressure is a function of the distance between the center of the hurricane and the spot of interest r_s, and of the minimum pressure at the hurricane eye and has been presented in chapter 2 (equation 2.5)

The eye pressure is a parameter highly correlated to the hurricane speed. The relation between these two parameters has been presented in a graph in chapter 2 (figure 2.6). According to this graph the following relation is derived, which is used in the probabilistic model.

$$p_o = 108106.67 - 222.22u \tag{3.20}$$

The radius R is only a function of the atmospheric pressure and the eye pressure, whose distributions have already been specified. Therefore it can be replaced by this function in the design formula.

The distance between the center of the hurricane and the spot of interest r_s is a function of parameters that have not been specified yet. These parameters are associated to the hurricane track and its relative position with respect to the spot on the jetty where the design is elaborated. An effective way to describe a hurricane track is by means of a line's equation y=Ax+C, on a coordinate system in the area of Galveston. Any track can then be described by two variables, the slope of the line with respect to x-axis A, and the spot that the line intersects with y-axis C. The center of the coordinate system is assumed to be at the spot where the south jetty meets the coast, while the coastline, which is assumed to be a straight line, constitutes the y-axis. The total formation of the problem geometry can be seen in the following figure.

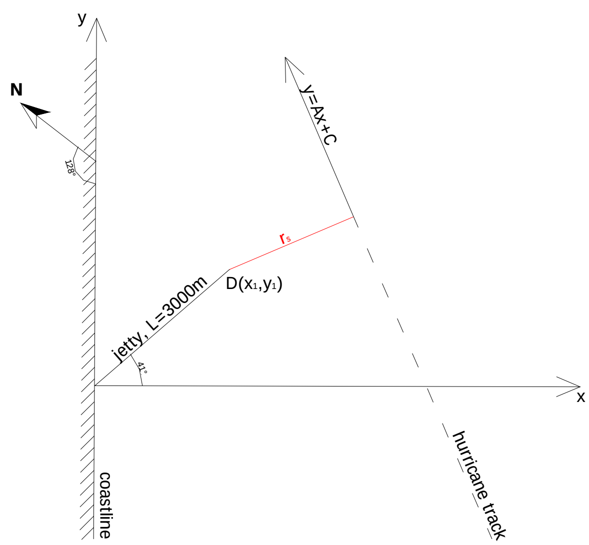

Figure 3.8: Geometric description of distance r_s

The distance between the design spot D and the center of the hurricane can be described by the algebraic equation that describes the distance of a spot from a line:

$$r_s = \frac{|Ax_1 - y_1 - C|}{\sqrt{A^2 + 1}} \tag{3.21}$$

Where x_1 = x-coordinate of spot,
y_1 = y-coordinate of spot,
A = slope of line with respect to x-axis,
C = y-coordinate of the spot that the line intersects with x-axis.

The orientation of the south jetty is already known, as it is an existing structure. Its angle with the x-axis is equal to 41°. The distance L between the design spot and the system center is also known and equal to 3000m. Based on this information, the coordinates of the design point can be easily calculated as follows:

$$x_1 = L\cos 41^o = 3000\cos 41^o \simeq 2264$$
$$y_1 = L\sin 41^o = 3000\sin 41^o \simeq 1968$$

The slope A of the line is in fact the tangent of the angle between the line and the x-axis:

$$A = \tan\beta \quad (3.22)$$

According to the hurricane records of NOAA, it seems that all hurricane tracks have the same probability of occurrence (see figure 2.4). It can be therefore assumed that both angle β and y-coordinate C are uniformly distributed. Angle β can take values in the range (-90^o, 90^o). The values 90^o and -90^o are not included in this range, because in this case the slope of the hurricane track with respect to x-axis becomes infinite, which makes no physical sense. Hence the range decreases to [-89^o, 89^o]. The range of parameter C is determined by the limitations that are imposed by assuming a straight coastline along Galveston. This assumption seems to be correct in a distance of 70 kilometers on either side of Galveston inlet. Thus parameter C is assumed to belong in the range [-70km, 70km].

3.7.3.3 Wind setup

The wind setup that is induced by a hurricane can be calculated with equation 2.7, in which the following parameters are present:
c_1 = calibration parameter for the friction between wind speed and water surface,
u = circular hurricane speed,
F = circular fetch
d = water depth along fetch
g = gravitational acceleration.

The analytic steps for choice of the appropriate distribution for each parameter are presented below.

Calibration parameter c_1
This variable is assumed to be normally distributed. As the Atlantic hurricanes are physically indifferent from the typhoons that occur in the area of Vietnam, the derivation of distribution parameters can be based on available data of the Vietnamese coast (Maaskant, van Vuren, Kallen, 2009). According to those data, the mean value and standard deviation of parameter c_1 are $\mu=4\cdot10^{-6}$ and $\sigma=1.27\cdot10^{-6}$ respectively.

Circular hurricane speed u
The total hurricane speed consists of two components, a linear motion and a circular motion, the latter of which is the dominating one. Based on this fact, it is assumed that the circular hurricane speed is identical to the one-minute-sustained wind speed u_{10}, which determines the hurricane category, and therefore is indicated in the available hurricane records. An important aspect that needs to be taken into account in order to come up with a correct distribution is the maximum hurricane speed that is physically possible to occur. This value has been indicated in chapter 2 equal to 113m/s.

Accounting for this maximum wind speed and the wind exceedance curve that has also been derived before, the best distribution type can be extracted via BestFit software. As optimal curve a logistic distribution is given, which is though not a good choice from a physical point of view. The reason is that a logistic distribution can take both positive and negative values, which is not the case for the hurricane eye pressure. The eye pressure is a non-negative parameter, and unless it is represented by a non-negative distribution, some inconsistencies can occur in a Monte Carlo simulation, which lead to errors or incorrect results, depending on the used model. The optimal non-negative curve according to BestFit is a gamma distribution with parameters α=13.33 and β=3.57, which is depicted in the following graph.

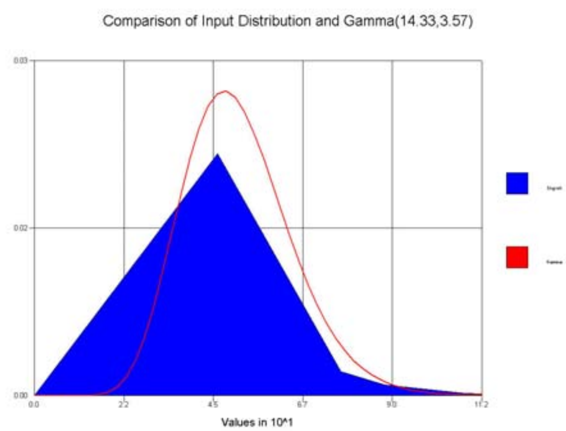

Figure 3.9: BestFit output for circular hurricane speed

For a parameter like the hurricane speed, whose range of values is known, while the most probable value can be fairly assumed, it is more convenient to use a triangular distribution. This is a distribution that from a mathematical point of view can be considered inadequate for the description of physical parameters, because it gives only an approximate representation of the reality. Nevertheless due to its simple definition it is always convenient to be used. For the case of hurricane speed, it has proven through a trial and error procedure that the use of a triangular distribution with parameters a=0, b=113, c=42 leads to results with insignificant divergence from the ones extracted when a Gamma distribution is used. Hence instead of Gamma a triangular distribution is used.

This conclusion could also be drawn by the results of BestFit analysis. As shown in the above graph, the hurricane speed density graph which is based on the input values is an almost perfect triangle. This is because the input values are also result of an approximating procedure, since the hurricane speed exceedance curve was derived by only four spots (figure 2.7).

Circular fetch F

It is assumed that the circular hurricane fetch is not correlated to the circular hurricane speed, but it is only dependent on the hurricane track. In particular it can be defined as the following product:

$$F = \varphi \cdot r_s \tag{3.23}$$

Where ϕ = epicentral angle of fetch

r_s = distance between the spot of interest and the center of the hurricane.

The distance r_s has already been defined as a function of the hurricane direction expressed with the angle β and parameter C, and of the coordinated of the spot of interest D (x_1, y_1). The epicentral angle φ is only a function of angle β, and can be defined as follows:

$$\varphi = \beta + 90^o \tag{3.24}$$

It should be noted that in the above equation the algebraic values of angle β have to be used, meaning both positive and negative values in the range [-89^o, $+89^o$]. The following graph indicates in which case angle β is taken as positive and in which cases as negative.

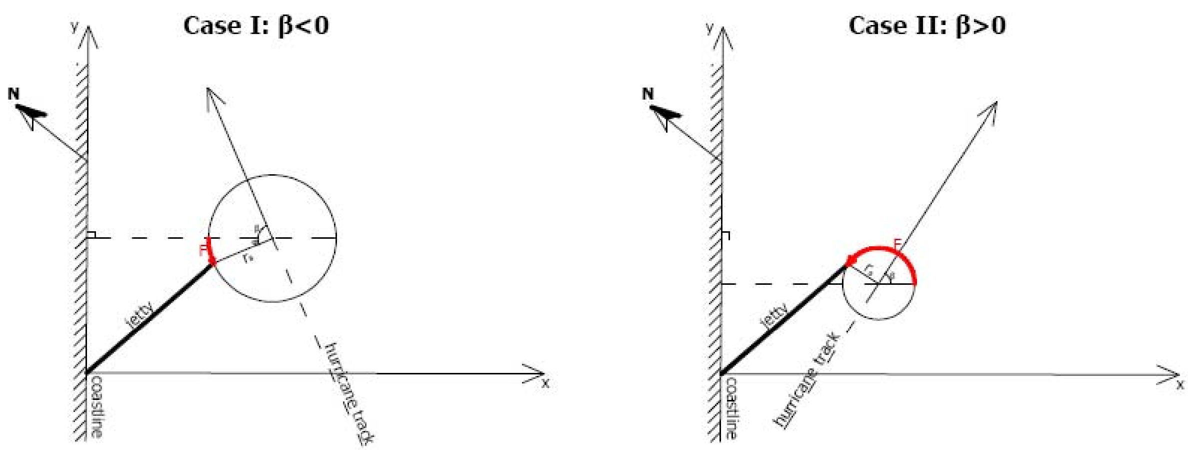

Figure 3.10: Graphical representation of fetch and angle β

Water depth along fetch d
The bottom slope in the area around Galveston jetties is considered small enough to be neglected. Therefore the water depth along the hurricane fetch can be considered deterministic with value equal to the water depth at the design spot, d=7m.

Gravitational acceleration g
This variable is assumed deterministic. Its value is g=9.81m/s^2.

3.7.4. Monte Carlo simulation

3.7.4.1 Limit state function

According to the above, a convenient way to rewrite the limit state function for programming a Monte Carlo simulation in MATLAB is the following:

$$Z = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (s_{m-1.0})^{0.25} \sqrt{\cot\alpha} \cdot \Delta d_{n50} - \gamma_b \left(h_d + h_t + h_p + h_w \right) \tag{3.25}$$

Where $$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{3.26}$$

$$h_p = \frac{p_n - p_o}{10\gamma_w \left[1 + \left(\frac{r_s}{R}\right)^2\right]} \tag{3.27}$$

$$h_w = \frac{c_1 u^2 \varphi r_s}{gd} \tag{3.28}$$

$$r_s = \frac{\left|2264 \tan\beta - 1968 - C\right|}{\sqrt{(\tan\beta)^2 + 1}} \tag{3.29}$$

$$R = 5443.6(p_n - p_o)^{0.5034} \tag{3.30}$$

$$\varphi = \beta + 90 \tag{3.31}$$

3.7.4.2 MATLAB input

The program input is the distributions of the problem parameters, i.e. the variables of the limit state function, together with their parameter values. An important remark at this point is that in MATLAB not all parameters can be used as derived above. Some modifications associated to the function of the program are necessary. The first one is that all angles have to be re defined in radius instead of degrees, because all trigonometric functions are calculated in radius in MATLAB. This affects two things: the range of variable β, which has to be defined as (-1.55, 1.55) instead of (-89, 89), and the definition of angle ϕ, which becomes:

$$\varphi = \beta + 1.57 \tag{3.32}$$

Another necessary modification is associated with the notation of Weibull distribution, which is different in MATLAB and in BestFit. While in BestFit the shape parameter and scale parameter are denoted with α and β respectively, in MATLAB the opposite notation is used. This affects the input notation of all Weibull distributed parameters, which are the hurricane eye pressure p_o and the tidal elevation h_t.

A summary of the input parameters as given in MATLAB is presented in the table below.

	Distribution	Parameter values
c_{pl}	Normal	μ=8.4 , σ=0.84
P	Normal	μ=0.4 , σ=0.02
S	Deterministic	S=5
N	Lognormal	μ_Y=8,71 , σ_Y=0,833
$s_{m-1,0}$	Normal	μ=0.041 , σ=0.012
cotα	Normal	μ=2 , σ=0,1
ρ_s	Normal	μ=2650 , σ=150
ρ_w	Normal	μ=1030 , σ=5
M_{50}	Normal	*determ.* *PIANC* μ=4500 , σ=399 / μ=12500 , σ=500
γ_b	Normal	μ=0.55 , σ=0.05
h_d	Deterministic	h_{depth}=7m
h_t	Weibull	α=0.29 , β=1.46
γ_w	Normal	μ=10250 , σ=50
p_n	Normal	μ=101552.5 , σ=3.94
β	Uniform	α=-1.55 , β=1.55
C	Uniform	α=-10000 , β=10000
c_1	Normal	μ=4e-6 , σ=1.27e-6
u	Triangular	α=0 , b=113, c=42
d	Deterministic	d=7
g	Deterministic	g=9.81

Table 3.12: Summary of MATLAB input

3.7.4.3 MATLAB output

After inserting the above data in the MATLAB script, the Monte Carlo simulation can be run. The number of samples used is 30000. Running the simulation twice, once for the classical design and once for the PIANC design, the probabilities of failure of the structure are extracted.

It should be noted that the type of extracted probabilities depends on the type of input that is used. If the inserted distribution functions refer to probabilities per year, then the final probability of failure will also be per year. If the input distributions refer to probabilities during lifetime, then the output will be a failure probability during lifetime of the structure. In this example no clear distinction between the two probabilities has been made during the presentation of the input parameters. This is due to the fact that for most of the parameters there is a lack off statistical data, and therefore the choice of their distributions and distribution parameters are based on critical considerations by the analyst. Parameters that refer to the hurricane though, and determine in a high degree the hydraulic load and the design outcome, such as the hurricane speed, are defined through statistics, since there are hurricane records in the area. The exceedance curve of wind speed gives probability values per year. Also the other hurricane parameters that determine in high degree the final design, such as the parameters related to the hurricane track can be considered uniformly distributed in a yearly basis. Hence the failure probabilities extracted from MATLAB are considered to be probabilities per year.
In order to compare the results of this simulation with the target probability, which is a probability during lifetime, the extracted values need to be transformed to probabilities during life time. This transformation is done with the following equation.

$$P_{f,lifetime} = 1 - (1 - P_{f,year})^{T_L} \tag{3.33}$$

Where T_L = lifetime of the structure.

The results of the simulation are presented in the table below.

	$P_{f,year}$ (%)	$P_{f,lifetime}$ (%)
Classical design	6.57	96.65
PIANC design	0.22	10.43

Table 3.13: Monte Carlo results

The target probability of failure of the two elaborated designs was 20%. According to the results, the classical deterministic design is not sufficient, as it is almost certain that the structure will fail during lifetime. On the other hand, the design with PIANC method is quite conservative, since the resulted probability of failure during lifetime is 50% lower than the target. This result was expected, since as mentioned before, the PIANC method gives incorrect results in cases that depth-limited waves determine the design load. Yet the divergence between the target probability and the one resulted for the classical design is higher than expected. This could be the cause of two things. First is the different definition of load in the classical and fully probabilistic design. For the deterministic design the wave height in $H_{2\%}$ as resulted from SwanOne simulations has been used as load, while for the probabilistic design the load is approximated as a percentage of the local water level, which is an approximation valid for shallow water conditions. Second is the random choice of a design hurricane in the deterministic design, which might be far from the most unfavorable hurricane case. This is always a risk inherent to a deterministic design, when the most unfavorable load cannot be easily estimated.

Through a trial and error procedure the armour unit size that corresponds to a probability of failure equal to the target, $P_{f,lifetime}$=20%, is derived. The appropriate unit size is 48 tones. This is a size that cannot be achieved with rock units, but artificial concrete units should be considered instead. This means that the whole design procedure should be repeated, since in case of artificial units the stability formula is also altered. Since this example is meant only for indicative purposes, this repetition is not further elaborated in this study.

3.8 Comparison of results

A summary of the results of the three elaborated designs is presented in the table below.

	M_{50} (kg)	stone class (t)	$P_{f,year}$ (%)	$P_{f,year}$ (%)
Deterministic design	12500	10-15	6.57	96.7
PIANC design	63000	[-]	0.22	10.4
Fully probabilistic design	48000	*(artificial units)*	0.0044	19.8

Table 3.14: Summary of design results

A comparative representation of the results is shown in the following graph as well, where the divergence of the three methods that has already been discussed in the previous paragraph, can be clearly seen.

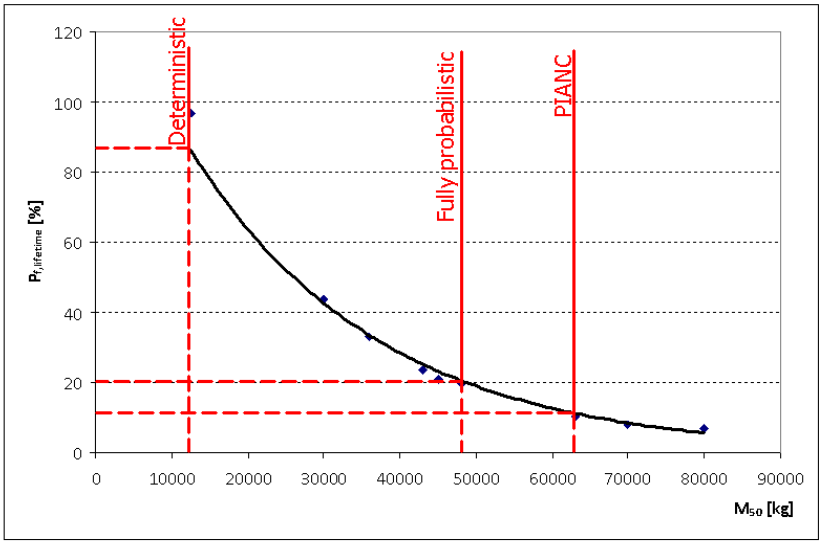

Figure 3.11: Comparison of designs

4. RATIONALISATION OF SAFETY FACTORS

4.1 Introduction

A probabilistic calculation on level 3 is supposed to be a very accurate one, yet a thorough understanding of the physical problem and a careful consideration of the uncertainties involved in the design is necessary. In most of the cases the application of a level 3 probabilistic calculation is observed as too complicated by breakwater designers, hence hardly ever chosen for the design of armour units. As mentioned before the most commonly used method for dimensioning of an armour layer is a deterministic application of the stability formula, with the use of a safety factor equal to 1.3 for the load. The basic concept of safety factors is a certainly good and effective method for designs that involve a lot of uncertainty like breakwater designs. However the use of a particular value for the safety factor is a very simplistic approach to the problem, since the degree of additional safety is undefined, thus there is a risk of being either too low, resulting in insufficient designs, or too high, resulting in over-dimensioning of the structure.

Since the concept of safety factors is very well established and preferred by the majority of designers, it is necessary to be modified in a way that leads to correct results. This can be achieved by re defining the safety factors in a way that takes into account the uncertainties inherent to the physical problem and the design itself. Ciria report 63 (Ciria, 1977) gives a format for rationalization of partial safety factors in structural codes, which is based on a level 2 reliability analysis. The overall procedure was also applied for the derivation of the PIANC partial safety coefficients.

For the case of Galveston a fully probabilistic design on level 3 has already been presented, in which all uncertainties have been explicitly indicated. Although there are improvements to be made in the model used for the fully probabilistic calculation, it is assumed that its results are sufficiently reliable and certainly more accurate than a level 2 analysis. Therefore they can be used as a basis for determination of a set of safety factors instead of following precisely the Ciria guideline that suggests a level 2 analysis. The main rationalization concept remains the same though as suggested by Ciria. The procedure as presented by W. Kanning, 2005 is the following and is elaborated in the paragraphs below:

1. Choice of safety format,
2. Calculation procedure based on the safety format,
3. Sensitivity analysis,
4. Evaluation of results.

4.2 Safety format

In order to come up with an optimal safety format the range of the guideline has to be chosen as well as the aim of the code. The range of the guideline is the following:

- Rubble mound breakwaters,
- Only shallow waters,
- Design conditions determined by hurricanes.

The scope of the code is to create a reliable and handy set of safety factors, which can be used in cases with the above mentioned characteristics, and will cover an important degree of the uncertainty inherent to the physical problem and the design.

One of the most important issues in the design of an armour layer is always the definition of the hydraulic load on the structure, which is determined by the action of waves. The design wave height is usually derived by wave records of questionable accuracy, and consequently high degree of uncertainty. In the Gulf of Mexico for instance, there are two sets of storm wave data, one from Global Wave Statistics and one from Argoss, which result in design wave heights of about 50% divergence. For locations like Galveston, where the design is meant for shallow water conditions, some extra uncertainty is inserted in the load definition due to wave breaking effects, which cannot be accurately estimated. Another important degree of uncertainty is imposed when hurricane conditions determine the design, which is also the case in Galveston, due to the stochastic character of hurricane loading. Then a design hurricane has to be defined, which is the one causing the most unfavourable load on the structure with a certain predefined frequency. The total uncertainty inherent to the load is therefore quite important.

An effective way of tackling this uncertainty would be the realisation of water level measurements in shallow waters when hurricane conditions occur, from which an exceedance curve of water levels during hurricane passes could be derived. The only uncertainty inherent to this curve, which should be compensated in a safety format, is the one related to the measurement technique. This solution is very theoretical though, since this type of measurements hardly ever exist. In most of the cases only water levels during normal weather conditions are available. Hence, given a particular return period, the determination of a design hurricane is most likely to be based only on the designer’s critical view. Eventually the safety format should take into account the totality of the above mentioned uncertainties.

The total uncertainty inherent to the load is therefore assumed to be much more important than the uncertainty of the strength, yet the latter can be easily defined and included in a strength safety factor. Since a handy tool is meant to be established with the new set of safety factors, the use of only two safety factors is suggested, one for the total load and one for the resistance, which will cover all load and resistance variations respectively. The limit state function has the following simple form:

$$Z = \gamma_R R - \gamma_S S \tag{4.1}$$

where the load and resistance are defined as in level 3 probabilistic design:

$$S = \gamma_b (h_t + h_d + h_p + h_w) \tag{4.2}$$

$$R = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (s_{m-1.0})^{0.25} \sqrt{\cot\alpha}\, \Delta d_{n50} \tag{4.3}$$

All parameters in the above equations have been analytically presented in paragraph 3. of chapter 3. The uncertainties covered from the load safety factor are the ones connected with load parameters and are listed below:

- Hurricane track (direction and starting point),
- Hurricane intensity (hurricane speed),
- Depth below mean sea level around Galveston,
- Tidal level,
- Depth below mean sea level at the design spot,
- Atmospheric pressure.

It is important to note that there are still some uncertainties related to the load which are not dealt with the load safety factor, the ones inherent to the wave steepness and the number of waves. These parameters are included in the strength term of the above limit state function.

The uncertainties covered by the resistance factor are connected with the determination of the following parameters:

- Coefficient c_{pl},
- Structure permeability,
- Number of incoming waves,
- Wave steepness,
- Armour layer slope,
- Salt water density,
- Rock density,
- Size of armour units.

It should be noted that the uncertainties associated with the probabilistic model are not incorporated in the safety factors.

The analytic steps that a designer should follow in order to come up with a design based on the above safety format are presented below.

1. Choice of design hurricane, and particularly of hurricane category, which is determined by the hurricane speed u, and of hurricane track, which is determined by the hurricane angle β and parameter C.
2. Based on the design hurricane, extract of load safety factor from graph that represents γ_S as a function of the design hurricane.

$$\gamma_S = f(u, \beta, C) \tag{4.4}$$

3. Calculation of characteristic load S^k, using design hurricane and mean values for remaining load variables.
4. Calculation of design load S*.

$$S^* = \gamma_S S^k \tag{4.5}$$

5. Choice of structural parameters and particularly of notional permeability P, allowable damage level S, slope of the structure cotα and type of armour units.
6. Calculation of armour unit size through application of Van der Meer stability formula, using the chosen structural parameters, mean values for remaining strength parameters and the design load of step 4. For the case of rock armour layer and plunging waves, like in Galveston, the equation has the following form.

$$c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} (s_{m-1.0})^{0.25} \Delta d_{n50}{}^{k} - S^* = 0 \tag{4.6}$$

7. Based on the calculated armour unit size, extract of strength safety factor γ_R from graph that represents γ_R as a function of the characteristic nominal mass.

$$\gamma_R = f(M_{50}{}^{k}) \tag{4.7}$$

8. Calculation of design armour unit nominal mass.

$$M_{50}{}^* = \gamma_R M_{50}{}^{k} \tag{4.8}$$

The above presented steps can be summarized in the following scheme, which can be considered as a guideline for future designers.

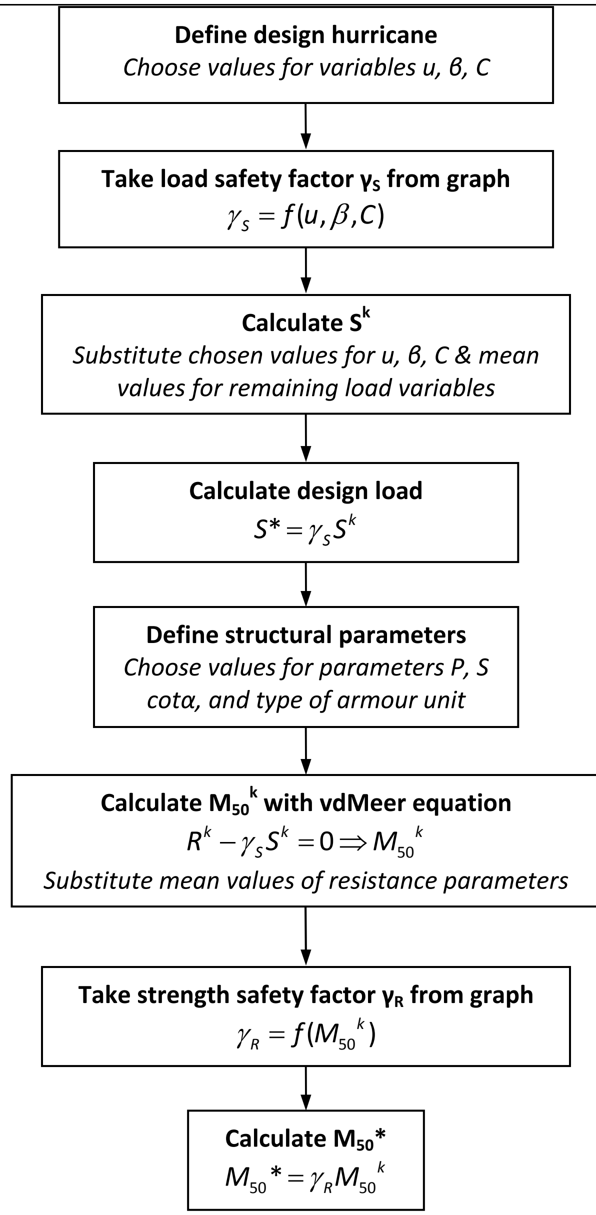

Figure 4.1: Guideline for future designers

4.3 Calculation procedure of safety factors

4.3.1 Calculation overview

An overview of the upcoming procedure for calculation of load and strength safety factors can be summarised in the following logical diagram.

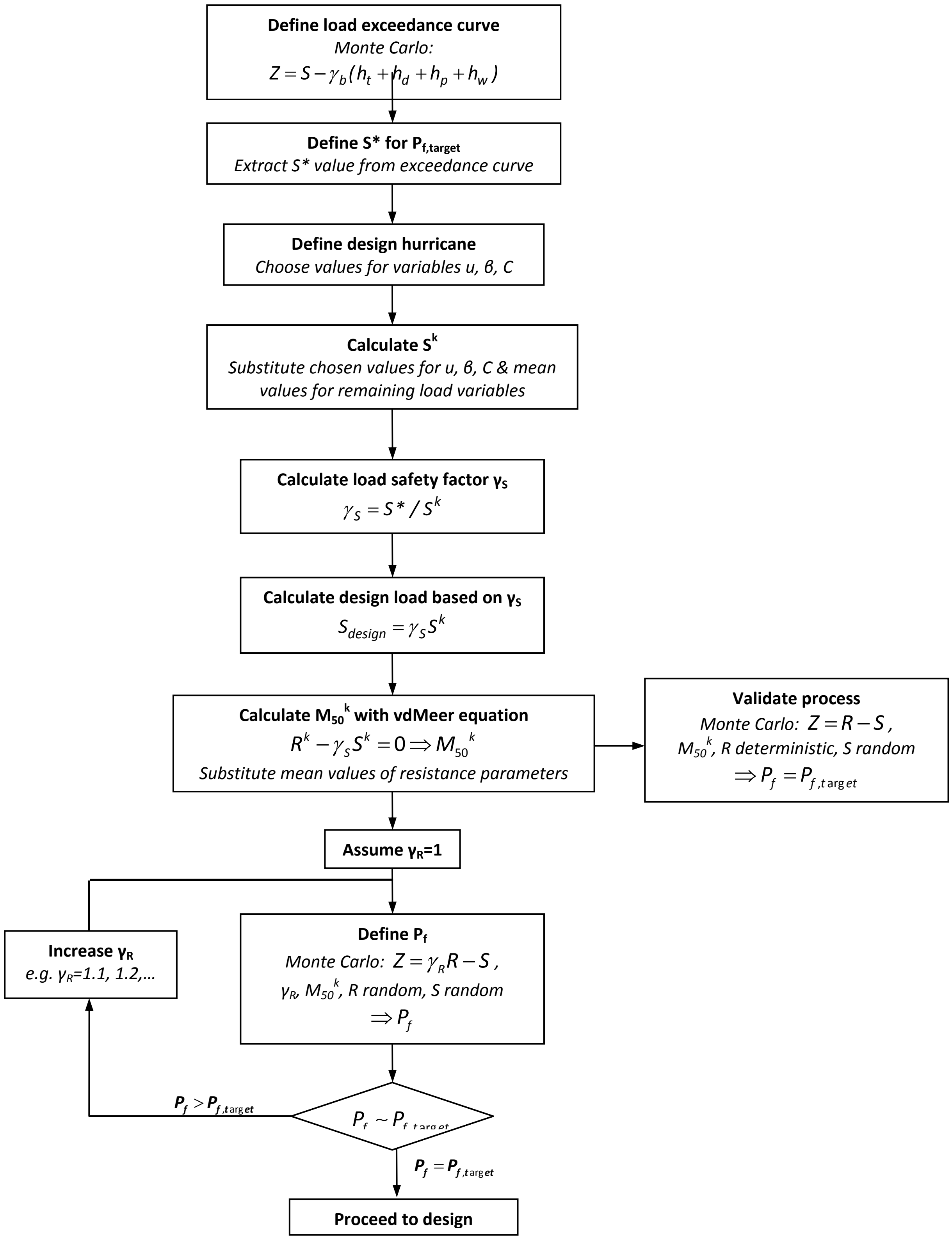

Figure 4.2: Guideline for code makers

This diagram constitutes a guideline for code makers who intend to derive safety factors for applications that belong to the range of the previously presented safety format. This is in fact a generic procedure that can be applied in all areas with the same characteristics as Galveston.

It should be noted though that the values of safety factors derived with this procedure will always be site-specific. This is due to the fact that the model used for the Monte Carlo simulation is site specific, as it contains features associated with the site, such as bathymetry. It is therefore necessary that in every case some appropriate modifications are made to the simulation model.

The analysis on which the above presented scheme has been based is presented in the following paragraphs.

4.3.2 Safety factor for load γ_S

A load safety factor is defined as follows:

$$\gamma_S = \frac{S^*}{S^k} \quad (4.9)$$

Where S^* = design load,
S^k = characteristic load.

The key element of a safety factor derivation is to determine these two types of loads.

4.3.2.1 Design load S^*

The design load can be calculated according to the model used for elaboration of the level 3 stability calculation. In particular, based on information used as input in Monte Carlo, a new fully probabilistic calculation of the total load can be elaborated in MATLAB with a new Monte Carlo simulation. The outcome is an exceedance curve of the load, which is supposed to be the one with the highest possible accuracy, as all uncertainties have been incorporated in a satisfactory way. Therefore, they can be used as design values, while a set of satisfactory results of a level 1 calculation should converge to the outcome of this simulation.

The limit state function for the new simulation is the following:

$$Z = S - \gamma_b (h_t + h_d + h_p + h_w) \quad (4.10)$$

Where S = total load, while the rest of the parameters have already been presented in chapters 2 and 3. There are in total 12 variables in the above function. For every particular deterministic value of the total load, a probability of failure is extracted, which is in fact the probability that the Z-function becomes negative.

$$P_f = P(Z < 0) \quad (4.11)$$

By giving various deterministic values to the total load, a design exceedance curve is created. An overview of the MATLAB input is presented in the table below, while the extracted design exceedance curve for the total load is shown in figure 4.3.

	Distribution	Parameter values
S	Deterministic	S=[0,40]
γ_b	Normal	μ=0.55 , σ=0.05
h_d	Deterministic	h_{depth}=7.5m
h_t	Weibull	α=1.46 , β=0.29
γ_w	Normal	μ=10250 , σ=50
p_n	Normal	μ=101552.5 , σ=3.94
β	Uniform	α=-89 , β=89
C	Uniform	α=-10000 , β=10000
c_1	Normal	μ=4e-6 , σ=1.27e-6
u	Triangular	a=0, b=113, c=42
d	Deterministic	d=7
g	Deterministic	g=9.81

Table 4.1: Summary of MATLAB input

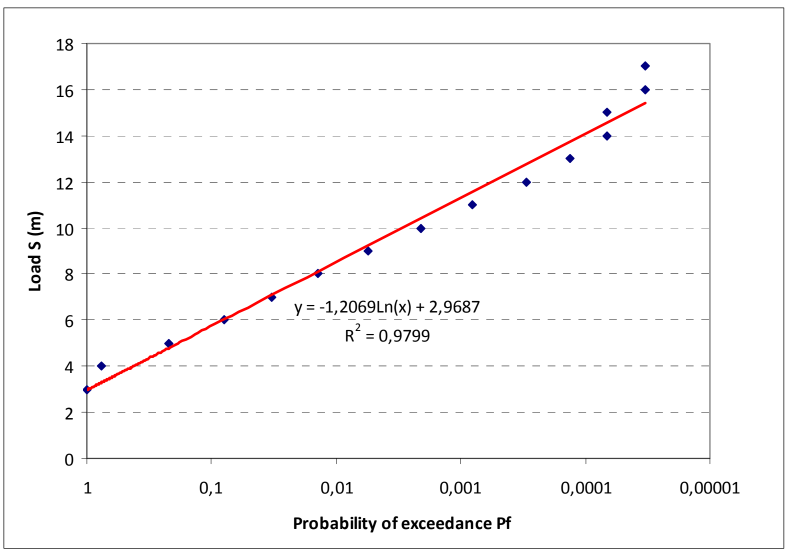

Figure 4.3: Design exceedance curve of total load

4.3.2.2 Characteristic load S^k

If in the above simulation some of the variables are replaced by deterministic values, then the outcome will be a different exceedance curve. This difference is indicative of the degree of uncertainty inherent to those particular variables, which is supposed to be incorporated by the safety factors. Using deterministic values for all the load variables, the total load becomes deterministic too, and the exceedance curve turns to be a straight line parallel to the x-axis. In order to come up with a line that represents the characteristic exceedance curve of the total load, all chosen deterministic values of the variables need to be their characteristic values.

Since there is no standard rule for the choice of characteristic values, but they vary in different designs depending on the overall design approach, a choice for all the load variables is necessary, which can be reasonably substantiated. The most commonly used choices for characteristic values are either mean variable values or values with probability of non-exceedance equal to 95%. For this project mean values are chosen for the majority of variables, which is in accordance to the PIANC report of 1987, Risk Analysis in Breakwater Design. It should be noted that the choice of characteristic values is not critical for the final design. A different set of characteristic values would result in a different set of safety factors, which would always cover the same degree of uncertainty though. This will be further explained later on.

For variables with a normal distribution, the mean values are already known. Apart from them there are four non-normally distributed variables, the tidal elevation h_t, the hurricane speed u, and the hurricane track parameters C and β.

For the tidal elevation, which is Weibull distributed the mean value of its equivalent normal distribution is used, as resulting from the transformation introduced by Rackwitz & Fiessler. This transformation can be easily elaborated in MATLAB and is based on the assumption that the values of the real and approximated probability density function and probability distribution function are equal at the design point. Mathematically this assumption is expressed with the following formulae (A.S. Nowak, K.R. Collins – Reliability of Structures):

$$F(X^*) = \Phi\left(\frac{X^* - \mu_X}{\sigma_X}\right) \tag{4.12}$$

$$f(X^*) = \frac{1}{\sigma_X}\varphi\left(\frac{X^* - \mu_X}{\sigma_X}\right) \tag{4.13}$$

Where, Φ = probability distribution function for the standard normal distribution,
ϕ = probability density function for the standard normal distribution,
μ_X = mean value of the equivalent normal distribution,
σ_X = standard deviation of the equivalent normal distribution
X^* = design point.

The hurricane speed has a triangular distribution with a most probable value randomly chosen equal to 42m/s, which corresponds to a category 1 hurricane. The characteristic value of this parameter depends usually on the acceptable risk levels of the area of interest. In the Gulf of Mexico a hurricane of category 3 is usually considered in the design, and for this reason its characteristic value is chosen equal to 58m/s, which corresponds to category 3.

The remaining variables, which are the parameters of the hurricane track, are uniformly distributed, with mean values equal to zero. The mean values of those two variables do not represent also their most probable value, as all their values are supposed to have the same probability. For this reason a different approach is used for determining their characteristic value. As characteristic hurricane track a hurricane track is chosen that seems to be in accordance with the tendency shown in the hurricane records by NOAA (figure 2.10). According to the records, the most frequent hurricanes in Galveston have directions S-SE-E. Hence a random South-East hurricane track is chosen with parameter values C=5000m and β=-0.75rad.

The chosen characteristic values of the 12 load variables are summarized in the table below. Substituting these values into equations 3.26-3.31 and 4.2, the total load is calculated, S=4.6m and constant in all possible return periods, since no variation of its parameters is taken into account. This means that its exceedance curve will be a straight line parallel to x-axis.

X_i	X_i^k	units
γ_b	0.55	[-]
h_d	7	m
h_t	0.23	m
γ_w	101552.5	N/m^3
p_n	10250	N/m^2
β	-0.75	rad
C	5000	m
c_1	4.00E-06	[-]
u	58	m/s
d	7	m
g	9.81	m/s^2

Table 4.2: Characteristic values of load variables

4.3.2.3 Safety factors

Since the design and characteristic total load have been defined, the safety factors can be easily derived from equation 4.9. As mentioned before, the safety factors account for the uncertainties which are neglected when the design parameters take deterministic values, and literally constitute a measure for the divergence between the characteristic and design exceedance curves (figure 4.4).

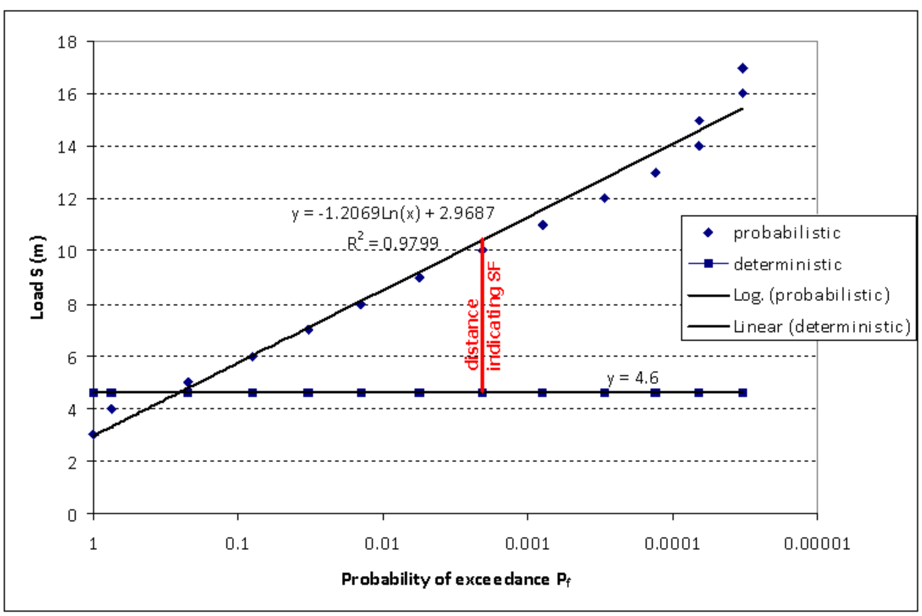

Figure 4.4: Divergence accounted by safety factors

According to the above figure, for very high probabilities the exceedance line of the probabilistic calculation is below the characteristic line, indicating that the presented deterministic design overestimates the load, and therefore there is no need for safety factors in that area.

The derived safety factors for the different probabilities of failure are shown in the following graph.

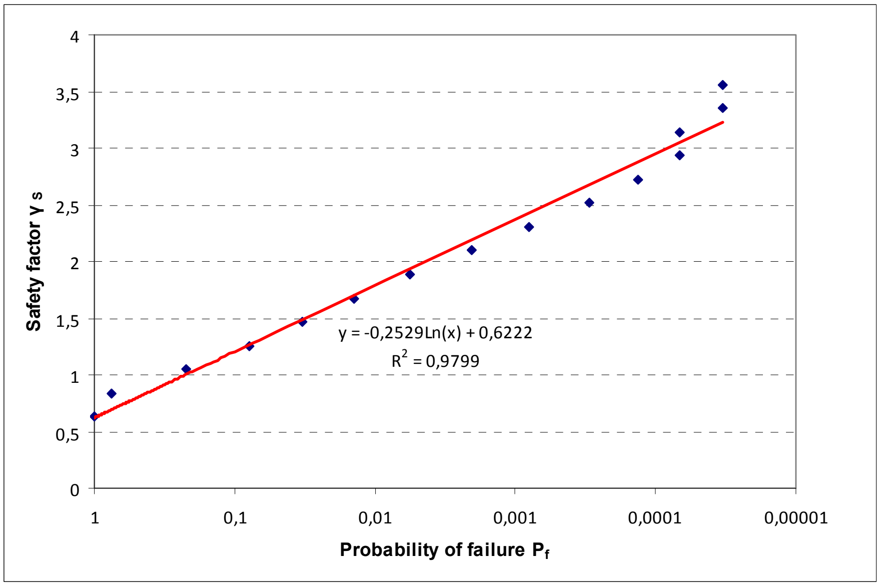

Figure 4.5: Safety factors for different target probabilities of failure

According to the graph, the load safety factor is a function of the target probability of failure. This is a reasonable and expected result, as the distance between the characteristic and design exceedance curves increases when the probability of failure decreases (figure 4.4). This is due to the fact that for lower probabilities of failure a higher degree of uncertainty is incorporated in a fully probabilistic design, and consequently in the resulting safety factors of a level 1 design.

If the characteristic load was defined by the 95% parameter values instead of the mean values, then the total load would most probably get a higher value (figure 4.6). As a result the characteristic exceedance curve would be displaced upwards, and the final safety factors would be lower, so that they keep incorporating the same degree of uncertainty.

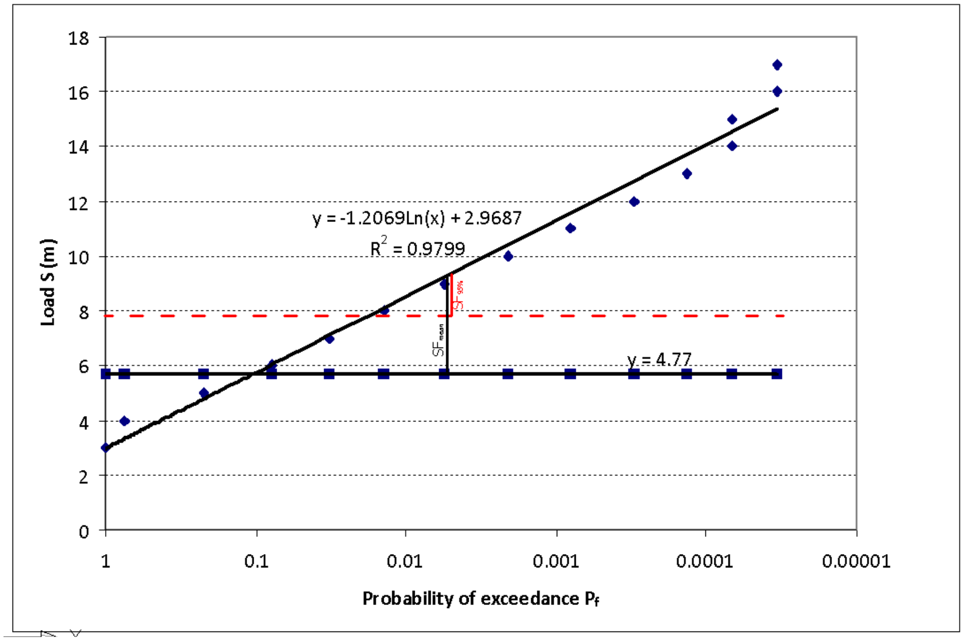

Figure 4.6: Indication of safety factors' variation, given different characteristic load

4.3.3 Safety factor for strength γ_R

For the strength factor a similar approach to the one adopted for derivation of the load factor is suggested. As mentioned before, the strength factor is meant to incorporate uncertainties inherent in the resistance part of the stability formula, which is also the case for the load factor, yet the uncertainties covered by the latter concern the load parameters of the stability formula. Therefore, similarly to the case of load factor, the resistance factor can be defined as the degree of divergence between a characteristic and a design resistance. An important difference between the two approaches is that in the case of resistance, the concept of exceedance curves makes no sense, as all strength parameters are related to the design and construction, and therefore their degree of randomness is a matter of choice.

For this reason, the calculation procedure presented below suggests the derivation of strength safety factors for certain discreet target probabilities of failure. This means that the outcome of this procedure cannot be a continuous line representing the safety factor along a range of probabilities of failure, like in the case of load (figure 4.5). A corresponding graph would depict only discreet spots instead. Moreover the procedure can be applied provided that the design hurricane and the load safety factor have already been specified.

The steps to be followed once the design hurricane and load factor are known are the following:

1. Calculation of total load S, using the design hurricane values for wind speed and hurricane track parameters C and β, and mean values for the remaining load parameters. For parameters that are not normally distributed, the mean values of their equivalent normal distributions are used.
2. Calculation of design load, using the selected load safety factor γ_S:

$$S^* = \gamma_S S \tag{4.14}$$

3. Application of Van der Meer stability formula for calculation of characteristic nominal armour unit mass M_{50}^K, using mean values for all resistance parameters. The definition of mean values is the same as in step 1.
4. Validation of process through a Monte Carlo simulation. The limit state function is the Van der Meer equation, as used in chapter 3 (equation 3.25). All resistance input parameters are deterministic, M_{50}^k for the nominal unit mass and mean values for the remaining parameters. The load parameters are inserted as random variables, with their own distributions, as presented before (table 3.12). The extracted probability of failure has to be equal to the target probability of failure. If this not the case, then step 3 and the total process of load safety factor derivation have to be checked for possible numerical errors.
5. Actual derivation of strength safety factor through a trial and error procedure, applied in a Monte Carlo simulation, with limit state function the equation (4.1). All resistance and load parameters are inserted as random variables, except for the nominal mass, which takes the characteristic value calculated in step 3. The safety factor γ_S is inserted as constant and its value is already known, while the strength safety factor γ_R is the trial parameter. Starting with γ_R=1, a probability of failure is calculated, which is higher than the target probability of failure. The simulation is repeated with gradual increase of γ_R until the target probability of failure is reached.

Once the strength safety factor is derived, the design nominal mass can be easily calculated as follows:

$$M_{50}^* = \gamma_R M_{50}^k \qquad (4.15)$$

4.3.4 Example

The above calculation procedure is applied in Galveston in order to derive a pair of safety factors for the design of Galveston jetties. The analytic calculation steps are presented below:

1. *Definition of load exceedance curve*
   The total load exceedance curve has already been defined in paragraph 4.3.2.1 and can be seen in figure 4.3.

2. *Definition of design load for the target probability of failure*
   The target probability of failure is 15% during lifetime, which corresponds to a load with return period 225 years, which is a load with a probability of exceedance per year $P_{f,target}$=0.0044. According to the load exceedance curve, this value corresponds to S*=9.52m.

3. *Definition of design hurricane*
   As design hurricane, the one used also in paragraph 4.3.2.2 is chosen: u=58m/s (category 3), C=5000m, β=-0.75rad.

4. *Calculation of characteristic load*
   The values of table 4.2 are substituted into equation (4.10): S^k=4.6m.

5. *Calculation of load safety factor*

$$\gamma_S = \frac{S^*}{S^k} = \frac{9.52}{4.6} = 2.07$$

6. *Calculation of design load based on load safety factor*
$S_{design} = \gamma_S S^k = 2.07 \cdot 4.6 = 9.52$

7. *Calculation of characteristic nominal unit mass*
The characteristic nominal unit mass is calculated with the Van der Meer equation, when characteristic values are substituted into all inherent parameters. The characteristic load has already been calculated in step 4. For the characteristic strength, mean values of all strength parameters are substituted into the stability formula. Those values can be seen in the table below:

X_i	X_i^k	units
c_{pl}	8.4	[-]
P	0.4	[-]
S	5	[-]
N	6063	[-]
$s_{m-1,0}$	0.041	[-]
cotα	2	[-]
ρ_s	2650	[kg/m^3]
ρ_w	1030	[kg/m^3]

Table 4.3: Characteristic values of strength parameters

$$R^k - \gamma_S S^k = 0 \Rightarrow 4.117 d_{n50}{}^k - 9.522 = 0 \Rightarrow d_{n50}{}^k = 2.32m$$

$$M_{50}{}^k = \rho_S d_{n50}^{k}{}^3 = 2650 \cdot 2.32^3 = 33091kg$$

The extracted nominal mass has a very high value. A rock unit of 33 tones cannot be produced and therefore cannot be used in the design of a rubble mound structure. For the design process this high value means that the use of artificial concrete units should be considered, as artificial units of this size can be produced and transported at the construction site. This means though that also the design stability formula has to be appropriately altered, and consequently the whole process for determination of safety factors should be repeated. Since the scope of this example is just to present the analytical steps for deriving safety factors, and not to come up with an actual design, the process will not be repeated for artificial units, but will be finalised for rock units.

8. *Validation of process*
Running a Monte Carlo simulation with M_{50}=33091kg, deterministic values for all strength parameters from table 4.2, and probabilistic values for all load parameters from table 3.12, a probability of failure P_f=0.004 is calculated, which is almost equal to the target, $P_{f,target}$=0.0044. This result is considered satisfactory, and therefore the calculation of the load safety factor is assumed to be correct.

9. *Determination of strength safety factor with trial and error procedure*
A new Monte Carlo simulation is run with the following limit state function:

$$Z = \gamma_R R - S \tag{4.16}$$

This time the strength safety factor γ_S takes deterministic values, the unit mass M_{50} is deterministic and equal to 33091kg, while the remaining strength and load parameters take probabilistic values from table 3.12. Assuming initially γ_R=1, a probability of failure is extracted P_f=0.0051, which is smaller than the target. By increasing gradually the strength factor, the target is reached for γ_R=1.47.

$$\gamma_R = 1.47 \rightarrow P_f = 0.0044 = P_{f,target}$$

The derived value for the strength safety factor proves to have a quite high value, as it causes an approximately 50% increase of the design resistance, while it is only 30% lower than the load safety factor. One would normally expect a larger divergence between the two factors, as the variations of the strength parameters, which are connected with the construction process, should be much smaller than the variations in the load, and therefore also the uncertainty inherent in the strength should be lower than that of the load. Nevertheless this does not count in the examined case, due to the chosen safety format. The stability formula used for the design is a Van der Meer equation, which does not separate the load and resistance parameters in its two terms. This structure has been also kept in the safety format, which suggests a limit state function whose resistance term contains also three load parameters, the wave steepness, the number of waves and parameter c_{pl}. The variation of these three parameters is considered to be the chief cause of that high value of the strength safety factor. This will also be confirmed in the next paragraph, where sensitivity analysis is performed.

10. *Calculation of design unit mass*
This step is in fact part of the design process and not of the determination of safety factors.

$$M_{50}* = \gamma_R M_{50}{}^k = 1.47 \cdot 33091 = 48644kg$$

The resulting design unit mass is equal to the one extracted by the Monte Carlo simulation of the previous chapter (see figure 3.11). This is an expected result, and validates the fact that the new set of safety factors results in a design equally reliable to a fully probabilistic design.

4.4 Sensitivity analysis

As mentioned before, the final values of the safety factors depend to an important extend on the designers' choice of characteristic values. For the load parameters it is common that the designers' choice represent either a load case that seems to be the most unfavourable for the structure, or the most probable to occur. In particular when the design conditions are determined by highly stochastic phenomena like hurricanes, it is more likely that the design hurricane will be one that seems to have the most frequent direction and its intensity is reasonably high according to the acceptable risk levels of the area of interest. In the Gulf of Mexico for instance, a reasonable choice is a category 3 hurricane. In any case, the safety factors will be appropriately altered so that the degree of safety they incorporate is always the same as defined in the safety format.

As the final choice of safety factors depends on the designers' critical view and it is not a unique number, it is interesting to see how sensitive their values are in the variations of the design parameters, and particularly the ones that mostly affect the final probability of failure. The degree that each variable affects the probability of failure can be indicated through their sensitivity factors, which can be derived from a level 2 probabilistic calculation of the armour layer stability. This calculation can be elaborated in MATLAB with simulation of a first order reliability method (FORM).

4.4.1 Sensitivity factors

The limit state function and all input parameters are identical to the ones used for the Monte Carlo simulation (equation 3.25, table 3.12). The extracted sensitivity factors for all the problem variables as well as the degree that they affect the final probability of failure are presented in the following table.

	X_i	α_i	Effect (%)
Resistance R	c_{pl}	0,252	9,52
	P	0,022	0,83
	S	0,000	0,00
	N	-0,203	7,66
	$s_{m-1,0}$	0,183	6,92
	$\cot\alpha$	0,062	2,33
	ρ_s	0,184	6,95
	ρ_w	-0,020	0,74
	M	0,046	1,74
Load S	γ_b	-0,220	8,30
	h_d	0,000	0,00
	h_t	-0,048	1,80
	γ_w	0,000	0,00
	p_n	0,000	0,00
	β	0,008	0,30
	C	-0,649	24,49
	c_1	-0,197	7,42
	u	-0,557	21,00
	d	0,000	0,00
	g	0,000	0,00

Table 4.4: Sensitivity factors of problem variables

According to the above results, the probability of failure of the structure is determined in an exceptionally high degree by two load variables, the hurricane speed u, and the hurricane track parameter C, whose contribution to the final result is 21% and 24.5% respectively. Although those two parameters are critical for the determination of the total load, and therefore of the structure stability as well, they have both been less accurately defined. The characteristic value of parameter C is randomly chosen equal to 5000m, while for the hurricane speed a triangular distribution has been assumed whose most probable value is randomly selected equal to 42m/s. For this reason a sensitivity analysis is chosen to be performed based on variations of those two parameters.

The above extracted sensitivity factors verify also the reasoning behind the derived high value of the strength safety factor. According to table 4.4, among the strength parameters, the most critical effect to the final probability of failure is caused by the combination of the three load variables present to the resistance term. Hence the high value of the strength safety factor can be explained by the existence of the three load parameters in the resistance term. This statement is not further elaborated in the sensitivity analysis.

4.4.2 Variation of hurricane track parameter C

This parameter takes values from -10 to 10 kilometres. Safety factors are calculated for

$$C = [-10, -7.5, -5, -2.5, 0, 2.5, 5, 7.5, 10] \tag{4.17}$$

The resulting safety factors for some standard probabilities of failure are presented in the table and figure below. The results are presented for a hurricane speed u=58m/s, which corresponds to a category 3 hurricane.

C \ P_f	0.10	0.01	0.001	0.004	0.0001
-10000	1.298	1.887	2.520	2.713	2.982
-7500	1.347	1.959	2.616	2.817	3.095
-5000	1.400	2.037	2.720	2.929	3.219
-2500	1.386	2.016	2.692	2.899	3.186
0	1.334	1.940	2.590	2.789	3.065
2500	1.285	1.869	2.496	2.688	2.954
5000	1.240	1.803	2.408	2.593	2.850
7500	1.198	1.742	2.327	2.505	2.753
10000	1.159	1.685	2.250	2.423	2.663

Table 4.5: Load safety factor for various C and u=58m/s

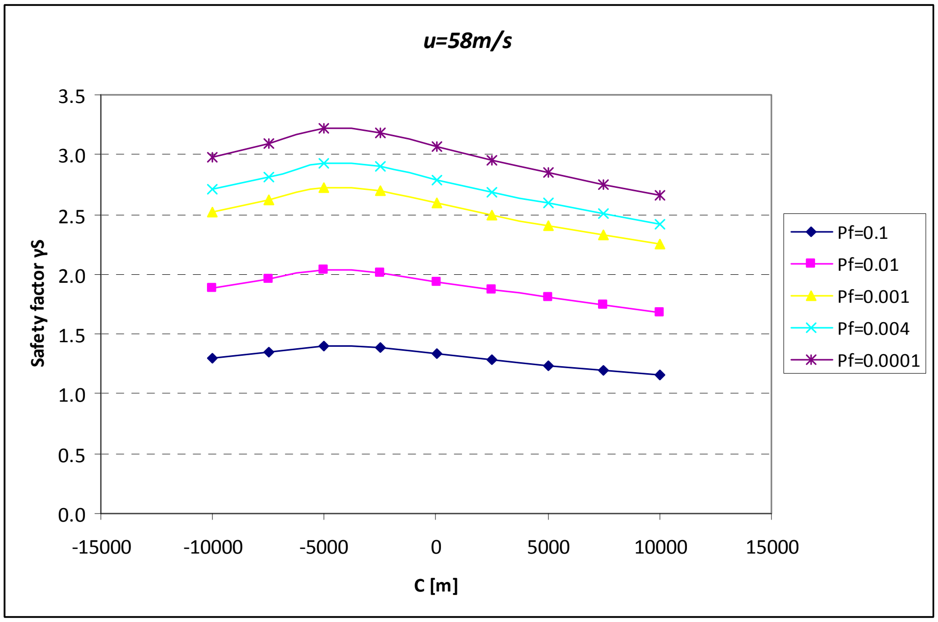

Figure 4.7: Load safety factor for various C and u=58m/s

As expected, figure 4.7 confirms that for higher target failure probabilities, the safety factor becomes lower. It is also perceptible that the higher values of the safety factor occur when the hurricane track parameter C takes values around -5000m, and it gets lower as parameter C increases from -5000 to 10000m. This implies that according to the developed model, the least unfavourable load case occurs when C=-5000m, which represents a hurricane with landfall 5 kilometres SE of Galveston jetties, while the worst case load occurs for C=10000m. This latter information is not critical for the design though, since the degree of safety incorporated by the safety factors is independent of the chosen design conditions.

A reverse representation of parameter C and probability of failure can be seen below. Based on the following graph, the logarithmic correlation between the safety factor and the target probability of failure is confirmed also for various values of parameter C. An approximate correlation line is depicted in figure 4.8. What can also be perceived is that there is a vertical spread of the safety factor values as the target probability of failure decreases. This spread indicates that for lower probabilities of failure, the effect of parameter C is more important. For instance in a 1/10000 years design the safety factor takes values from 2.6 to 3.4.

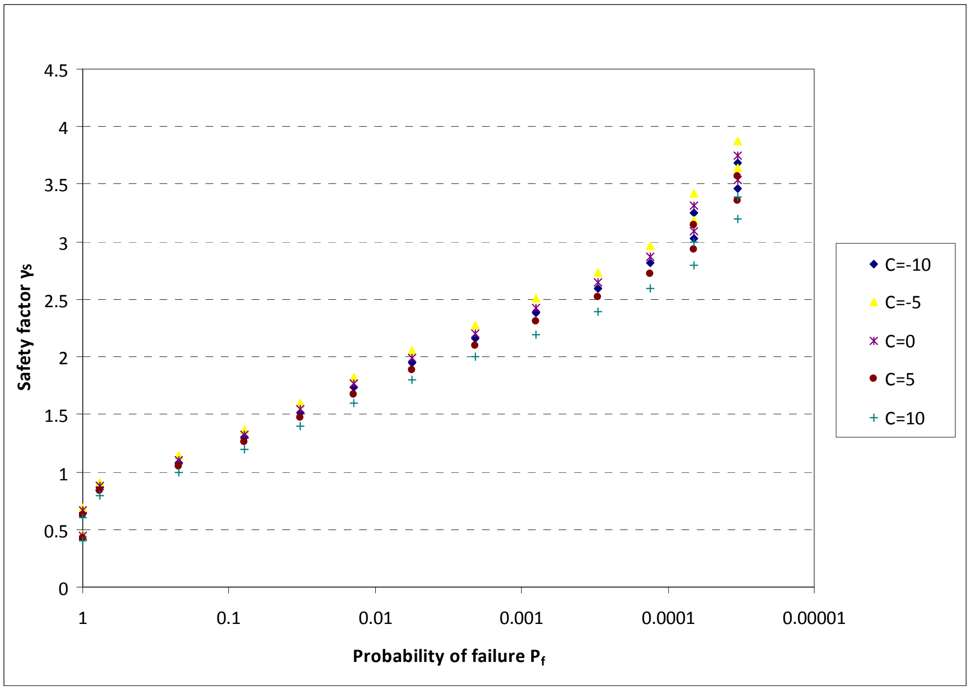

Figure 4.8: Safety factors for variation in parameter C

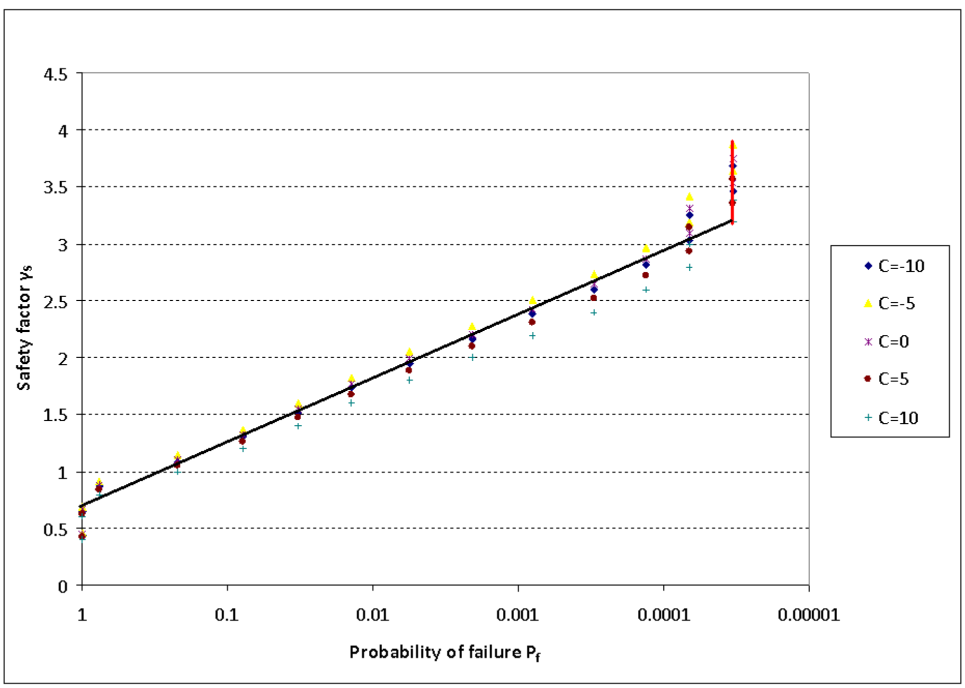

Figure 4.9: Safety factors' approximate function and vertical spread

4.4.3 Variation of hurricane speed u

The hurricane wind speed takes values from 0 to 113m/s, and as most probable value the upper limit of category 1 hurricanes has been considered, u=42m/s. Safety factors are calculated for all the boundary speeds of hurricane categories as well as of the general upper boundary.

$$u = [42, 49, 58, 69, 80, 113] \tag{4.18}$$

The resulting safety factors are presented in the following table and graph. The results are presented for a hurricane track parameter C=5000m.

u \ P_f	0.1	0.01	0.001	0.004	0.0001
42	1.326	1.928	2.575	2.773	3.047
49	1.290	1.876	2.506	2.698	2.965
58	1.240	1.803	2.408	2.593	2.850
69	1.174	1.708	2.281	2.456	2.699
80	1.106	1.609	2.148	2.313	2.542
113	0.904	1.315	1.756	1.891	2.078

Table 4.6: Safety factors for various hurricane speed u, and C=5000m

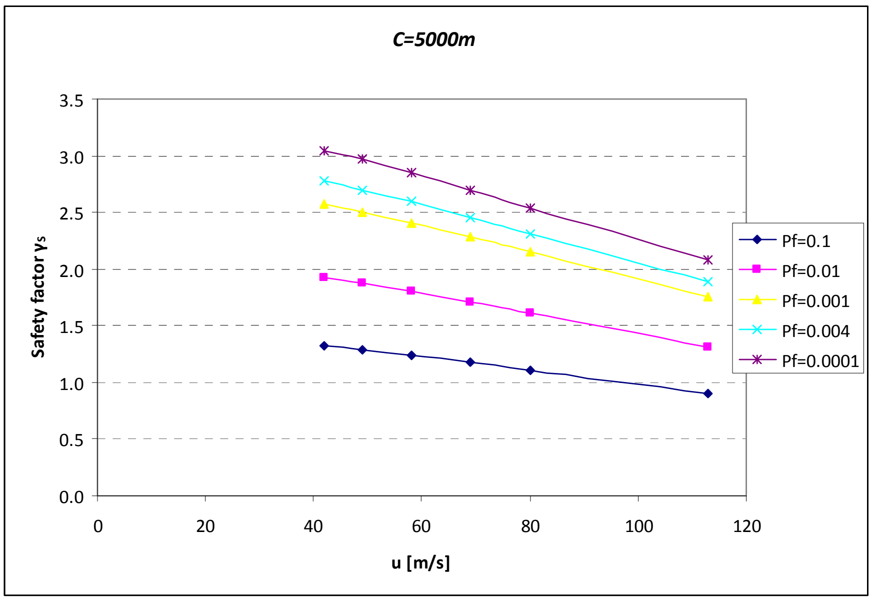

Figure 4.10: Safety factors for various hurricane speed u, and C=5000m

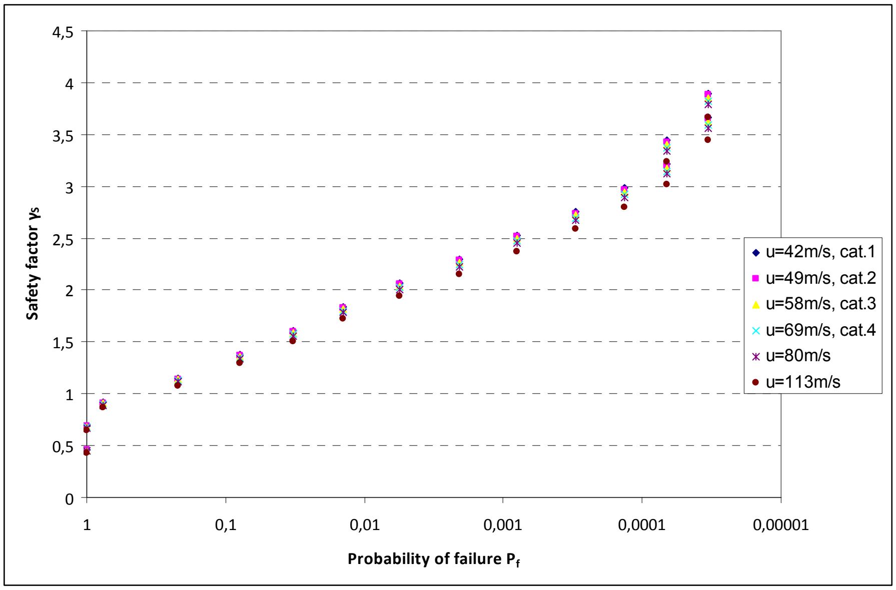

Figure 4.11: Safety factors for variation in hurricane speed u

The conclusions for variation of the wind speed are similar to the ones for various values of parameter C. The only difference is that the vertical spread of the safety factor that occurs for lower target probabilities of failure is less important in case of wind speed variations. For a 1/10000 years design the safety factor varies from 3.04 to 3.24, which is a divergence about 50% smaller than the case of C variations.

4.4.4 Variation of both hurricane speed and parameter C

In order to get a broader insight of the results and facilitate the design process, it is handy to have an overview of the safety factors when both hurricane speed u and parameter C vary. This simultaneous variation can be represented for a certain probability of failure. Therefore, once the target probability of failure is known to the designers, they can get the right value of the load safety factor γ_S directly from a graph for the respective failure probability. In the following graph the load safety factor for a target probability 1/225 is represented.

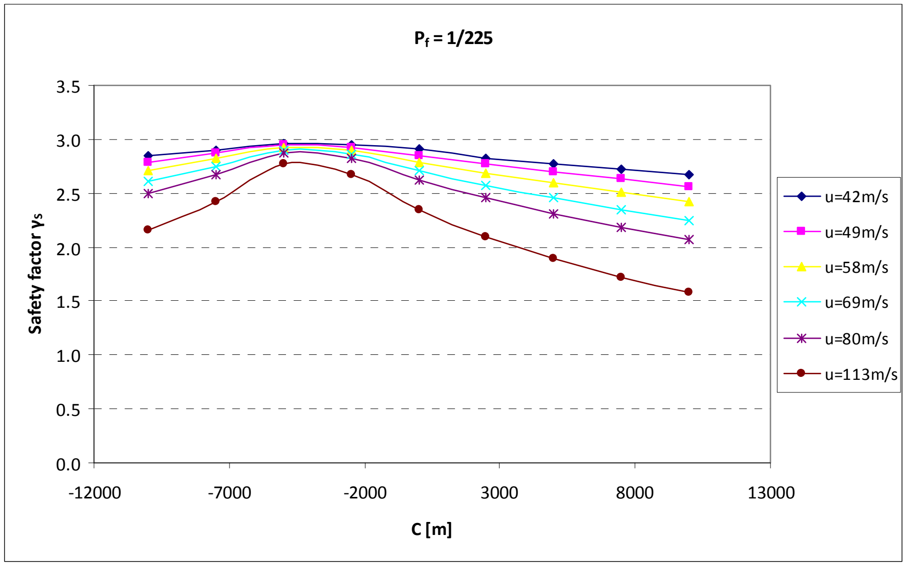

Figure 4.12: Safety factors for C and u variations and target failure probability 1/225

According to the graph, for C=-5000m and for all hurricane speeds the highest safety factors are extracted. Moreover for higher hurricane speeds the safety factor becomes lower. This result was indeed expected, since it implies that when a hurricane of higher intensity is considered, then a safer design is realised, and consequently a lower safety factor is sufficient for reaching the target safety level.

4.5 Evaluation of results

4.5.1 Conclusions on the results

Based on the sensitivity analysis, a number of conclusions have been drawn, which are summarised as follows:

- The load safety factor is mostly affected by variations of the design hurricane, and particularly of the hurricane speed u and the hurricane track parameter C.
- The main cause of the high value that the strength safety factor takes is the existence of three load parameters in the resistance term as defined with the safety format.
- The load safety factor is a logarithmic function of the target probability of failure.
- For lower probabilities of failure, the results become more sensitive to variations in hurricane speed and parameter C.
- The highest safety factors are extracted for C=-5000m.
- The higher the design hurricane speed, the lower the safety factors.

4.5.2 Final choice of safety factors

The designers' choice of safety factors varies according to the design conditions that are taken into account, and particularly the design hurricane, as can be seen in figure 4.12. The whole design process could be further simplified if for a certain area fixed values of safety factors could be used, independently of the design hurricane. However this choice would mean that not the same degree of uncertainty is incorporated in the set of safety factors. Hence the fixed values should be chosen in a way that the minimum required safety is incorporated in any case. This would only be possible if the highest values of safety factors are chosen, according to the graphs of the previous paragraph, yet there will always be a risk that the safety factors result in a conservative design. For this reason it is recommended that the designers choose the appropriate safety factors from graphs themselves, according to the design conditions that they use, and no fixed safety factors are adopted for a certain region.

4.5.3 Accuracy of results

The accuracy of the extracted results is directly associated with the accuracy of the performed Monte Carlo simulations, which depends on the number of samples that are used each time. An acceptable accuracy is considered to be reached when the number of used samples fulfils the following condition *(TU Delft, Probability in Civil Engineering):*

$$n > 400\left(\frac{1}{P_f} - 1\right) \qquad (4.19)$$

Where n = number of samples
P_f = probability of failure, i.e. the probability that the limit state function becomes negative ($P_f = P(Z < 0)$).

According to the above rule, the minimum number of samples required in order to come up with acceptable results for different target probabilities of failure is presented in the table below.

P_f	n
0,1	3600
0,01	39600
0,0044	90509
0,001	399600
0,0001	3999600

Table 4.7: Minimum number of samples for different target probabilities

For all Monte Carlo simulations in this project 30000 samples have been used, which give satisfactory results only for designs with very high probabilities of failure. This choice was made only for practical reasons, and particularly due to time limitations, since a small increase in the number of samples results in a much longer simulation time. For this reason for a real design it is recommended that the analysis is repeated, using the appropriate number of samples in the simulations.

4.5.4 Generalization of results

As mentioned before, the above analysis is based on a level 3 fully probabilistic design, for which a model describing the physical problem has been created. This model is site-specific, since it is developed especially for the area around Galveston. As a consequence also the results of the above determination of safety factors are site-specific, and therefore only useful in Galveston. If this analysis was generic and could be used also in other areas with the same general characteristics, then it would certainly be more valuable. In order to transform this site-specific procedure into a generic one a validation and if necessary, reconsideration of some model aspects is needed.

Three main aspects can be identified in the developed model, the strength of the structure, the nature of the load, and the examined area. All the problem variables and assumptions are related to one of them. The only aspect of interest for generalization of the site-specific model is the examined area. In particular all assumptions and considerations that are related to the bathymetry and topography of the area must be reconsidered. They are the following:

1. Assumption of straight coastline
2. Range of hurricane track parameter C
3. Coordinates of design spot (x_1,y_1)
4. Depth below mean sea level.

4.5.4.1 Assumption of straight coastline

This assumption is related to the structure of the model. Based on this, a Cartesian system has been chosen on which the hurricane track has been defined as a straight line. The range of application of the model is therefore limited to areas where the coast can be assumed a straight line. If this is not possible, then the model should be rearranged in a way that is suitable for the respectively examined site. Depending on the new site, the new model concept can be similar to the one of Galveston, or follow totally different principles. In any case, it should be ensured that the model contains only independent variables.

4.5.4.2 Range of hurricane track parameter C

The range of parameter C is an input value in the model, and therefore it is not directly connected to the structure of the model. This is a consideration to be made once the validity of the Galveston model in the examined site has been confirmed with the assumption of a straight coastline.

However this consideration is indirectly related to the structure of the model as well. As mentioned in a previous chapter, if the range of parameter C is very wide, then there are some model errors that decrease the accuracy of the simulation. Those errors are related to the definition of the circular hurricane fetch. First of all, the assumption of constant water depth is cancelled when the range of parameter C is very large. Second, for hurricane tracks that are close to the boundaries of the examined area, the circular fetch cannot be determined in the same way as indicated in the model. This is always the case, no matter how wide or narrow the range of parameter C is. If the range of parameter C increases then the number of hurricane tracks that the fetch calculation is incorrect increases too. As a result, the inherent simulation error increases, and it might get unacceptable values. The limits in which this error is considered acceptable, as well as the way to reduce this error are an issue for discussion, which is though not further elaborated in this project.

4.5.4.3 Coordinates of design spot (x_1, y_1)

In accordance to parameter C, the coordinates of the design spot are not related to the structure of the model. They could also be an input value, but they have been inserted as constants in the definition of the distance r_s between the hurricane track and the design spot instead (equation 3.29). Those constants need to be appropriately altered in the new site.

4.5.4.4 Depth below mean sea level

The depth below mean sea level needs to be determined separately at the design spot and the broader area around Galveston. These two depths are represented by two variables in the model, which in the case of Galveston take deterministic values. For a different site they should be determined by a certain distribution and its parameters. Particularly for the water depth in the broader region, it might be necessary to divide the examined area in smaller parts, and determine the water depth separately in each part. This would be necessary in cases with very steep bottom slopes, or with bottom incontinuities.

5. CONCLUSIONS

Based on the above presented research a large number of conclusions and concluding remarks have been drawn, which are summarised in the following paragraphs, with respect to the various issues that have been examined

5.1 Hydraulic boundary conditions

1. In order to come up with the design load, both short waves and swells have to be taken into account, for the cases of a normal storm and a hurricane attack. The consideration of swells is necessary because their action imports a considerable amount of energy in the total wave spectrum.
2. Due to the combined action of short waves and swells, the total wave spectrum in Galveston is a double-peak spectrum, and therefore the design conditions should be described by this. However this combined action is neglected in the determination of hydraulic boundary conditions and in the design, and one-peak spectra are considered instead. This assumption might introduce some error to the final design. For this reason the design results should be tested with modelling.
3. The determination of storm waves is based on local wave data provided by Global Wave Statistics and Argoss. The two data series are considered equally reliable, hence average wave values are used. However there is a large divergence between the two series, meaning that their reliability is questionable, and therefore they import a high degree of uncertainty in the design.
4. The determination of hurricane waves is based on a 160-year hurricane record provided by NOAA. This record gives information on the one-minute sustained wind speed. The consideration of this wind speed results in an overestimation of the hurricane waves. More realistic results could be extracted if a three or six-hour sustained speed was considered.
5. The jetties of Galveston are located in relatively shallow water, meaning that the structure is attacked by breaking waves. Hence shallow water effects have to be taken into account in the determination of the design load. Once the deep water wave characteristics are determined, the shallow water waves can be derived through wave modelling.

5.2 Design

1. The critical design conditions are determined by hurricane-induced short waves.
2. The design of PIANC results in extremely high values for the armour unit mass. This is due to the fact that depth-limitations in the wave height are not taken into account in the method, and therefore incorrect results are extracted in case of shallow water designs, like Galveston jetties.
3. A fully probabilistic computation is the one that deals explicitly with the majority of uncertainties inherent to the design, and therefore it is assumed to be the most reliable design method.
4. The results of the classical and PIANC design can be evaluated through the developed probabilistic model, by computing the corresponding probabilities of structural failure. Based on this computation the classical design proves to be insufficient, while the PIANC design conservative, as the corresponding probabilities of failure during lifetime are 96% and 10% respectively, while the target probability of failure is 20%.

6. The very poor results of the classical design can be the cause of two things. First is the different definition of load in the classical and fully probabilistic design. For the deterministic design the wave height in $H_{2\%}$ as resulted from SwanOne simulations has been used as load, while for the probabilistic design the load is approximated as a percentage of the local water level, which is an approximation valid for shallow water conditions. Second is the random choice of a design hurricane in the deterministic design, which might be far from the most unfavorable hurricane case. This is always a risk inherent to a deterministic design, when the most unfavorable load cannot be easily estimated.

5.3 Probabilistic model

1. For a fully probabilistic computation, special attention should be paid to the problem parameters, as the developed model should only consist of uncorrelated variables.
2. In the developed probabilistic model the design is approximated to a percentage of the local water level. This consideration allows for a more explicit determination of the effect of a hurricane pass to the design.
3. The type of extracted probabilities depends on the type of input that is used. If the inserted distribution functions refer to probabilities per year, then the final probability of failure will also be per year. If the input distributions refer to probabilities during lifetime, then the output will be a failure probability during lifetime of the structure.
4. There are some model errors introduced when the range of parameter C increases. Those errors are related to the definition of the circular hurricane fetch. First of all, the assumption of constant water depth is cancelled when the range of parameter C is very large. Second, for hurricane tracks that are close to the boundaries of the examined area, the circular fetch cannot be determined in the same way as indicated in the model. This is always the case, no matter how wide or narrow the range of parameter C is. If the range of parameter C increases then the number of hurricane tracks that the fetch calculation is incorrect increases too.

5.4 Suggested design approach

1. A new design method is suggested which is based on the concept of safety factors. The main goal is to develop a design equally reliable to a fully probabilistic design on level 3. This can be achieved with the definition of a set of safety factors that deals explicitly with all uncertainties inherent to the physical problem and the design itself.
2. The rationalization of the new safety factors is based on a concept suggested by Ciria in 1977, yet a modification has been made in order to increase the reliability of the results. In the new approach a level 3 probabilistic calculation is used as basis for deriving the safety factors instead of a level 2 that Ciria uses.
3. The suggested safety format indicates the use of only two safety factors, one for the total load and one the strength, which will incorporate all uncertainties inherent to the load and strength variables.
4. The analytic steps for deriving the set of safety factors constitute a generic procedure, which can be used as a guideline for code makers. However, since the used probabilistic model is site specific, the resulting safety factors will also be site specific.
5. In order to derive safety factors in a new location, a validation and if necessary, reconsideration of some model aspects is needed, and particularly of assumptions and considerations related to the topography and bathymetry of the examined area.

5.5 Values of safety factors

1. The uncertainty inherent to the load is much higher than the uncertainty of the strength. For this reason one would expect the load safety factor to be much higher than the strength factor. The difference between the two factors is less important than expected though. This is due to the chosen safety format, which doesn't make a clear distinction between the strength and load parameters. In particular there are three load variables included in the strength term, which are the main cause of the strength factor increase.
2. The load safety factor is mostly affected by variations of the design hurricane, and particularly of the hurricane speed u and the hurricane track parameter C. For different values of hurricane speed and parameter C, different values of load factor are extracted.
3. The load safety factor is a logarithmic function of the target probability of failure.
4. For lower probabilities of failure, the resulting load factors become more sensitive to variations in hurricane speed and parameter C.
5. The accuracy of the extracted safety factors is directly associated with the accuracy of the performed Monte Carlo simulations. The accuracy of the performed simulations is only sufficient for very low probabilities of failure, and therefore the extracted safety factors are not accurate enough for use in a real design.

6. RECOMMENDATIONS

This project constitutes a critical view to the design practice of breakwaters, while it also introduces a new probabilistic design approach. In all stages of the performed analysis a number of simplified assumptions have been made, many of which have not been validated, or proved to be incorrect. As a consequence, the reliability of the extracted results and the follow-up conclusions can be questioned. An optimization of the total analysis is therefore necessary, which can be achieved through a validation of the assumptions made, and reconsideration of all the weak points that limit the value of the overall outcome. The actions that are recommended with respect to the core parts of the performed analysis are presented below.

6.1 Hydraulic boundary conditions

1. The choice of storm duration at the design area needs to be further elaborated, while it would be interesting to check how much this choice affects the extracted hydraulic boundaries and the final design.
2. A redefinition of the wave types is necessary, in a way that no distinction is made between short waves and swells. The combined action of the two waves should be considered instead, so that the design load is defined by the total wave energy.
3. The existing data for normal storm waves introduce an important degree of uncertainty to the design. It is therefore important that this uncertainty is quantified and taken into account in the design process, if necessary.
4. The hydraulic boundaries occurring during a hurricane pass need to be redefined by taking into account a wind speed averaged over 3 to 6 hours, instead of a one-minute sustained wind speed.

6.2 Design

1. The design wave height has been determined in a different way in the classical deterministic design and in the fully probabilistic design. In the classical design it was derived through a SwanOne simulation, while in the fully probabilistic design it was approximated as a percentage of the local water level. Since the classical design resulted in a very poor result, it is interesting to investigate what the effect of the different determination of design load is in the final result. This investigation could also give some valuable insight into the definition of the breaker index.
2. The determination of variable distributions for the fully probabilistic computation has been based partly on statistical data and partly on the analyst’s critical view. In many cases there was more than one distribution seemingly appropriate for description of a parameter. It is therefore important to investigate the effect of considering different distributions on the design results through a sensitivity analysis.

6.3 Probabilistic model

As mentioned before, there is an error introduced in the model due to parameter C, whose magnitude is expected to increase when the range of parameter C increases. However its actual magnitude and the degree that it affects the final results have not been defined. This definition is important in order to conclude which is the maximum range of parameter C for an acceptable error value. It would also be interesting to investigate what type of modifications could minimize this error.

6.4 Suggested design approach

1. A validation of the new design approach is necessary. This can be achieved by deriving safety factors in a different shallow, hurricane-prone area through application of the guideline for code makers.
2. A concretization of the guideline for designers is necessary. This guideline has only been briefly presented due to the fact that a sensitivity analysis concerning the strength safety factor is missing, which would clarify which are the most appropriate parameters that the choice of the strength factor should be based on.

6.5 Values of safety factors

The above mentioned validation concerns the procedure for deriving safety factors. The designers could only benefit from this research, if safety factors existed in all possible design areas. For this reason, a step further stands the creation of a world database with appropriate safety factor values depending on the design conditions as specified by the designer.

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