


Integral coastal defence for Malecón of Havana, Cuba

Multidisciplinary project: MDP249, main report incl. appendices

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I. Preface

The report is written by a group of four students from the Delft University of Technology (DUT) for a Multidisciplinary Project as part of their Civil Master programme in Havana, Cuba. The project is executed at the Instituto Superior Politécnico José Antonio Echeverría in Havana, Cuba (CUJAE) under supervision of Professor L. F. Córdova. The project group consist of students from the master track Construction Management and Engineering, Hydraulic Engineering and Structural Engineering.

The aim of the project is to find an integrated solution for the Malecón Coastal defence system using studies from previous years performed at the Centre for Hydraulic Investigations of the CUJAE in cooperation with the DUT students. The project took place over a period of 8 weeks for which we would like to thank Professor Córdova for his guidance and expertise in the subject. From the DUT we would like to thank professors M.G.C. Bosch-Rekveldt, H.J Verhagen and Y. Yang for their assistance during the project.

We would like to emphasize that this report is written by students for a master programme of the DUT and that this is not a consultancy report.

12th of January 2018,

Havana, Cuba.

Johan Jansen, Luca Lopriore, Frank Vester and Max van Lambalgen

II. Summary

The objective of this project is to develop a sea defence system for the Malecón of Havana, Cuba, using all previous studies and proposed solutions in the process. The main objective is to prevent unacceptable flooding of the city and damage to the seawall itself. The scope of this project is a 6 kilometre stretch of the Malecón, from Calle 12 till La Punta. This part of the Malecón is divided in 4 sections, using the division made by Professor Córdova.

From previous studies and a site visit to the Malecón itself the current situation is assessed. Valuable data such as the height of the wall, existence of the berm and structural integrity are obtained from this analysis and are used later in the project. The general conclusion of this assessment is that the wall will require repairs or replacement in most sections, especially section 2 and partly 3.

In order to present a solution that is supported by the different parties involved a stakeholder analysis is made, each stakeholder is evaluated and rated in power and interest in the project. Critical actors are determined and described, after which an engagement plan is made for all stakeholders.

As a basis for the design cycle, boundary conditions and criteria are derived from previous studies or formulated from new information. The list is presented and gives boundary conditions for the project and the solutions that may be applicable. Boundary conditions like the amount of wave overtopping (0.05 m^3/s/m) and the limited increase of wall height (max 1.25m from street level) present significant challenges for the design proposals.

Following previous studies, a marine data analysis is made which gives the required input for modelling in SWAN and ANSYS. The SWAN models are used to make computations of the wave climate in front of the wall, after which the significant wave height, wave setup and water elevation are used to calculate the wave overtopping at the Malecón seawall. With the finite element program ANSYS the wall to be constructed is modelled and used to determine governing stresses. The critical values for the stresses in the wall and dowels will be used to produce the detail design of structure.

In order to come up with solutions for the wave overtopping problem, four alternatives are proposed: an economic one, a critical one, an alternating one and a combination. Based on the technical criteria, the social criteria and the costs a multi criteria analysis is made, where after the combination option is advised. This option contains four breakwaters with a total length of 2 kilometers, several berms and a curved wall over the entire section. A detailed design is made of the curved walls, berms and breakwaters, and the wave overtopping is calculated in the new situation.

Figure 1 Overview of the proposed solution

The costs of the proposed solution are relative high, 1.1 billion CUC, for Cuban norms. So it is questionable if this option is feasible. This is mainly due to the high costs of breakwaters causing for an exponential increase in costs for a higher level of safety. Therefore phasing will play an important role as part of the solution. A curved wall will decrease the overtopping with 45% on average, so implementing this as a first step is good option to later expand on with further investments for berms and breakwaters.

Table 1 Overtopping reduction with proposed solution

Section	Existing overtopping m^3/sec/m	After measures m^3/sec/m	Reduction in %
2	1,11	0,12	89,28
3	0,56	0,22	61,17
4	0,48	0,11	77,62
5	0,46	0,12	73,23

When applying the proposed solution, the reduction in wave overtopping on average is 77.3% over the entire Malecón. The demand of 0.05 m^3/s/m is not met in this design, as it was only possible to reach this value when implementing a breakwater at the full length of the Malecón, which is undesirable and too expensive.

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1. Introduction

The aim of this investigation is to propose an integral solution for coastal defence of the Malecón seawall which protects the coast of Cuba's capital, Havana. The investigation is part of a multidisciplinary project which involves four Masters Students from various disciplines within the faculty of Civil Engineering from the Delft University of Technology, namely Hydraulic, Structural and Construction Management Engineering. The project is part of an ongoing and longstanding cooperation between the DUT and Havana's technical university, the CUJAE, of which the Centre for Hydraulic Investigations (CIH) provided the main objectives.

The project is focused on a 6 km stretch of the Malecón seawall beginning at La Chorrera, a castle at the mouth of the River Almendares (East) and ending at Castillo de la Punta at the entrance of the harbour (West) as shown in Figure 2.

Figure 2: Overview of the study area (Google Earth, 2018)

The primary function of the Malecón is coastal defence and during extreme weather conditions, such as frequent cold fronts and regular Hurricanes, wave overtopping represents a significant problem for inhabitants and governmental institutions. Flooding causes significant damage to property several hundred meters inland and hinders economic development of the area. In the area directly behind the seawall one of Havana's busiest roads connects large parts of the city. During extreme weather the road is often a dangerous route to drive and presents a significant inconvenience when closed. Moreover, in the evenings the Malecón attracts large numbers of both local inhabitants and visitors serving an important social function for the city. The Malecón in all its facets has become an icon of the city of Havana and preserving this perception is an essential facet of this study. On September 8th, 2017 Hurricane Irma struck Cuba, only two years after Hurricane Wilma, once again highlighting the vulnerability of the city's coastal defence. The project should encompass all these issues and aim to give an adequate solution.

Figure 3: The Malecón Tradicional (left) and the same area during storm conditions (right)

This report starts by defining the scope of the project in chapter 2 followed by an analysis of the current situation on the characteristics and structural assessment of the coastal defence system. A study regarding the stakeholders is made in chapter 4. In chapter 5 all design criteria, boundary conditions and assumptions are discussed and listed, where after the hydraulic boundary conditions are defined, which serve as the basis for the wave modelling in SWAN. Chapter 7 contains the results of the wave modelling in SWAN and after that, in chapter 8, various design alternatives are presented and evaluated. Following on this evaluation, the final design is presented and worked out in detail in chapter 9. Finally, the report contains the conclusion of this project and recommendations for further research. The sources are listed in chapter 10 according to the APA style following the appendices.

2. Project description

This chapter describes the starting points of the project. Firstly defining the main research question which steers and forms the primary goal of the project. In order to make the goal more manageable sub-questions are formulated. Another key aspect is placing this research in context. The scope must be defined, both geographical and in terms of goals to be achieved, and previous research performed on this topic must be analysed to move forward more efficiently.

2.1. Problem statement

The prevailing issue facing Havana and its existing coastal defence structures are extreme weather events and the resulting wave conditions. These conditions result in wave overtopping which is responsible for significant flooding and large forcing on the seawall which damages the structure and further exposes the area inland. Given the severity of the storm conditions traditional coastal defence strategies point to applying heavy, intrusive structures, an approach which would be exacerbated by the stringent allowable limits for overtopping set by the local administration. The goal however remains to maintain the characteristic aspect of the Malecón, iconic and appealing, while providing a high level of coastal protection for the city and its inhabitants. The frequency of Hurricanes coupled with their severity has put significant pressure on national and local authorities to provide a solution to the growing hindrance caused by these events; furthermore it provides an opportunity to draw investment into an area with growing economic potential.

Since 1995 a number of project groups from the CUJAE and DUT have investigated the area and cooperated with local institutions to find possible solutions for the Malecón and its boulevard. Previous research has been focused on specific areas of the Malecón characterised by particular features, such as location, bathymetry or wave climate, and the goal has now shifted towards integral design. In order to finalise this series of projects and combine them into a feasible solution the aim is to combine existing results and generate a single proposal for coastal defence.

2.2. Research questions and goal statement

Derived from the problem statement the following research question has been defined:

In what way can the Malecón coastal defence system be improved to withstand, up to an acceptable level, hurricane conditions while taking into account technical, environmental, social and financial aspects?

To answer the main research the following sub questions arise:

1. *What is the current situation of the Malecón coastal defence system?*
    a. *Wave overtopping per section during 1/50 year storm conditions/hurricanes*
    b. *Design criteria*
    c. *Current state of the wall and infrastructure*
2. *Which measures have already been investigated per section which meet the design requirements?*
3. *What is the optimal configuration of the available solutions of the integral coastal defence system?*
4. *What is the preferred execution method and phasing?*

Goal statement.

The goal of this project is to create an integral design plan for the coastal defence system of the Malecón, section 2 till 5, which can withstand hurricane conditions comparable to Wilma (2005) and Irma (2017). Design conditions related to technical, environmental, social and financial aspects must be taken into account and used to evaluate effectiveness of the proposal.

2.3. Methodology

This paragraph describes the project methodology which will be used to answer the sub questions and research question. The project will focus on finding a solution for the wave overtopping issue of the Malecón in Havana, Cuba. In this way this project has an evaluating and designing character, as previous research will be evaluated and new solutions will be proposed.

In order to answer the research question, the following approach is used:

1. Evaluating previous reports

There are previous researches on the subject of the Malecón. The reports of these studies will be studied, as earlier knowledge can be obtained and the best solutions can be selected. Important is to find out what has already been studied, what the conclusions and recommendations were and why? Secondly, these reports may be an easy way to get familiar with the subject. The information of the previous reports also helps to define the boundary conditions of the project

2. Theoretical background

In order to understand the problem, a theoretical background will be studied in order to gather determine which approaches, formulas and programs can be used.

3. Site visit and current situation

In September 2017 hurricane Irma brought some devastation and severe flooding. So it is necessary to do an onsite visit to assess the current situation, in order to come up with a solution that will be suitable in the current state of the wall.

4. Stakeholder analysis

Many stakeholders are involved in the project for improving the coastal defence for Havana. Each stakeholder has a different power, interest and resources regarding the project. In order to optimise those, and minimise risks regarding stakeholders, a stakeholder analysis needs to be conducted. This analysis leads to design criteria, boundary conditions and an engagement plan for each of the stakeholders which can be used to gain their support

5. Wave statistics and modelling

For the design of the solutions, the governing wave climate in front of the wall has to be determined by using SWAN. Next, the 2-D wave model SWASH will be used in order to translate the offshore wave climate to the nearshore wave climate.

6. Designing solutions

Taking all gathered data and solutions into mind, one can now design some solutions that fit the boundary conditions of the project.

7. Structural Analysis

In order to design a solution for the wall, a structural analysis will be made with ANSYS. With this analysis, it is possible to make a detailed and realistic design of the wall segments.

8. Evaluating alternatives

All alternatives have to be compared in terms of costs, effectiveness and meeting boundary conditions. In order to compare the solutions, a multi criteria analysis (MCA) is used. This is a decision making method which is suitable for addressing complex problems featuring high uncertainty, conflicting objectives and multi interests and perspectives (Mateo, 2012). This is a great method for deciding which of the alternatives should be implemented.

9. Final Solution

After comparing alternatives and selecting the most viable, a final design will be worked out in more detail in order to give a better overview of the new situation. This final design also includes a cost estimate.

2.4. Geographical scope

The Malecón is a 7 km seawall acting as the coastal defence of Havana. In 1993 a committee of experts divided the total area of the Malecón in 6 different sections, based on the characteristics of each section (See Figure 4):

1. Between La Puntilla and Calle (street) 12, including the river mouth Almendares
2. Between Calle 12 and Calle J
3. Between Calle J and Calle Marina
4. Between Calle Marina and Calle Galiano
5. Between Calle Galiano and Castillo de la Punta
6. Between La Punta and Muelle de Caballería (Entrada de la Bahía de la Habana)

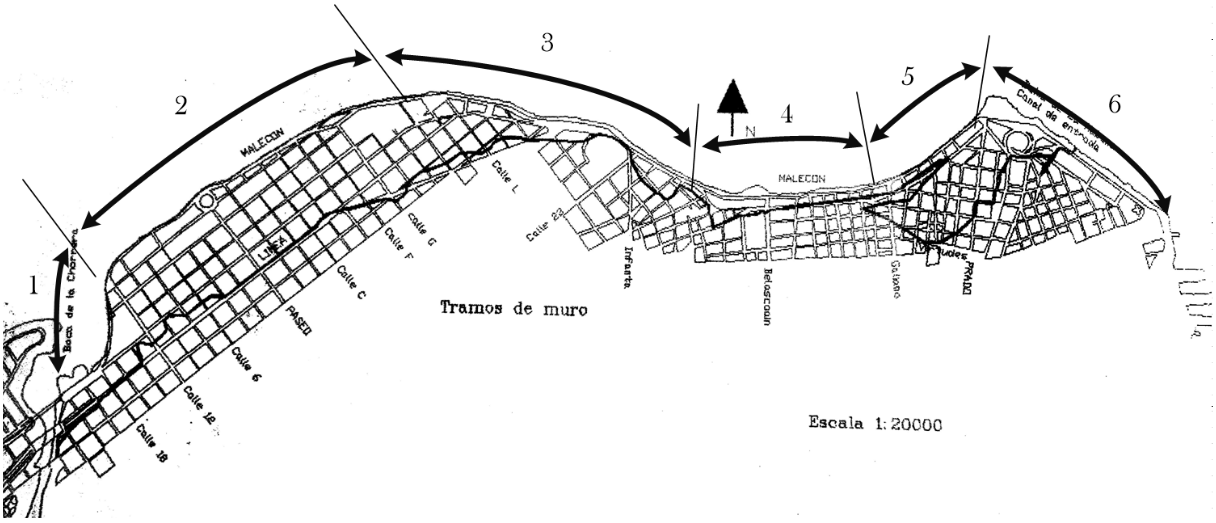

Figure 4: Geographical scope of the project

This research will focus on sections 2 till 5. Sections one and six will be excluded from the research, since they are constructed near the river and the entrance to the harbour and are not adjacent to the sea. Previous studies have all researched different parts of the Malecón. This research will combine the knowledge gained from these studies to come up with an integral solution for the entire Malecón seawall defence.

The city of Havana is situated directly behind the boulevard. Any wave overtopping can directly harm the buildings or infrastructure in the city.

2.5. Previous research

Since 1995, several studies have been conducted in order to strengthen the Malecón Seawall Defence. Some by Professor Córdova, others by Hydraulic Engineering students of the TU Delft. Different teams studied different parts of the Malecón. The main previous studies that are executed for the study area are listed below:

- 1995: Ensayos de rebases para la modificación del Malecón de Habana (Cuba) (Córdova L. , 1995)
- 2003: Havana City Seawall Malecón (Muilwijk, Versmissen, Meijer, Groenendaal, & Veenstra, 2003)
- 2006: Coastal defence for Centro Habana (Baart, van Kruchten, McCall, & van Nieuwkoop, 2006)
- 2015: Coastal Protection Malecón seawall (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015)
- 2016: Analisis de los resultados de los estudio mediante modelacion fisica del rebase del oleaje, presiones sobre los muros costeros y estabilidad de los elementos que componen las bermas y rompeolas. (L.F. Córdova et al, 2016)

With the project outlines in place the following chapter will map the current situation of the Malecón coastal defence system.

3. Current situation

Chapter 3 describes the condition of the Malecón seawall and the coastal defence system in which it is integrated. The aim is to map the current usage, characteristics, condition, interest in, and damage to the study area. This will gather all relevant design and phasing criteria into a single analysis in order to facilitate the design process in later stages.

3.1. Characteristics

During various project site visits the current state of the Malecón coastal defence system has been assessed. This is done on the basis of characteristics and on a visual structural assessment of the wall. The project is divided according to the sections and further detailed based on sections between roads perpendicular to the coastline. The data is assembled in two tables in Appendix A and B. The following chapters describe these observations and provide an analysis of the current situation. Characteristics entail the wall height with regard to mean sea level (MSL), wall height from street level, berm structure, berm length, visible repairs and the number of lanes on the road behind the Malecón.

3.1.1. Section 2

Section 2 lies between Calle 12 and Calle J which protects the Vedado area of Havana. Behind this section are mostly hotels near the coast and a residential area further inland. The seawall has an average height of 0.9 meters from the sidewalk, the crest height from MSL varies between 3 – 4.4 meters, and the length of the berm varies between 2 and 4 meters. Figure 5 illustrates the main characteristics with a cross section and Figure 6 gives an impression of the area.

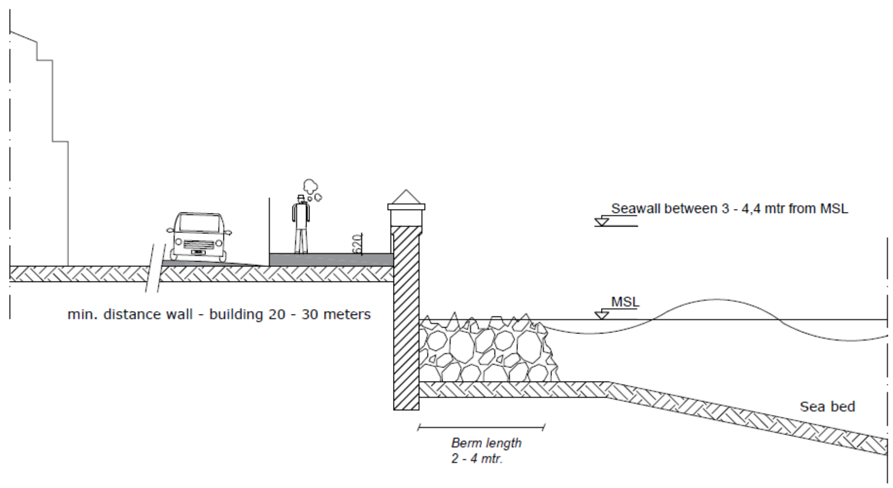

Figure 5: Sample cross-section from section 2

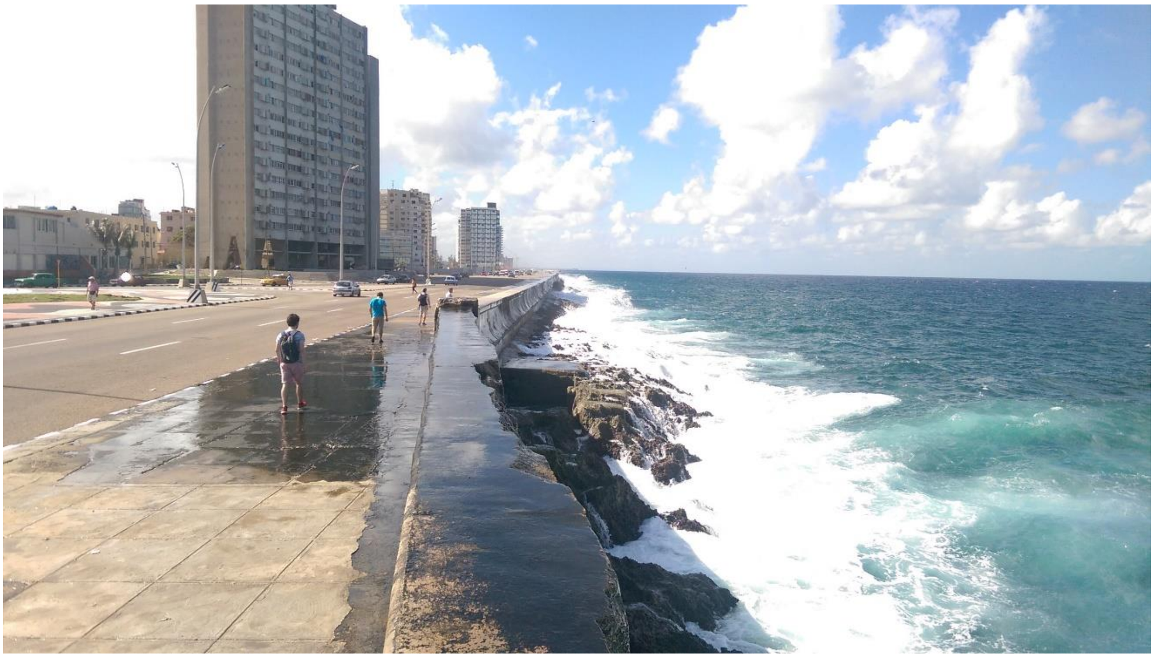

Figure 6: View of section 2 as seen from the seawall

The site visit did not indicate any recent repairs to the construction of the wall. The berm is relatively low and in several parts the berm contained gaps.

3.1.2. Section 3

Section 3 starts at Calle J and ends at Calle Marina which protects parts of Vedado and Centro Habana. The area behind consists mostly of the same functions as section 2, but it also includes important buildings such as the Embassy of the United States of America. The crest height data measured from MSL for this section is missing. In collaboration with professor Cordova an estimation is made based on crest height data from section 2 and 4, which is between 4 – 4.3 meters. The section has two different wall heights, from Calle J – Calle O an average of 1.12 meters and between Calle O – Calle Marina 0.82 meters as measured from the sidewalk. The berm length varies between 2 and 12 meters and in certain points there are parts of the berm missing. The street has 3 to 4 lanes in each direction. Figure 7 gives an impression of the area.

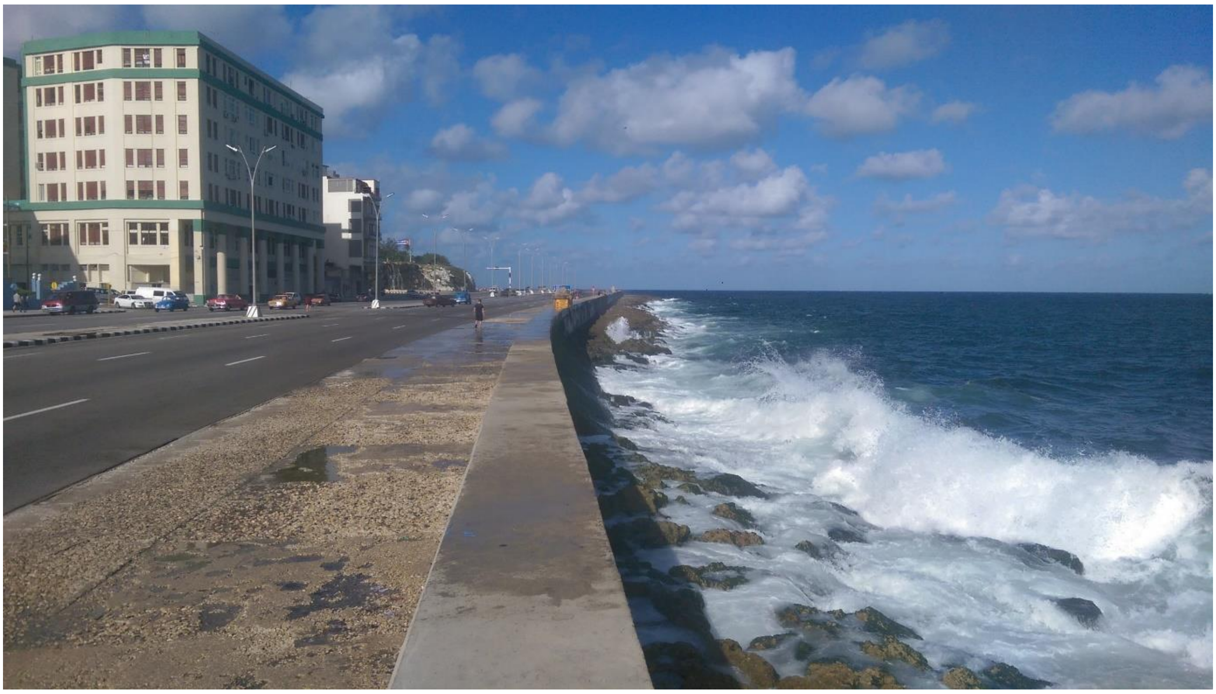

Figure 7: Impression from the coastal defence section 3

3.1.3. Section 4

Section 4 starts at Calle Marina and runs until Calle Galiano which protects a large part of Centro Habana. This includes for instance the Hospital Hermanos Ameijeiras. Section 4 together with section 5 is also addressed as Malecón Tradicional since these parts where constructed first. Buildings behind this section and 5 are severely damaged due to erosion from the sea, as shown in Figure 8a. The crest height measured from the MSL varies between 4 – 4.3 meters. From the street level the height varies between 1.2 and 0.7 meters. There is a natural berm with a length between 2 and 6 meters, with the exception around outcrops as shown in Figure 8b. The road has a mostly 3 lanes and is in good condition, large parts of the sidewalk have been renovated.

Figure 8: (a) Damaged building behind the Malecón (left), (b) example of an outcrop (right)

3.1.4. Section 5

The final section starts at Calle Galiano and runs up to Castillo de la Punta which protects part of Centro Habana and Havana Vieja. In these areas many historical building from the Spanish colonial period are present which are protected by UNESCO. The crest height measured from MSL is around 4 meters along the whole section 0.7 meters from street level. There is a natural berm with a length between 8 and 20 meters with ruins of pools in them. These pools where constructed in the berm in the beginning of the 20th century by cutting holes in the berm including an overhead structure. These are no longer in place but characterise the berm in this section as shown in Figure 9.

Figure 9: Former pools in the natural berm

3.2. Structural assessment

3.2.1. Section 2

Section 2 is subjected to some of the most severe wave conditions in the entire Malecón and the effects on the structural conditions of the seawall are immediately noticeable. Starting from section 1, between Calle 12 and Calle Paseo, the wall is generally in good condition. In Calle Paseo all the structural assessment criteria used in the analysis become critical with significant damage to the crown of the wall and exposed reinforcement already having suffered serious corrosion. A large portion of the crest is missing and severe cracks are visible, this shown in Figure 10.

Figure 10: Large segment of crest missing with corroded reinforcement

Moving from Calle Paseo towards Calle B large longitudinal cracks (along the length of the seawall) can be seen, these are shown in Figure 11. When large waves hit the seawall water can be seen flowing through these cracks indicating the crack crosses the entire width of the structure, it is likely that the next significant weather event will severely damage this track of the Malecón.

Figure 11: Longitudinal cracks in section 2 near Calle B and C

Between Calle D and Calle F large longitudinal cracks just below the crown are visible; these span around 1m and are much less wide, in the order of 1mm, than those shown in Figure 11. The track between Calle F and Calle G also has significant parts of the crown missing and severe corrosion of reinforcement, while the section up to Calle H only has small longitudinal cracks on the crown of around 0.5m in length. The last track of section 2, between Calle H and J, has large segments of the seaward facing side of the wall missing with severe corrosion of the reinforcement steel. Scour has also caused settlements and holes to appear in the sidewalk behind the wall. Overall section 2 is deemed critical in terms of existing damage and it is recommended to begin construction in this section to avoid further deterioration and risk of structural failure.

3.2.2. Section 3

Section 3 begins between Calle J and K, here, and over almost the entire section, many transverse cracks (crossing the crown) are visible at regular intervals. An example of a particularly large transverse crack is shown in Figure 12. The track between Calle M and Calle O has, as mentioned previously, many smaller transverse cracks and large longitudinal cracks just below the crown with lengths of more than 1m. The track between Calle P and Calle 23 has suffered significant damage, with parts of the crown damaged and reinforcement corrosion, due to the presence of a drainage pipe stretching into the sea in front of this section. These elements, present in a number of points along the Malecón, cause waves to impact harder against the wall of which the exact influence, and how to alleviate the issue, is being studied by other parties outside the scope of the project. Section 3 shows numerous signs of significant structural damage, with large longitudinal cracks in several areas, and extra care must be taken to reduce the negative impact of drainage pipes on wave conditions.

Figure 12: Large transverse crack crossing the crown into the seawall

3.2.3. Sections 4 and 5

Similarly to section 2, sections 4 and 5 lie perpendicular to the prevailing wind direction. However for this part of the Malecón the damage, at least what can be observed from visual inspection, appears to be limited. For this reason, together with the relatively short length of these two sections compared to the others, sections 4 and 5 are analysed together. Unlike other parts of the wall there appears to be no significant damage to the structure. This may be due to the position of these sections with sections 2 and 3 acting as a buffer when winds blow from the North-West, the prevailing direction. Another possibility is that the loading conditions are more favourable in these sections due to the lower crest height, due to lower tensile and shear stresses being generated and the wall being loaded predominantly horizontally in compression structural damage is less likely to occur. Finally, it may simply be that the indicators for structural damage are not visible. Figure 13 shows a part of the wall in section 4 and 5, in this area the seawall and boardwalk have been covered in rough, waterproof cladding which has been worn away in parts as can be seen in the picture. This cladding shows few signs of wear or deterioration indicating it has likely been applied recently and may be covering significant cracks or other structural flaws. This seems like the most plausible option as this track is also the most frequented by tourists for which the Malecón was made to look more presentable. Another issue in these tracks are the large rectangular outcrops that stand around 5m further into the sea. While their effect has not been studied, the walls that bound these outcrops show more signs of surface damage than in other sections. If the wall is replaced in this section it is recommended that special attention is paid to find a solution which reduces the influence the sharp corners on wave impact. In summary there is a large degree of uncertainty in the structural state of the Malecón in this section and care must be taken not to underestimate the state of degradation.

Figure 13: Damage to the outer layer in section 4 and 5

3.3. General Findings

There are a number of important conclusions to be drawn from the observations made on the existing situation. The variation in length of the existing berm, referred to as the 'natural berm', can be explained by the construction method of the wall. The wall was constructed in such a way as to maximise the land winning by following the existing coast line, of which the berm was part of. In several sections the natural berm has discontinuities as parts have been demolished for use as filler materials to raise the level behind the wall. The berm varies greatly in width, height, type and effect on waves therefore careful adjustments must be made if it is to be replaced.

During various site visits it becomes clear that the absence of a berm and drainage entrances cause higher amounts of overtopping, these effects are visible even for minor weather events as shown in Figure 14.

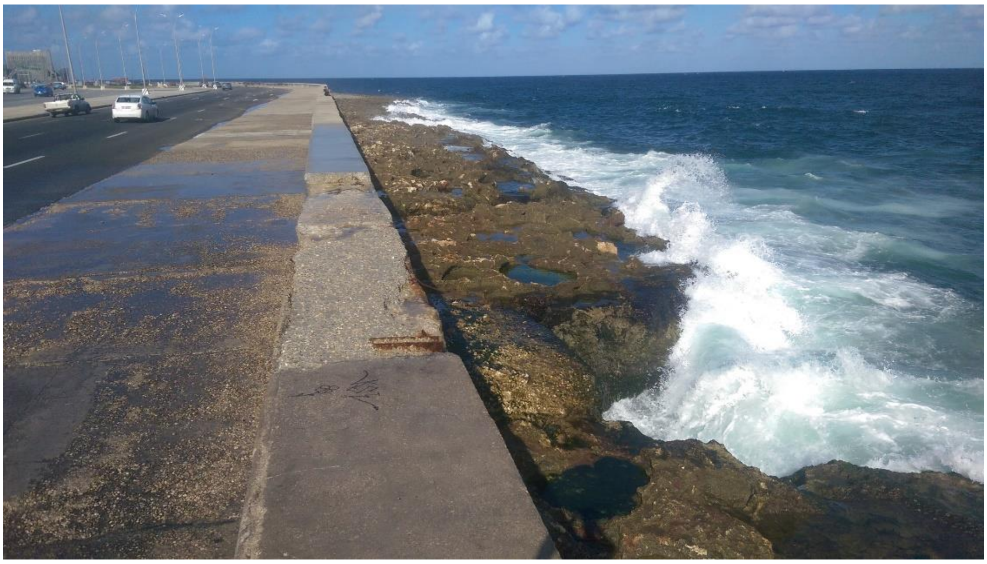

Figure 14: Drain entrance in the sea causing additional overtopping and structural damage

While the asphalting of the streets is in good conditions the sidewalk is severely damaged in parts. In several points there are holes in the top layer which has crumbled and settled. Regarding repairs, no serious structural repairs have been executed but more superficial repairs to cover up cracks in sections 2 and 3 have been performed together with cosmetic repairs in sections 4 and 5.

3.4. Damage assessment

In this chapter the causes and consequences of damage to the coastal defence system with regard to the area inland will be analysed. The Fault Tree Analysis (FTA), presented in Appendix C, is a graphical representation of possible sequences of failures leading to damage or flooding.

3.4.1. Causes

Wave conditions, cold fronts, hurricanes and sea level rise

These factors will be discussed in detail in the chapter 6.

Drainage
The rainwater drainage system of Havana discharges behind the Malecón seawall and is separated from the sewer system. As explained previously, during high water and wave conditions it is observed that the sea water penetrates into the drainage system through the exits located in the sea wall. The high wave pressures at the seaward opening of the system have adverse effects during severe weather conditions and contribute to water inflow into the area. Quantities from this phenomenon are not included in the modelling.

Rainfall
As discussed above the rainwater drainage system does not function during severe weather conditions. As a result the rainwater cannot be discharged into the sea during extreme weather events and will contribute to flooding in the area behind the Malecón. Additional influx from rainfall will not be included in the hydraulic modelling or overtopping calculations.

Structural failure of the sea wall
As indicated in the structural assessment, high water in combination with high waves can damage the sea wall in such a way that the defence system is breached. An example is shown in Figure 15 when during hurricane Wilma a part of the wall in section 2 was separated.

Figure 15: Structural failure of the sea wall during hurricane Wilma

3.4.2. Consequences

The major consequence of the wave overtopping during high water in combination with high waves is flooding of the area behind the Malecón. Figure 16 and Figure 17 indicate the reach of the flooding during Hurricane Irma on the 9th and 10th of September 2017 in the different sections. The red area indicates the flooded area after a standard heavy flood and the yellow area represents the flooding after Hurricane Irma. These floods severely hinder the daily lives of people and cause damage to infrastructure and buildings. The flooding line can be explained due to elevation of the area, as elevation suddenly rises further inland the water can no longer spread.

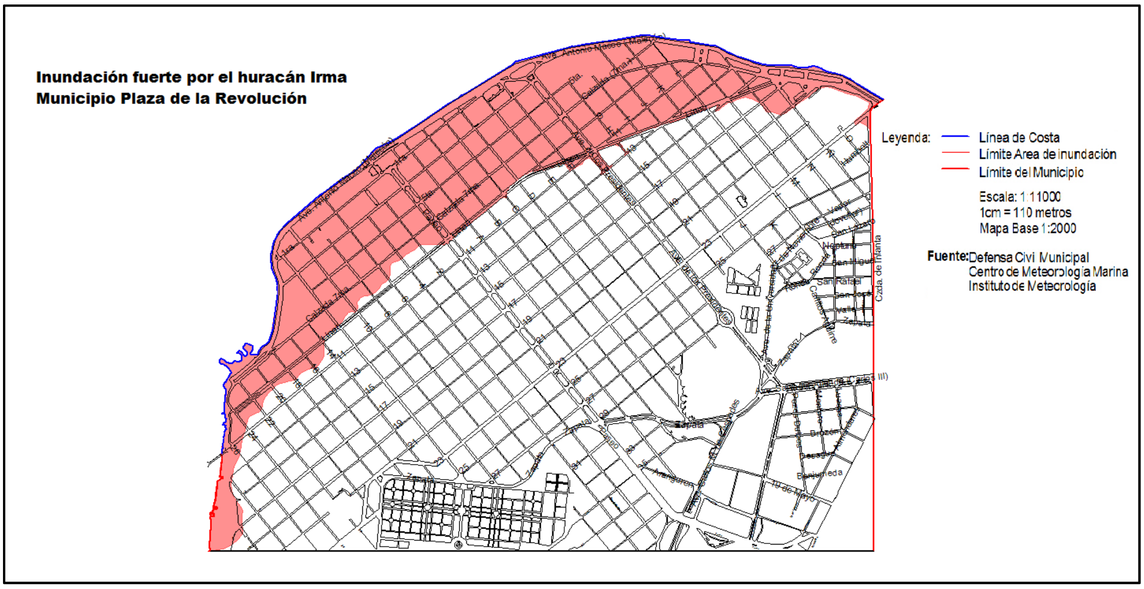

Figure 16: Flooded area after Irma section 2 – 3 (Nilo Hernández Orozco, 2017)

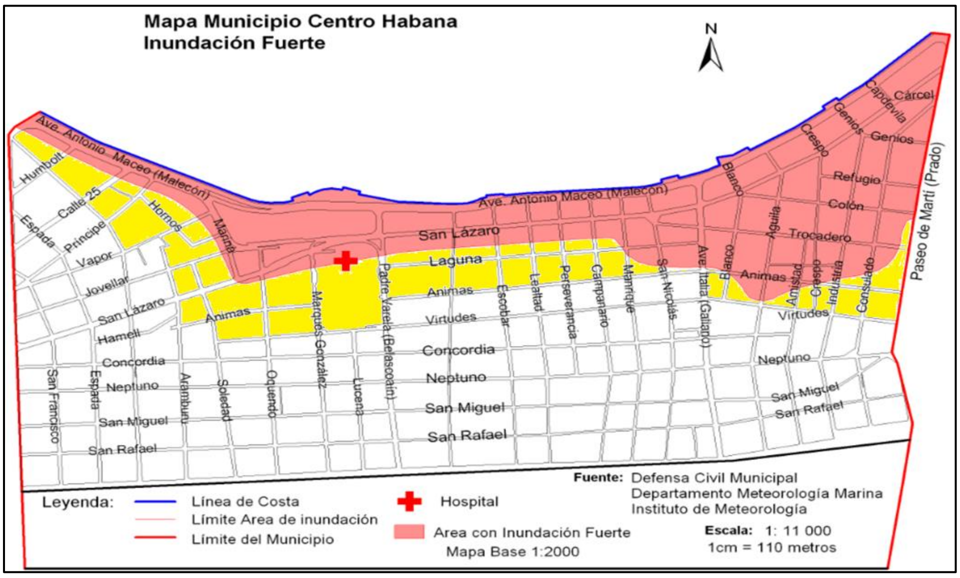

Figure 17: Flooded area after Irma section 3 – 5 (Nilo Hernández Orozco, 2017)

High waves can cause structural damage to the seawall, resulting in a loss of strength for the defence. Parts that are pulled off during a hurricane can become lethal debris or damage the infrastructure and buildings. Lastly, if overtopping of 0,01 m^3/m/sec occurs the road of the Malecón will be closed off for traffic and pedestrians (Oficina del Historiador, Centro de investigaciones Hidraulicas CUJAE, 2012).

3.5. Interfaces between sections

The Malecón coastal defence system is made up of a number of sections and in order to create an integrated solution the transitions between these sections must be determined. To do this the key differences must be identified and managed accordingly.

- Height of the wall with respect to MSL and street level as given in Appendix A.
- The type and length of the berm varies per section and within individual sections.
- The walls are constructed in different periods with different methods resulting in different material properties.
- Structural conditions as described in paragraph 3.3. and Appendix B.
- Wave climate and wind (orientation) in front of the wall

Overall it can be concluded that based on the structural assessment the wall is in need of structural repairs or replacement. Section 2 is most critical and suffers from severe structural damage, in particular the area between Calle Paseo and Calle J. The information gathered in this chapter serves as input for the boundary conditions, design criteria, models and phasing in the coming chapters. The next chapter will list the set of conditions and design criteria for the coastal defence system.

3.6. Financial impact of flooding

In order to compare the current situation of the Malecón with alternatives, the impact of the flooding on Havana should be determined. This way the costs of the alternatives can be compared to the costs of flooding and the advantages of a higher protection level in terms of increasing investments. For the comparison, data from the hurricane Irma will be used, since this is the most recent and extreme flood.

3.6.1. Construction Damages

After Irma swept over Cuba, the National Defence Council issued a detailed damage report of the hurricane Irma. In this report, the government states that the state budget would finance of 50% of the cost of construction materials for people facing the total or partial destruction of their homes and a 50% discount on goods of basic necessity for the affected population (Havana Times, 2017). The total damage of Irma in Cuba according to the United Nations is 13.6 billion Cuban pesos (513 million CUC). The damages of Hurricane Wilma in 2005 were 704 Million CUC (Government of Cuba, 2005). However, in the past, Cuba could stay afloat as a result of the help of their ally Venezuela. But since the oil prices have dropped and Venezuela has problems of his own Cuba will likely receive less help.

In Havana alone, nearly 200 houses were completely destroyed in Havana (Marsh, 2017) and 4288 weakened (Oppmann, 2017). According to the provincial housing authority, a quarter of the buildings in Havana were already in ‘bad or regular’ shape. The residents of Havana complained that Irma would not have been as deadly if the authorities addressed their housing needs. (Marsh, 2017). The city aims to build new homes for the residents. However, a lack of resources makes this hard. The cost of building a house is anywhere between 6.400 to 8500 CUC, repairs work is estimated around 1000 CUC (Darias, 2013). This means that the total rebuilding costs of the damaged and destroyed houses in Havana is between 5.5 and 5.9 Million CUC. These costs are relatively low, since the damage Irma caused in Havana is relatively little.

3.6.2. Impact on Tourism

The biggest effect the flooding might have is on the tourism which generates some 3 billion CUC each year (Roque, Grosbois, & Alonso, 2017). Havana is one of the must see places for tourists when visiting Cuba and the hotels, villas and guesthouses amount to 20% of the national institutional tourism capacity (Havana Reporter, 2016). However, flooding's as a result of hurricanes do reduce the attractiveness of the trip.

In the first half of 2017 2.530.000 tourists has come to Cuba, an increase of 22% for the same period last year. At November 26, the number of international visitors stood at 4.200.000 persons (Veraz, 2017). However, the increase of tourists does not mean the revenues from the tourism sector were not affected. Many travel agencies offered massive discounts up to 65% for trips to Cuba (Boobbyer, 2017). The travel operator which operates in Old Havana is offering a 15% discount on bookings up to April 30, 2018. However, it is unclear how much revenues of tourism were lost as a result of the hurricane damages.

3.6.3. Impact on Investment opportunities

The Historians Office wishes to invest in the area behind the Malecón by restoring buildings and by creating new hotels and restaurants (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malec´on seawall, 2015). However, in order to attract investors for these projects, the investors will need proof that their investment is safe from flooding. Even though Cuba attracted 2 billion dollars in foreign investment deals in 2017 (Frank, 2017), these investments are not in new hotels and restaurants near the flooding area (Mincex, 2016).

Assuming the investments in hotels in Havana can be compared to hotels in other parts of Cuba, the investment which Havana misses out in is between 100 million and 200 million CUC.

3.6.4. Other Impacts

As a result of hurricane Irma, the GDP of Cuba will be negative, even though the economy grew 1.1% in the first six months of the year (Whitefield & Torres, 2017). More negative effects will occur in the next years. This also happened in 2008, when 3 hurricanes caused 10 million CUC in damages. The GDP decreased from 4.1% down to 1.4%. These numbers represent the whole country, and not just Havana. However, since Havana is the capital of Cuba, the damages in this city will likely have a big impact on the overall GDP.

3.6.5. Conclusion

The biggest impact a flooding has is the increased investment risk which makes investors reluctant to invest in hotels and restaurants in the flooding area. The impact on the tourism industry is unclear, but it is assumed that the revenues from this sector are affected. The construction damages of Irma were relatively small, since Havana did not receive the full force of the Hurricane. However, other hurricanes might not spare Havana. Due a lack of information it is not possible to quantify the total financial impact of the hurricane.

4. Stakeholders

In this chapter the various stakeholders in the Malecón Coastal defence project are introduced. These are all the institutions, agencies, companies, and groups or people who are affected by the project, or have concerns with, or interest in the project. The stakeholders are divided into four categories; governmental, research, users and others as mapped in Appendix D. First, a short description will be given of each stakeholder. Next, their powers and interests in the project will be compared followed by an assessment about whether or not the stakeholder is a critical actor or not. Lastly, an engagement plan will be made to give insight in how to deal with the actors discussed.

1.1. Governmental

Government of Havana

The government of Havana is one of two parties responsible for the Malecón coastal defence system which consists of three municipalities; Havana Vieja, Centro Havana and Plaza de la Revolution. The government of Havana is responsible for the welfare of its inhabitants and the development of and maintenance of infrastructure projects in Havana. However, section 4 and 5 of the Malecón are deemed historical monuments and the responsibility of these sections falls under the jurisdiction of The Oficina del Historiado de la Ciudad de la Havana. In order to strengthen the measures which form the complete Malecón Coastal defence system a solution must be found which meets the requirements and interests of both parties.

Oficina del Historiado de la Ciudad de la Havana (Office of Historians)

The Oficina del Historiado de la Ciudad de la Havana is responsible for the historical monuments in Havana. The goal of this institution is to preserve and restore the monuments in Havana while maintaining the characteristic view of Havana. The oldest part of the Malecón (sections 4 and 5) is among these monuments. In order to preserve the characteristic view of the Malecón this office has a list of required characteristics for the Malecón however in order for the Malecón to maintain its function and preserve characteristics of the old part of the city a reduction of flood risk is necessary.

Enterprise of Projects of Transport Works (EPOT)

The Enterprise of Projects of Transport works is an enterprise which belongs to the Ministry of Construction of Cuba. The EPOT is responsible for the final design of the Malecón when the project moves from the design and planning toward the execution phase.

Enterprise of Maritime Works (EOM)

The Enterprise of Maritime Works is also part of the Ministry of Construction of Cuba. This enterprise will be the contractor of the project when the design is finalised. Due to uncertainty in the capabilities and capacity of EOM for executing the entire structure it is likely that foreign engineering and construction firms will be used for support. Possibilities for involvement by external companies in Cuban projects are increasing, particularly from South America, and should be contracted if necessary.

National government of Cuba

The national government of Cuba is not directly involved but will allocate financial resources through the government of Havana and Oficina del Historiade. Awareness for the necessity of this project is increasing as consequences of recent extreme weather events become more severe. Cuban policy regarding foreign involvement could be crucial if it appears to be needed.

1.2. Research institutions

Centro de investigaciones Hidráulicas (CIH) CUJAE

The Hydraulic Research Center (CIH) of the CUJAE has been involved in coastal protection of Cuba since 1995, performing numerous studies regarding the Malecón and its development. The primary goal of this institution is gathering data and creating hydraulic models in order to engineer costal defence structures. CIH will provide their knowledge, results of studies, and advice regarding further contacts, resources, or regulations that maybe of interest in Cuba.

Geo Cuba and Empresa Geominera Oriente (GEOM)

The Geological Institute studies soil conditions in Cuba and holds data for the area around the Malecón, this information will be used as input for the design.

Instituto de Meteorologica

The Meteorological Institute can provide bathymetric information, historic climate data and data of previous hurricanes. This is used to assess the wave climate and to eventually to validate the final design during hurricane conditions.

Facultad de Arquitectura, CUJAE

The Faculty of Architecture of CUJAE will be part of the new aesthetical design team of the project as designated by the Oficina del Historiado.

1.3. Users

Residents and companies nearby the Malecón

Residents and companies nearby the Malecón suffer greatly from the overtopping when the area behind the Malecón floods; transport, services, and tourism are all severely hindered and damage to property must be compensated. Although local inhabitants benefit from larger structures they also use the Malecón for its social function and want the view of the ocean to be preserved. Hotels by the coast also have considerable interest in the development of the project as the level of protection influences insurance costs. Improved coastal protection is likely to increase interest from future investors and potentially raise real-estate prices to help further fund government enterprises.

There is no law or rule which lets citizens directly oppose building plans however plans must be approved by the Instituto de Planificacion Fisica (IPF). The owners of the project have to present their plans to a commission consisting of governmental, monumental, and other institutions' representatives. From this the commission may advise the project owner to present the plans to the citizens as well to gain their insight and support, this is the only way citizens can influence the project.

Recreational users of the Malecón

As discussed previously the Malecón and its boulevard serve as a social meeting place for both locals and tourists. People walk along the sidewalk with the view of the ocean and use the Malecón as an area to relax, talk, and listen to music. Tourists also regularly visit the Malecón, drawn by the numerous pictures taken from this location which has become an icon of Havana. As the Malecón is of great value to inhabitants and a great attraction for tourists, this characteristic should be preserved. The total contribution of travel and tourism to the gross domestic product (GDP) in Cuba was of 9.6% in 2016 **Invalid source specified.** and this percentage expected to grow annually. Therefore the view on the ocean and the possibility to sit on the wall, or an alternative, must be preserved.

Road users of the Malecón

The Malecón is one of the main roads in Havana, with 3 to 4 lanes in each direction it has a large capacity and connects the old to the new part of the city. During severe weather conditions, the road is closed due to hinderance of the water. Obstruction of traffic during construction and renovation of the Malecón should be taken into account for the road users.

1.4. Other

Hospital Nacional Hermanos Almejeiras

The Nacional Hermanos Almejeiras hospital is one of the largest Hospitals in Cuba and is located only 150 meters from the Malecón in Havana Centro; past flooding of the Malecón has caused the hospital to partly shut down. For this reason the hospital has significant stake in reducing overtopping to remain functional during storm conditions, a crucial aspect for people requiring medical assistance.

Foreign engineering/construction firms

Cuba is seen as an opportunity for foreign engineering and construction firms for coastal protection. Several foreign firms are or where involved in projects in Cuba, such as Bordstein-Ries, Boskalis, and Deltares. These engineering firms can contribute knowledge and experience in coastal engineering for both design and execution. These firms in turn would be contracted and have the opportunity to work of significant renown.

UNESCO

The old part of Havana, Havana Vieja, was classified as a World Heritage Site in 1982 by UNESCO. UNESCO and local authorities aim to keep the characteristic aspects of this part of the city intact as much as possible and continue to fund restoration works. UNESCO has also identified the threat of flooding 'Havana is occasionally subjected to severe tropical weather (including hurricanes, as in 2008), which can threaten the authenticity of the property.' (UNESCO, 2015). This highlights the notion of safeguarding characteristics while improving protection from extreme weather, the interest and financial contribution remains unclear however.

1.5. Power and Interest

All stakeholders involved in the project have a certain power and interest related to the strengthening of the Malecón, these are shown in Figure 18. A table of the Power versus Interest is shown in Appendix E. All the governmental actors have a high degree of power and thus should be actively involved in the project. Of the research organisations, CIH has the highest interest and power, since they are conducting research on the Malecón for several years and have valuable information which can be used to further the project. Users have different levels of interest but few have any significant influence on the proceedings. The requirements for most users regarding the Malecón often overlap with stakeholders with more power which helps guarantee their satisfaction. As for the other stakeholders, they have a high degree of interest, but only a small form of power in the current situation. The power of engineering firms might increase if the problem owners decide to use foreign engineering firms in the project which might prove useful given the knowledge and technologies at their disposal.

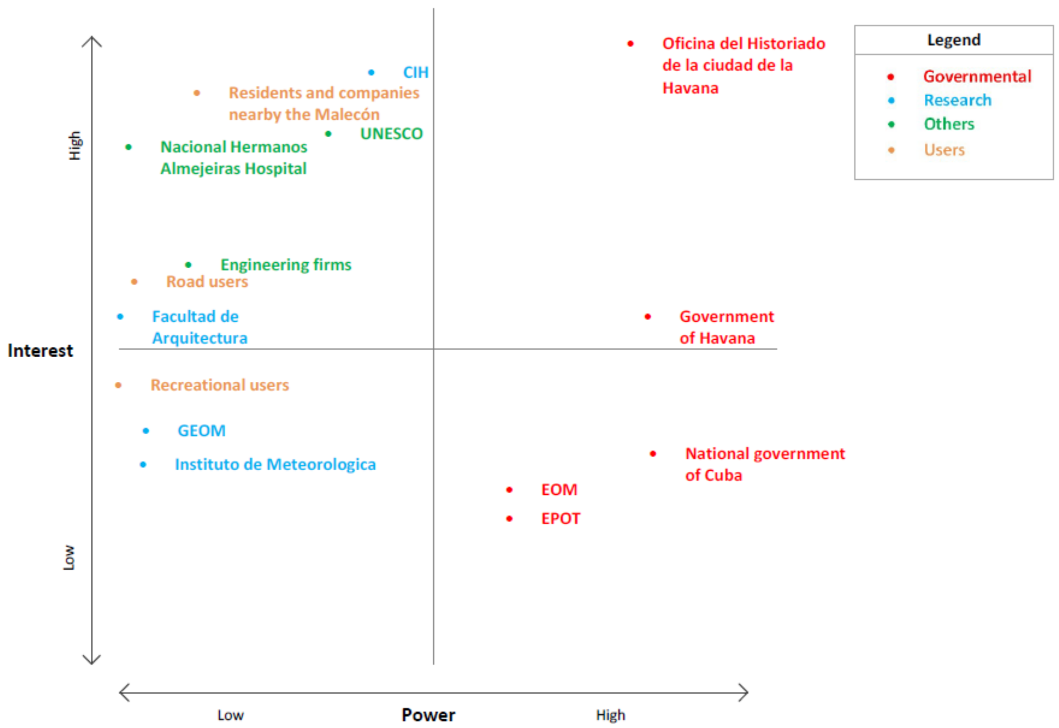

Figure 18: Power-interest grid

1.6. Critical Actors

Critical actors hold executive power within the project and decide whether the project or a component hereof can advance to the next stage of development. Whether or not an actor is critical depends on its resources, possibilities for substitution, and how dependent the project is on the actor. The full assessment for critical actors is listed in Appendix F. The governmental actors are all deemed critical and hold a medium to high degree of resources and the project depends on them to a similar degree. Since all these actors are governmental institutions they are irreplaceable.

From the research institutions, the Centro de Investigaciones Hydráulicas of the CUJAE is the sole critical actor. This institute has more than 20 years of research and studies on coastal defence of the

Malecón which represents irreplaceable knowledge which the project depends greatly upon. Lastly, UNESCO is also deemed as a critical actor because of their substantial subsidies for preserving Old Havana.

1.7. Engagement plan

Based on the general description, power, and interest of the stakeholders an engagement plan is set up. The engagement plan aims to evaluate the differing interests of the multiple project owners from a neutral standpoint and objectively analyse each one. Stakeholders can be engaged, involving them in the project, or disengaged, to reduce their influence. Involvement can be described in several forms (Leijten, 2017), namely; informative, consulting, involvement, collaborative and empowering. This is summarized in Appendix G. In this project, all the stakeholders are being engaged. The most important engagement in this project is the engagement of the government of Havana and the central Cuban government. While they are one of the two project owners they do not yet perceive it as such and need to be activated in their role. The government of Havana should collaborate with the Historical office and the CIH to find an overall solution to the overtopping problem. The historical office is responsible for the old part of the Malecón and the CIH has done many studies on the Malecón and are currently the experts on the overtopping problem. Including the CIH and using their expertise would maximize the effectiveness of the proposed solution. Users of the Malecón can be informed of the project in order to anticipate on the construction and possible hindrance it may bring. If the Instituto de Planificacion deems it necessary, the plans can be presented to the citizens for their support. However, this would be unfavorable as it may slow down the decision making process. Furthermore, the interests of inhabitants are aligned with these of the historical office, protection against overtopping while keeping the characteristics of the Malecón, creating an unnecessary added step. These characteristics include the social aspects and should not be hindered by the requirements set by the proposed plans.

The engagement plan for engineering firms depends on the level of the proposed solution and building techniques required. Engineering firms will likely provide a large portion of the equipment, skills and expertise necessary to construct the coastal defence system. The degree to which these are needed determines level of allowable involvement allowed by Government policy and financial resources.

Lastly, the stakeholder UNESCO is a financial resource for the Malecón and should be consulted on the project. It is also possible to involve them further in the process if they can provide extra funding to maintain the character of the Malecón.

1.8. Conclusion

The interests and stakes described in this chapter are coupled with requirements which ensure the project remains aligned with the wishes of the parties involved, these requirements will be discussed in the following chapter. The actors with the most power and interest are the local governmental stakeholders. National stakeholders have power but less interest in the project. A single, central project owner or organisational group regarding the project is currently lacking and would be preferred. Given the size, impact, and governing structure of Cuba this should be initiated from the central government. This should unite the parties responsible for the different sections and develop a funding in line with government strategies, allowing room for foreign investment for example from tourism or greater involvement in the project.

Other critical actors are UNESCO and CIH for their resource provision in terms of financial support and knowledge respectively. For each of the stakeholders, an engagement plan is constructed with concrete steps. The most important engagement is to activate the government of Havana as a party responsible for the sea defence. Together with the Historian office and the CIH, they must collaborate in order to elaborate a solution for the issues currently facing Havana.

5. Design criteria and boundary conditions

The list presented in this chapter describes the boundary conditions and design criteria for the proposed solution for the Malecón coastal defence system. This is based upon analysis of the current situation, hydraulic and structural conditions, stakeholder interest, previous research and discussions with relevant parties.

5.1. Boundary Conditions

Each proposed solution has to meet the boundary conditions as listed below.

1. The study area during this project is the Malecón adjacent to the sea, sections 2 till 5. The study area starts at Calle 12 and ends at Castillo de la Punta. The length of the study area is 5950m.
2. The bathymetry as provided by the Office of the Historian will be used.
3. The ground in the area around the coastline consists mainly of rock (Baart, van Kruchten, McCall, & van Nieuwkoop, 2006).
4. The provided solution should be designed for a service life of 50 years.
5. The area behind the Malecón should be protected from storms with a return period of 50 years. This storm represents the serviceability limit state (SLS) or 'design conditions'.
6. Hurricane Wilma represents the ultimate limit state (ULS) for which the structural elements must be designed.
7. The structural integrity of the structure should be guaranteed during the design life.
8. The maximum allowable difference in height between the crest level of the seawall and the adjacent sidewalk is 1.25 meters (Oficina del Historiador, Centro de investigaciones Hidraulicas CUJAE, 2012). This height is the result of a study for section 4 and 5 but, in consultation with professor Cordova, it was extended to section 2 and 3 also.
9. Plans for reducing flood risk should not intervene with the plans of the Office of the Historian for the project area.
10. Each design must comply with Cuban norms (building codes) and legislation.
11. If there is no Cuban norm available or suitable, European standards will be applied.
12. The maximum allowable mean wave overtopping is 0.05 m3/s/m for a storm event with a return period of 50 years. This entails that the road cannot be used for normal traffic during design conditions (Verhagen, d'Angremond, & van Roode, 2009).
13. For a return period of 50 years the offshore significant wave height is 9.2 m.
14. For a return period of 50 years the total surge amounts 1.67 m.
15. For a return period of 50 years the total elevation is therefore MSL +1.95 m, considering the total surge and sea level rise.
16. The sea level rise will be 0.27 m by 2050 (Centella, Benzilla, & Leslie, 2009)

17. The wall can currently be characterized as concrete of strength class H(12/15) with E-modulus of 27 GPa.
18. The coastal protection system must remain functional during construction.
19. The proposed alternatives and solution should be assessed using storm conditions, wave height and set up, comparable to those during Hurricane Wilma (2015).

5.2. Design Criteria

The following design criteria will be used in a Multi Criteria Analysis to assess the proposed alternative.

1. The design should reduce the overtopping of the Malecón as much as possible, aiming for a value of 0.05 m3/s/m , while taking the other design criteria into account. Structural integrity must be preserved over the design life.
2. The characteristic view of the Malecón and boulevard should be preserved as much as possible. In consultation with Professor Córdova characteristic elements identified are:
   - The rounded edge of the existing wall
   - Tower constructions of the wall
   - Aspects of the natural berm e.g. the old pools 'Baños' dating from 1910 – 1920
3. The social function of the Malecón should be preserved as much as possible.
4. A minimum level of disturbance during construction should be sought.
5. Value in relation to costs should be evaluated.
6. Low maintenance costs are more favourable than direct low building costs.
7. Standardisation of design as well as local materials and knowledge are favoured over applying foreign practices.

5.3. Assumptions

To realise results for this study several assumptions are made for physical and technical situations as listed below:

1. Division of tracks is performed according to (Córdova Lopez, 1995) with 6 different sections based on wave directions and characteristics of the hinterland.
2. Existing wall height with regard to mean sea level (MSL) and sidewalk vary therefore allowable crest height with regard to MSL and berm height also fluctuates. For modelling of the wave overtopping standard heights are used for all sections, the new crest height of the wall with recurve is MSL +4.46m. Where berms are applied 'Berm I' from the physical model tests by (Córdova López, et al., 2016) is applied (crest height MSL +3.28m and berm width 5m).
3. Wall height in relation to MSL in section 3 is estimated on the base of data from section 2 and 4 since data was not available.
4. The drainage system does not function during severe weather condition as inflow exceeds discharge capacity, this will be neglected in the proposed solution and during modelling.
5. Due to limited availability of pressure data non-dimensional pressures were extrapolated from sections 4 and 5 to 2 and 3 using the highest, and therefore most conservative, value.

6. Marine data analysis and forcing on the Malecón seawall

The Malecón seawall is exposed to several combinations of hydraulic loading. To be able to define a working model of the Malecón and to calculate the different loads this chapter explains the hydraulic boundary conditions and the assumptions that are made in order to come to a good approximation of the reality. With the obtained wave conditions combined with previous research a pressure profile on the curved wall can be determined in paragraph 6.8.

6.1. Level of protection

An important boundary condition for designing flood defences is the level of protection and the probability of failure that will be accepted. According to the EurOtop manual, for flood defences protecting large areas at risk, the design life should be 50-100 years and the level of protection 100-10,000 years (see Table 2) As the Malecón is protecting a large city and its inhabitants, a design life of 50 years is chosen. The level of protection that will be applicated is set to a 1 in 50 year storm event, this is lower than the advised level of protection in the manual, but this is the level advised by Professor Córdova. The level of protection is the overall level of protection so all failure probabilities will be added up in order to check if the level of protection is guaranteed.

Table 2: Level of protection according to EurOtop manual

Hazard type and reason	Design life	Level of Protection(1)
	(years)	(years)
Temporary or short term measures	1–20	5–50
Majority of coast protection or sea defence walls	30–70	50–100
Flood defences protecting large areas at risk	50–100	100–10,000
Special structure, high capital cost	200	Up to 10,000
Nuclear power stations etc.	–	10,000

6.2. Currents and morphology

The project location is located at the north coast of Cuba and according to the Cuban Department of Oceanography two important currents exist near this coast (Frag, Morale Abreu, Rondón Yero, López Garcia, & Díaz Llánez, March 1995). The first type is the tidal current with a periodic character and secondly is the residual current, with a non-periodic character. In earlier research of the study area (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) it is stated that these currents have small velocities and therefore will only have little effect on this study. Therefore currents are neglected in further calculation.

The seabed of Havana Bay consists mainly of rock and large elements. Therefore in this study morphological effects and changes are not taken into account, cross-shore and long-shore sediment transport will also not be considered in this study.

6.3. Wave climate

As introduced in section 2.1 the governing situation for the Malecón seawall is found when hurricanes or cold fronts occur around Cuba. These phenomena can cause large significant wave heights in front of the seawall which result in wave overtopping. The Cuban meteorological institute has recorded data about these events over the past 40 years (Appendix H), which provides input data for designing under storm conditions.

6.3.1. Tropical cyclones

Tropical cyclones are a frequent occurrence in the Gulf of Mexico, almost every year the island is hit by a tropical cyclone. Tropical cyclones can develop into hurricanes, where hurricanes are classified by the Saffir-Simpson Hurricane Wind Scale (1974). One may speak of a hurricane when the wind speeds exceeds 118 km/h.

To start the calculations in the first part of the project the wave data from past tropical cyclones and hurricanes, given by the Metrological Institute, is used.

6.3.2. Cold fronts

Cold fronts divide masses of cold dry air at high latitudes from masses of warm and humid air at lower latitudes. They generally occur between October and April and can cause very strong winds from the North together with rainfall and high waves.

Cold fronts can be classified based on the maximum wind velocity at an elevation of 10 meters:

Weak: $V_{max} < 10$ m/s
Moderate: 10 m/s < $V_{max} < 33$ m/s
Strong: 33 m/s < V_{max}

Cold fronts of moderate and heavy intensities have historically caused coastal flooding. In Appendix H wave data for cold fronts can be found.

6.3.3. Significant Wave height

An important factor in the design of the renewed Malecón is the significant wave height. Out of the data from previous reports and from the Meteorological Institute, a graph can be made in which the significant wave height is related to the return period in years. This graph will be used in probabilistic design approach in order to determine the probability of failure of the complete Malecón defence system. The significant wave height for the return period of 50 years is 9.2 meters, see Table 3.

Table 3: Return periods for different significant wave heights, Hs

Return Period [yrs]	Hs [m]
5	6.0
10	6.9
20	7.8
50	9.2
100	10.1

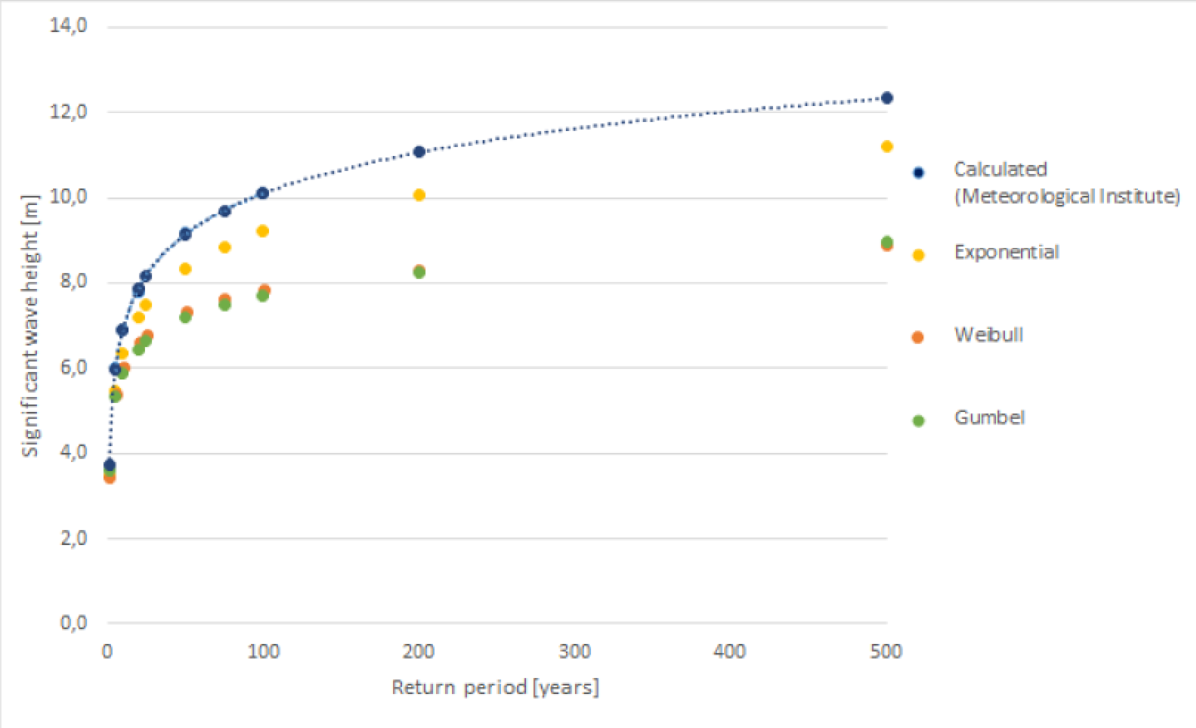

Figure 19: Significant wave height and return period

6.4. Water level elevation

Next to the significant wave height, the water elevation is also an important factor in contributing to the volume of wave overtopping. The total water level elevation consists of several different phenomena; tidal elevation, storm surge and sea level rise mainly due to climate change.

6.4.1. Tidal elevation

The tidal elevation at the north side of Cuba has a diurnal character with a small tidal range. The average tidal amplitude is 0.31 m and during spring tide 0.61 m (NOAA, 2015). This value is already included in the storm surge calculations of the Meteorological Institute (Meteorological institute, 2015)

6.4.2. Storm surge

During storm events storm surges occur, this surge is caused by the following mechanisms; wind setup, wave setup and regional low atmospheric pressure. Especially hurricanes are accompanied by larger storm surges (Meteorological institute, 2015).There is a difference between the storm surge offshore and onshore, in the onshore storm surge shallow water mechanisms such as wave shoaling and refraction also play a role, leading to wave setup in the nearshore area. The meteorological institute provides data for the combined storm surge and tidal effects, given in Figure 20.

The data in Figure 20 is taken from an offshore buoy, so wave setup due to shallow water phenomena must be included for the near shore values of the water elevation.

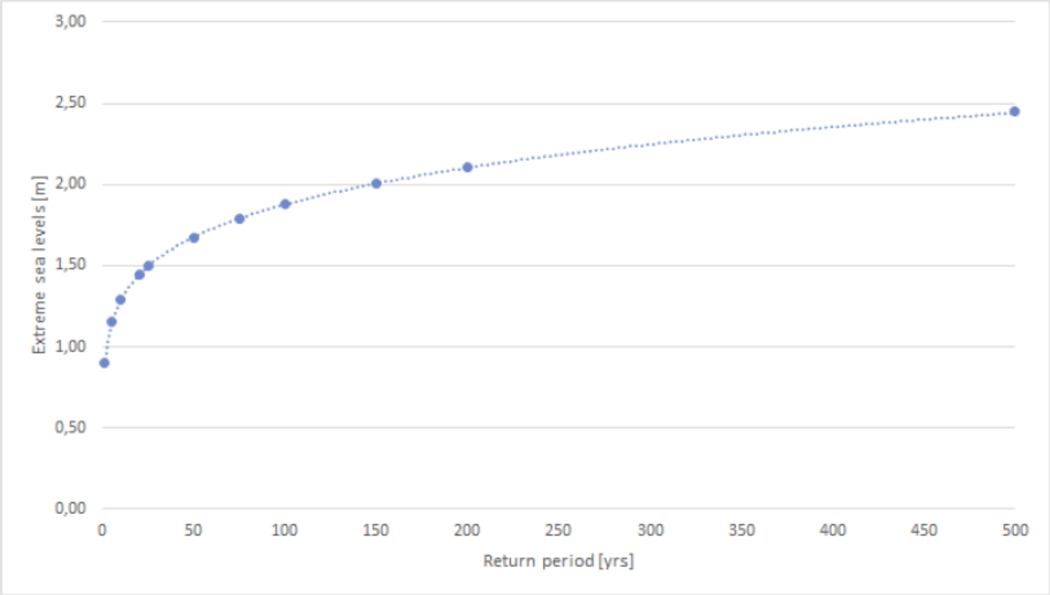

Figure 20: Extreme sea level and return period

6.4.3. Sea level rise

Due to the changing climate the sea level is rising. According to the IPCC (J.A. Church, 2013).

"Regional sea level changes may differ substantially from the global average, showing complex spatial patterns which result from ocean dynamical processes, movements of the sea floor and changes in the gravity due to water mass redistribution (land ice and other terrestrial water storage) in the climate system."

Combining all scenarios for climate predictions by the IPCC, the climate agencies foresee a relative sea level rise of 0.08 m to 0.27 m in 2050 (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015). For the design life of 50 years, a conservative value of 0.27 m is chosen.

6.4.4. Total water level elevation

The total water level elevations that will be taken into account during the mathematical modelling and project design is MSL+1.95 m. This value includes sea level rise, storm surge, and tidal elevation.

6.5. Probabilistic design

In order to determine the design event, the storm event that is governing and on which the design of the seawall will be based, a probabilistic design approach will be used to combine the different loading parameters. In this chapter this process will be described and the loading combinations will be defined.

6.5.1. Variables

There are two main loading parameters: the significant wave height, H_s, and the water level elevation, zeta (or ζ). The most straightforward and conservative approach is designing with both parameters at values with a return period of 50 years. In this way, the overall return period will be higher, this will be referred to as the 'zero' combination.

For return periods of 50 year the significant wave height is 9.2m and the water elevation is 1.95 m.

In the probabilistic design approach, three different ways in handling the possible relation between H_s and zeta exist: I) full dependence, II) full independence, or III) partial correlation. As discussed in previous research by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015), the partial correlation is the best way to approach these parameters.

The data used by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) was used as no new data could be obtained from the Meteorological Institute. For this data set a correlation between the parameters H_s and zeta of 0.44 was found. However, it was stated that based on the limited data provided, the correlation may be biased and a safer approach should be used. Therefore the 98% confidence interval was used and a correlation of 0.8365 was found (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015).

Using this correlation and the Ditlevsen method a range of probabilities can be calculated (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2016). In this project the upper limit of the Ditlevsen boundaries is used as this results in higher loading in the combinations.

6.5.2. Loading combinations

In Table 4 the different combinations that will be modelled are listed, the loading combinations proposed in (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) are used and further expanded with data from Hurricane Wilma. This last combination is one of the strongest hurricanes in the past decades which resulted in significant flooding in Havana. These differ slightly from the rest of the combinations; therefore these will be added as separate combination.

Table 4: Combinations of wave climate parameters for loading situations

		Wave Height		Water level elevation	
Combination	**Combined RP [yrs]**	**Hs [m]**	**RP [yrs]**	**Zeta [m]**	**RP [yrs]**
0 'Zero'	92.6	9.14	50.0	1.95	50.0
1	50	6.52	7.5	1.95	50.0
2	50	7.80	18.9	1.88	40.0
3	50	9.14	50.0	1.50	7.5
4	50	8.84	40.0	1.70	18.9
5	50	8.44	30.0	1.80	28.6
6 'Wilma'	299.4	5.80	5.0	2.28	300.0

6.6. Wave overtopping theory

A key parameter in designing a solution for the Malecón seawall is the amount of wave overtopping over the current wall and also for the proposed solution. This can be done in several ways, with numerical models like SWASH (T. Suzuki, 2011) by calculating it with formulas from the EurOtop manual, or by data derived from physical model tests. In this section the formula from the EurOtop manual and the data from physical model test will be discussed and compared. SWASH will be used to model the various combinations.

6.6.1. Wave overtopping by EurOtop Manual

For a vertical seawall with a submerged toe the manual states that two types of waves can be distinguished in front of the wall; non-impulsive and impulsive waves.

Non-impulsive waves occur when waves are relatively small in relation to the local water depth and have lower wave steepness, under these conditions overtopping waves run up and over the wall applying smoothly fluctuating loads to the wall.

Impulsive conditions occur when waves are larger in relation with the local water depth, perhaps shoaling up over the approach bathometry or the toe of the structure itself. Under these conditions the waves will break violently against the wall and forces up to 10-40 times greater than under non-impulsive conditions are generated. In order to calculate the wave overtopping the wave condition must first be determined, this can be done by calculating the impulsiveness parameter h_* for vertical walls:

$$h_* = 1.35 * \frac{h_s}{H_{m0}} * \frac{2\pi * h_s}{gT_{m-1.0}^2}$$

In which:

h_* $= impulsiveness\ parameter$

h_s $= waterdepth\ at\ the\ toe\ of\ the\ structure$

H_{m0} $= wave\ height\ at\ the\ toe\ of\ the\ structure$

$T_{m-1.0} =\ average\ wave\ period$

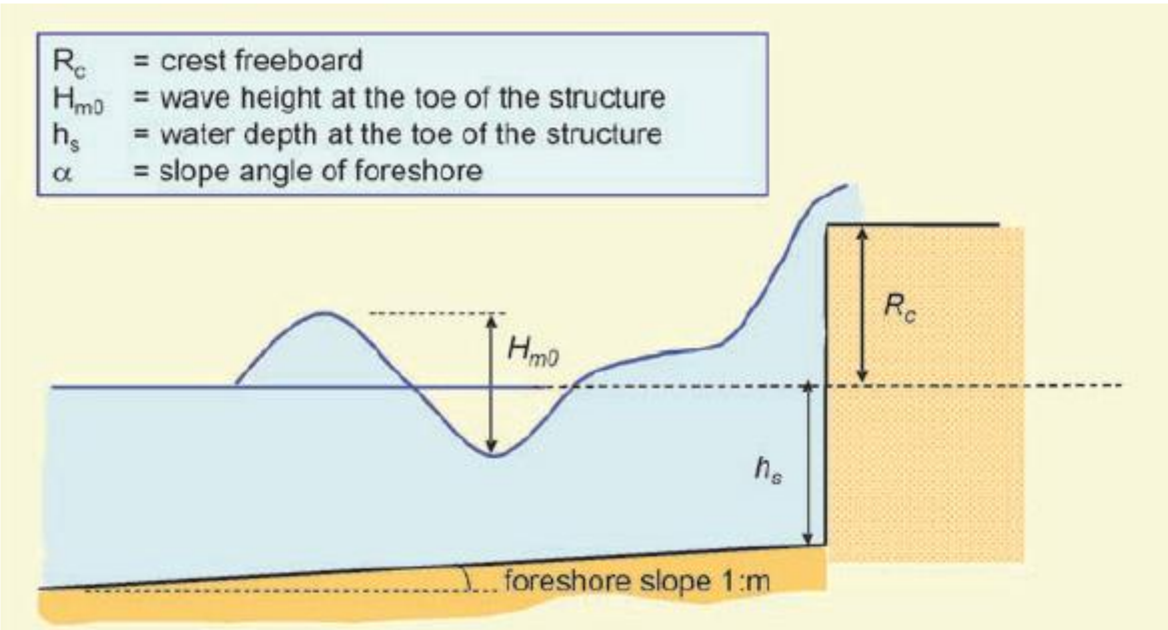

Figure 21: Wave overtopping according to EurOtop manual

Non-impulsive conditions dominate at the wall when h_* >0.3, and impulsive conditions occur when h_* <0.2. The transition between conditions for which the overtopping response is dominated by breaking and non-breaking waves lies over 0.2<h_* <0.3. In this region, overtopping should be predicted for both cases, and the larger value assumed.

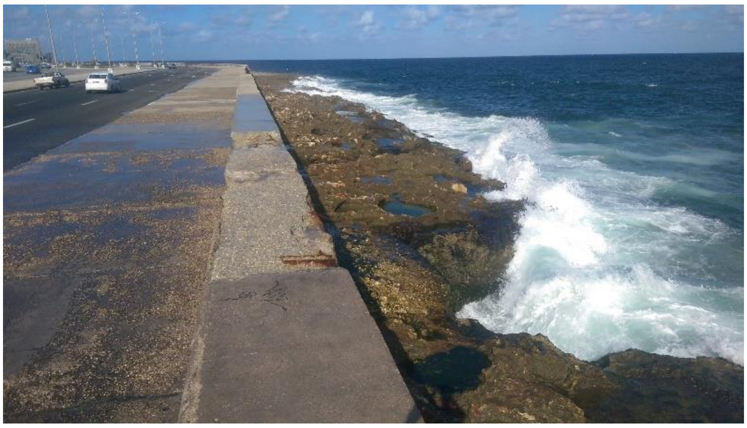

Figure 22: Waves breaking at the natural berms in front of the Malecón

It is well established that a relatively small toe berm can change wave breaking characteristics, thus substantially altering the type and magnitude of wave loading (Oumeraci, 2001)). This toe can be classified in three categories according to the EurOtop manual:

1. Small toe mounds which have an insignificant effect on the waves approaching the wall – here the toe may be ignored and calculations proceed as for simple vertical walls.
2. Moderate mounds, which significantly affect the wave breaking conditions, but are still below water lever. Here a modified approach is required.
3. Emergent mounds in which the crest of the armour protrudes above still water level. Prediction methods for these structures may be adapted from those for crown walls on a rubble mound.

During the site visit it became clear that in the case of the Malecón all three categories are present (see Figure 22). In some parts the berm is missing, in some parts it lies above still water level, meanwhile other parts are flooded during normal conditions. In order to determine an accurate estimate of the overtopping volume the plain vertical wall will be used to calculate the overtopping. Afterwards, in areas where a natural berm is present, a reduction factor of 0.88 will be applied (L.F. Córdova et al, 2016).

Depending on the impulsiveness parameter, the wave overtopping over a plain vertical wall can be calculated using the following formula:

For h_* >0.3 (Non-Impulsive):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04\mathrm{e}^{-2.6*\frac{R_C}{H_{m0}}}$$

For 0.2<h_* <0.3 (Transition phase):

$$\frac{q}{{h_*}^2\sqrt{g{h_s}^3}} = 1.5 * 10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-3.1}$$

And for over h_* <0.2 (Impulsive):

$$\frac{q}{{h_*}^2\sqrt{g{h_s}^3}} = 2.7 * 10^{-4}\left(h_*\frac{R_c}{H_{m0}}\right)^{-2.7}$$

6.6.2. Physical model tests

In 2016 new physical model tests were performed on both the straight vertical wall, the curved wall, the curved wall + berm and the curved wall + breakwater option (L.F. Córdova et al, 2016). From these results new parameters were found in order to determine the wave overtopping for the different configurations. In Table 5 the results of these tests are shown.

Table 5: Summary of physical model test results

Variant	Combination	a	b	R^2
Vertical wall	All frequencies + Tm + Wave setup	0.008	-2.05	0.974
Curved wall	All frequencies + Tm + Wave setup	0.004	-2.16	0.985
Curved wall + berm	All frequencies + Tm + Wave setup	0.000161	-2.78	0.993
Curved wall + breakwater	All frequencies + Tm	0.000006	-3.23	0.899

The graphical representation of these results and further explanation can be found in Appendix I. The wave overtopping for different variants can now be calculated using the following formula:

$$\frac{q}{h_* \sqrt{g h_s^3}} = a * \left(h^* \frac{R_C}{H_{m0}}\right)^b$$

6.6.3. Comparison between the two methods

To compare both methods the wave overtopping for the Malecón was calculated using data from SWAN. In this way it was found that the EurOtop Manual results applied to the case of the Malecón underestimate the volume of wave overtopping by 40% compared to the physical model tests. Therefore in this project the physical model data will be used to calculate the overtopping and determine the measures to be taken, for this method the wave overtopping is not underestimated and calculated more specifically for the Malecón. More detailed results can be found in Appendix K.

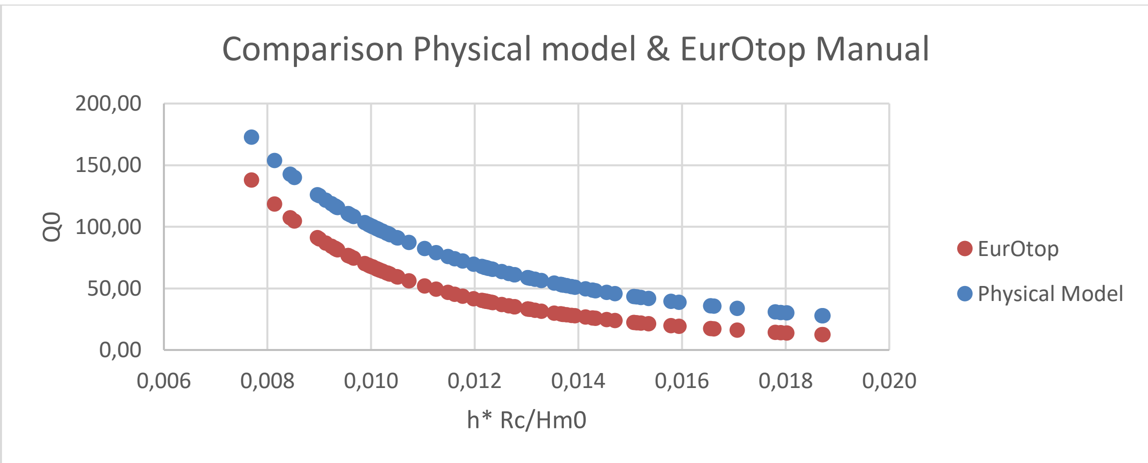

Figure 23: Comparison of physical model test results and EurOtop

6.7. Wave transformation with SWAN

In order to calculate the wave overtopping, the wave climate just in front of the wall has to be determined. In this project this has been done by using a SWAN model. SWAN is a numerical, third generation wave model that can be used to compute the wave transformation from deep water to nearshore, based on a wave action balance. It does not solve for individual waves, but only for wave spectra.

Description of the model

SWAN is run in stationary 2-D mode, since none of the parameters are time dependent. The built-in nesting technique is used in order to go from a large coarse grid to a fine smaller grid containing all points in the Malecón. The nesting technique is used with a rectangular grid and in three size steps. The coarse grid is 75 x 100 km with a cell size of 1000 m, the nested grid is 30 x 30 km with a cell size of 100 m, and the finest grid is 7.2 x 4.2 km with a cell size of 15 m. In this way it is still possible to calculate many different scenarios and the resolution is high enough to deal with local disturbances.

Using MATLAB and the data from the Historian Office a bottom grid for each run is created to exactly match the computational grid of the model. Land points were filtered out to arrive at a grid with only wet points. For the finest grid this led to some minor problems which were solved by changing the square grid to a smaller, rectangular grid.

A wind speed of 25 m/s is used for different wind directions, varying from -45 to 45 degrees. This is the maximum wind speed measured over a period of 37 years (Appendix H). All physical phenomena that are included and used during the computations are listed in Appendix J.

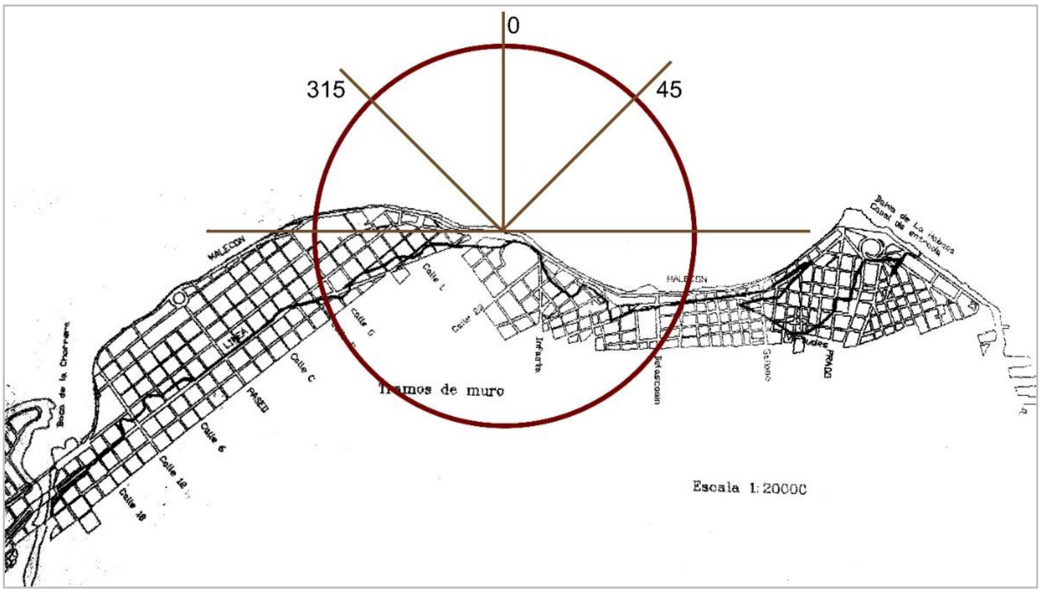

Figure 24: Different wind directions used in SWAN models

A JONSWAP spectrum is assumed at all sides since in this way no shadow zones are present and the wave climate is at least not underestimated (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015).A peak enhancement factor of 3.3 (default SWAN setting) is used and frequencies in the range of 0.03 Hz and 1.0 Hz are included, as is advised by the SWAN team (SWAN USER MANUAL, 2016) for hurricane conditions.

6.8. Forcing on the Malecón seawall

Before the preferred solution, a wall with recurve of height +4.46m (Buccino & Salerno, 2013), can be evaluated the forces and pressures to be applied must be determined. The pressure on the seawall is divided into two main parts; the pressure on the vertical section and the pressure on the recurve. This chapter will deal with these separately given the different approaches taken to determine the distributions. The pressure distribution on the preferred solution 7 is first analysed. By comparing the results of the physical model tests with other another study (Pearson, Bruce, Allsop, Kortenhaus, & van der Meer, 2005), the influence of the recurve on pressure distribution can be better understood. While the pressure profiles are based on the physical model tests for Sections 4 and 5, they serve as the most accurate and relevant source of information about pressures on the wall and will therefore be extrapolated to the other sections.

6.8.1. Comparing physical model tests to theory for walls with recurve

As a first verification a general comparison was made between the vertical wall of +3.96m and the curved wall of +4.46m. The pressure for each test was compared for the vertical wall and the curved wall for the upper two sensors. Only the top two sensors are taken into account as it is expected that these are the most likely to be influenced by the presence of the recurve and the measured pressures for these sensors governing in terms of magnitude. The black line indicates the expected load increase factor as described by (Pearson, Bruce, Allsop, Kortenhaus, & van der Meer, 2005), the data points represent the ratio of the pressure for the two wall types tested for the two sensors.

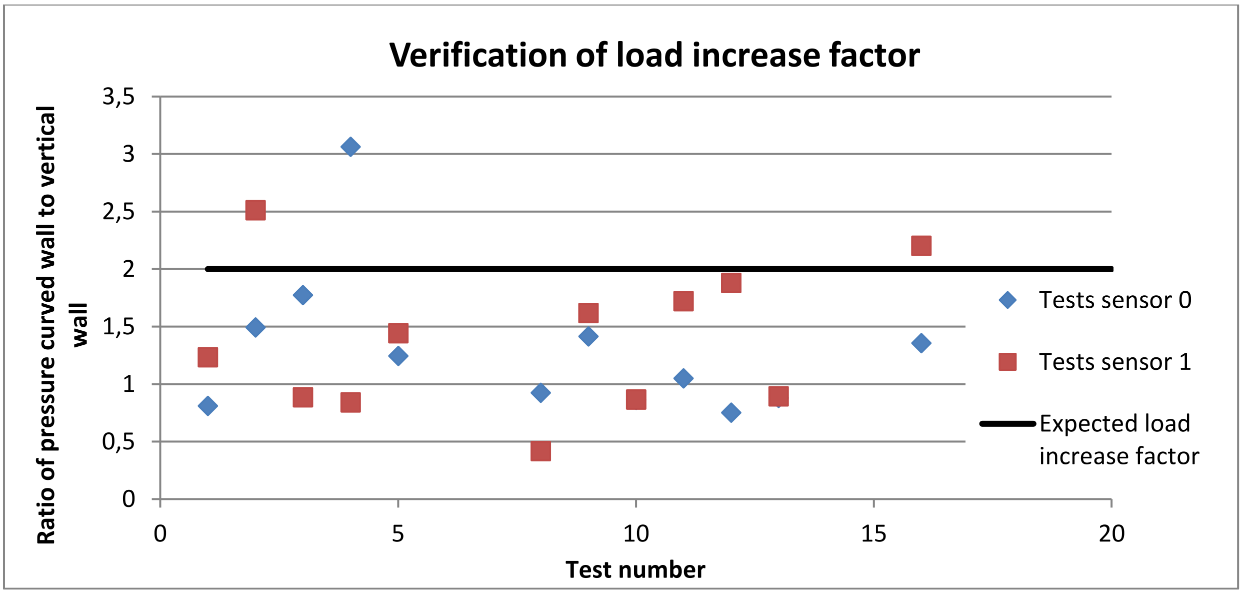

Figure 25: Ratio of pressures between curved wall and vertical wall

Figure 25 shows no clear relationship between pressure for the curved wall and the vertical wall; most of the points lie below the expected ratio of 2 but do not cluster in any significant way. While the physical model test results do not match the expected ratio, this does not indicate a contradiction due to the difference in crest heights of the two test structures compared above. This is due to the position of the sensor being at the same height for both tests and the same wave climate being used. As can be seen in Figure 26 the recurve begins at a distance of 0.43m from sensor 0. This means that the presence of the recurve is unlikely to have had any influence on the incoming waves thus resulting in the scattered results shown in Figure 25.

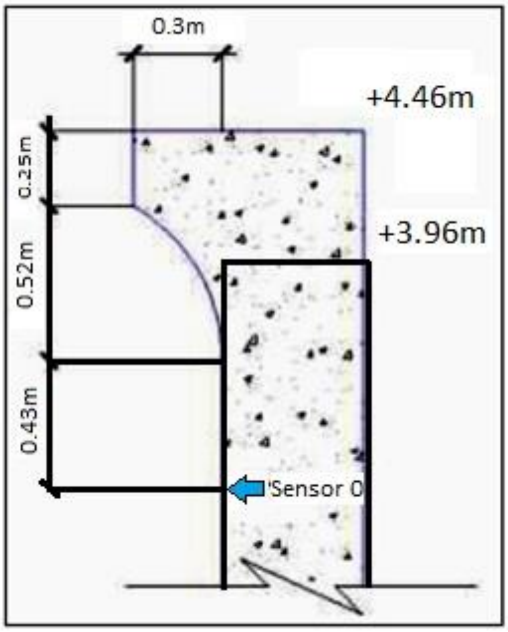

Figure 26: Position of sensor 0 in the physical model tests

What is of further interest, is whether there is a relationship between sea level during a storm, wave height, and the pressures acting on the seawall. Figure 27 shows the non-dimensional pressures on the curved seawall height +4.46m for increasing wave heights. The non-dimensional pressure is given as the measured pressure divided the density of water and significant wave height at the structure (p=p_max/(Hs_at_wall*1025). The blue lines indicate sea levels during Hurricane Wilma (2005) and the red lines refer to a storm with a return period of 50 years. It is interesting to note that increasing the wave height does not directly result in higher pressures acting on the wall. It is clear that for the storm conditions during Hurricane Wilma the governing significant wave height at 20m depth is 4m whereas for the 50 year storm the wave height of 2.7m produced the largest pressures. It is important to note that the actual pressures between these two storm conditions differ due to the conversion from non-dimensional to actual pressures.

It can be concluded that for Sections 4 and 5 the governing situation for pressure occurs for a storm surge of +2.28m and a significant wave height at 20m depth of 4m.

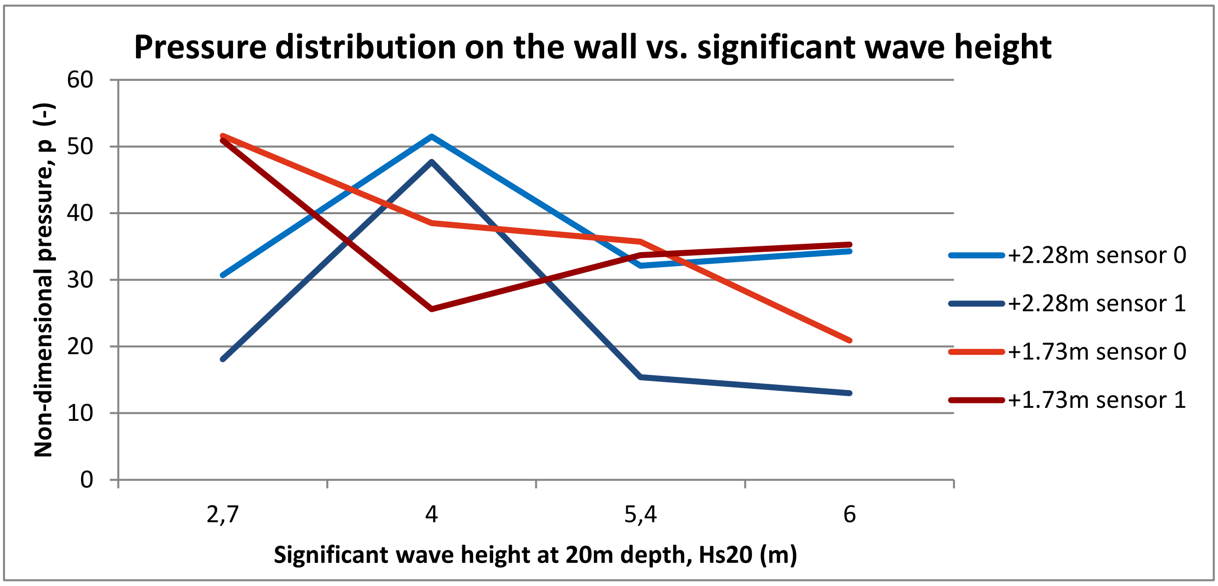

Figure 27: Pressure distribution on the wall for different significant wave heights

To determine the pressure profile not only for Sections 4 and 5, for which pressures can be determined directly from the physical model tests, but also for Sections 2 and 3 further analyses are required. The relationship between the wave conditions, in this case described by the ratio of the wave height at 20m depth and the water level at the wall, and the pressures was investigated. For every available combination of wave height and water level, as tested by (Buccino & Salerno, 2013), the pressure, both measured and non-dimensional, was plotted against this ratio. The results are shown in Figure 28. It is clear that neither plot shows a clear relationship between the wave conditions and the pressures on the wall, while surprising this simplifies the process for determining the pressure profile for other sections. The highest measured non-dimensional pressure is 51.50, which will be rounded off to 52 for simplicity in calculations and an added degree of safety; this value can be multiplied by density of water and significant wave height at the wall to determine the governing pressure at each section.

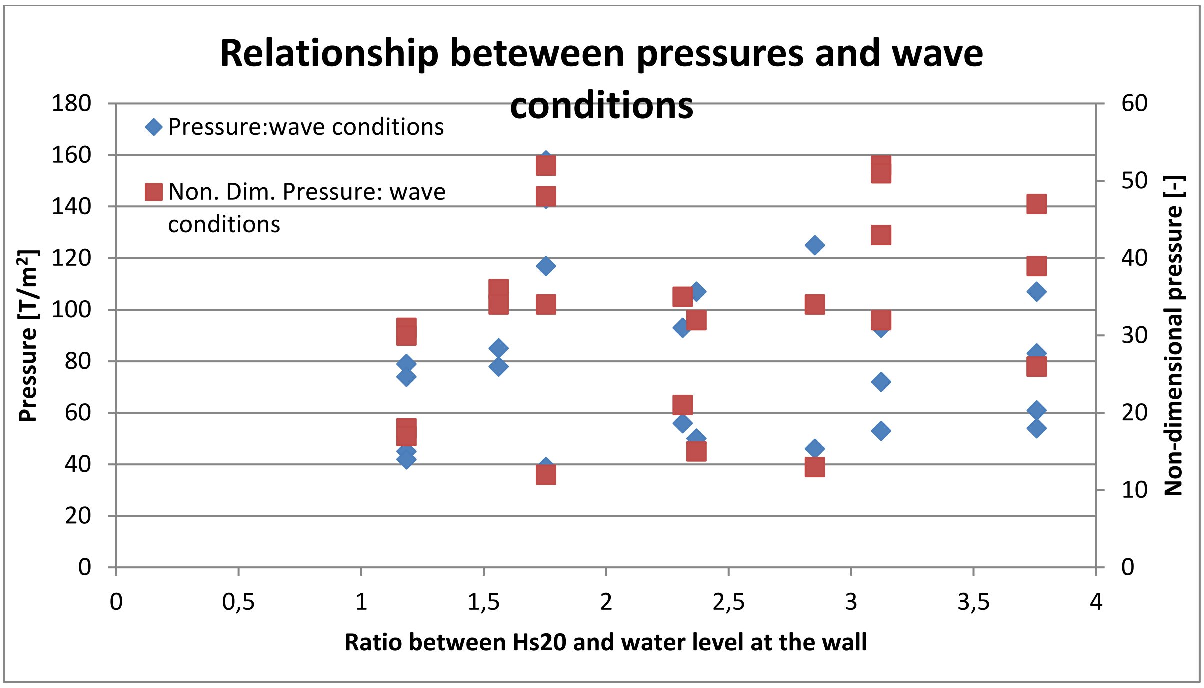

Figure 28: Relationship between pressures at sensor 0 and wave conditions

6.8.2. Pressure on vertical part of the wall

The pressure on the vertical part of the wall (below the recurve) can largely be determined based on the results of (L.F. Córdova et al, 2016) and (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015). The determined pressure distribution from these reports is shown in Figure 29 is a combination of physical model tests and the Goda formula for impermeable vertical seawalls.

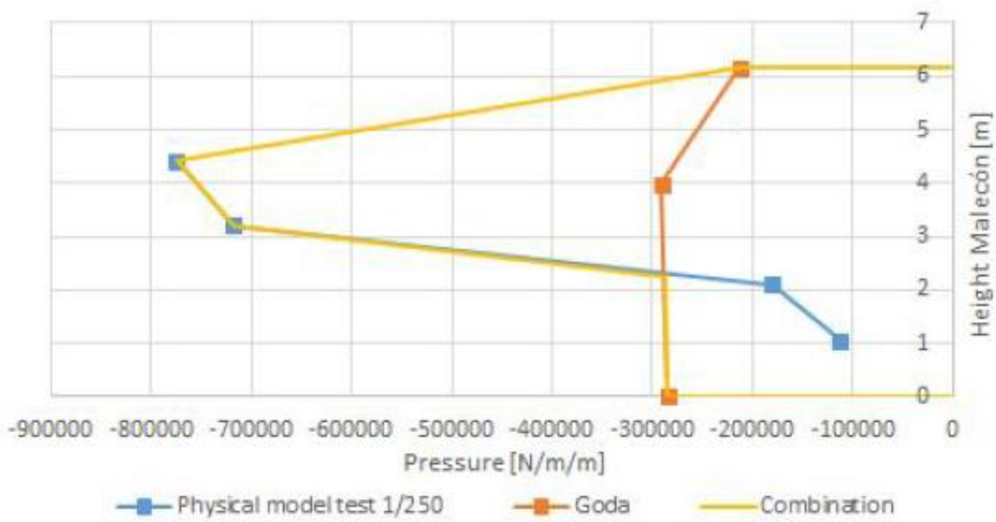

Figure 29: Pressures on a vertical wall +4.46m in ULS conditions (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015)

It is important to note that while the above pressure distribution is also based on the physical model tests results for a curved wall the values have been divided by 2 to correct for use on a vertical wall. These values are derived from Test 2 as shown in Appendix L for ultimate limit state (ULS) and from

Test 9 for 50 year return period. For both ULS and storm conditions with a 50 year return period the combination of pressures used to make the definitive profile is made using a probability of exceedance of 1/250. This is because the pressure profile according to Goda is determined using this probability and the values matching this probability must be applied from the physical model tests.

The pressure distribution to be used in ULS for the proposed solution is determined using the unmodified results from (L.F. Córdova et al, 2016) and the Goda stress distribution from Figure 29 multiplied by a factor 2 at the top to account for the recurve. Due to the small distance between sensor 0 in the physical model test, the last point for which the pressure is reliably known, and the end of the straight section of the wall, from which a different pressure distribution will be applied, this estimation of the pressure according to Goda will not lead to a significant inaccuracy relative to the total pressure distribution. The pressure profile at ULS due to wave impact on the wall of height +4.46m with recurve is given in Figure 30. As shown in the graph the red line indicating the pressure distribution according to Goda continues higher, up to 6.16m, than the straight part of the wall for the wall with recurve which ends at 5.39m. This is due to the fact that the Goda pressure is calculated using the full height of the wall, it was chosen to leave the data point at 6.16m in the diagram to better illustrate the calculation procedure. The pressure profile that will be applied in the structural analysis is the blue line labelled 'combination'.

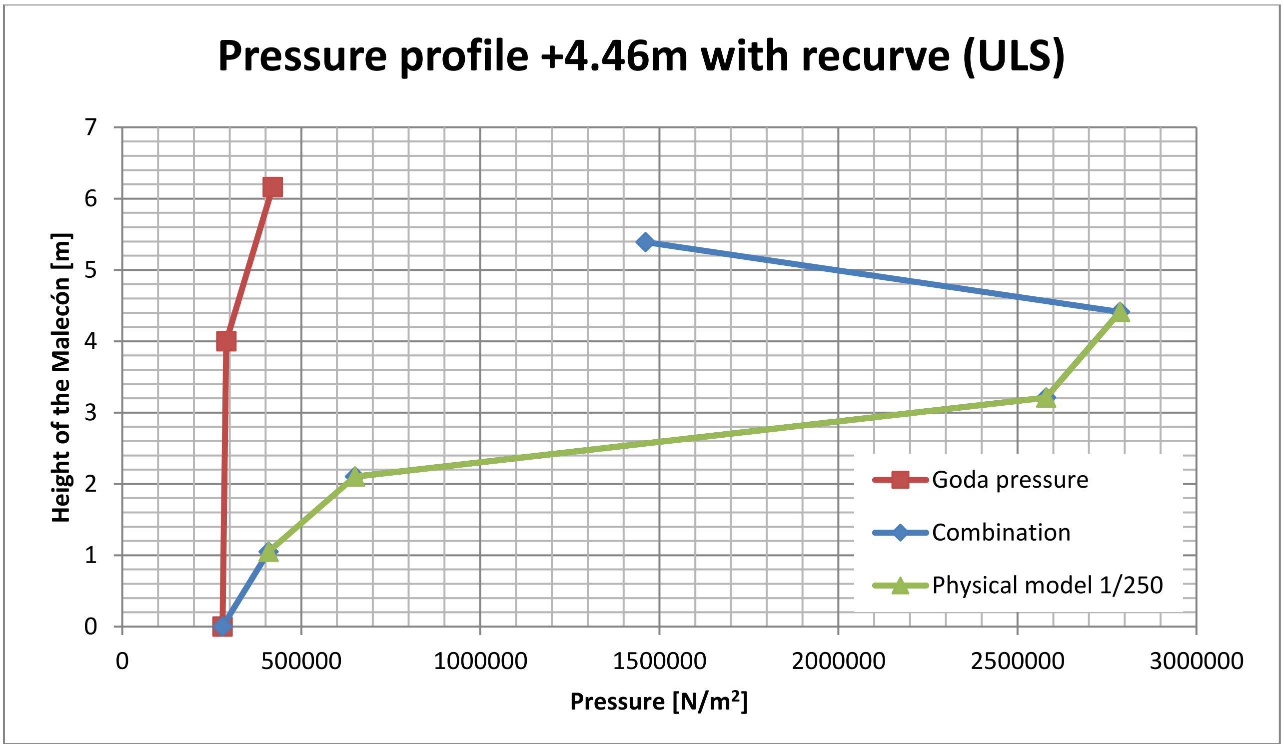

Figure 30: Pressure profile on the vertical section of the proposed solution at ULS

6.8.3. Pressure distribution on the recurve

The pressure distribution on the recurve has been determined based on the report of (Pearson, Bruce, Allsop, Kortenhaus, & van der Meer, 2005) which states that both horizontal and vertical pressure on a seawall and its recurve respectively should be increased by a factor $k_F \approx 2$. It is reported that for a "small" recurve, in the tests performed this consists of an overhang of 0.75m with respect to a wall width of 1m, the vertical pressures are only slightly lower than in horizontal direction. For "medium" and "large" overhangs the pressure are slightly higher. To make a conservative estimate it has been chosen to apply the same pressure over the recurve as the horizontal pressure at the top of the vertical section.

The shape of the distribution has been chosen based on observation of the physical model tests. As shown in Figure 30 the waves hit the wall and are forced backward (seaward) by the recurve. The result is that the force of the wave will be transmitted along the entirety of the recurve. The pressure will act perpendicular to the face of the recurve.

Figure 31: Wave action on the wall from physical model tests

7. Results wave modelling and ANSYS model analysis

This chapter describes the results from the wave modelling analysis in SWAN and the structural analysis from the ANSYS model. To calculate the wave overtopping in the different sections, the necessary parameters are found using SWAN. In the first and second paragraph the SWAN results are presented and the wave overtopping is calculated. Following the ANSYS analysis in the third paragraph.

7.1. SWAN Results

As stated before, the SWAN model calculates the significant wave height for the different wind directions and combinations. Table 6 indicates for each section and for each wind direction the governing combination together with the calculated wave overtopping. The more detailed results for the different directions and combinations can be found in Appendix M.

It was found that for all sections and for all wind directions combination 6 is governing. These are the hurricane Wilma conditions and have a return period of almost 300 years. As it was determined that a return period of 50 years would suffice (see Chapter 6.1), this combination as well as the 'zero' combination is left out of consideration. The wave overtopping for these combinations will still be calculated to give an indication of how much wave overtopping can be expected during such conditions.

Table 6: Governing combinations for wave climate

Direction (Clockwise)	**Section 2**		**Section 3**		**Section 4**		**Section 5**	
	Combi	**Overtopping**	**Combi**	**Overtopping**	**Combi**	**Overtopping**	**Combi**	**Overtopping**
315.0°	2	1.065 $m^3/s/m$	1,2	0.376 $m^3/s/m$	1	0.417 $m^3/s/m$	1	0.432 $m^3/s/m$
0.0°	**1**	**1.149 $m^3/s/m$**	**1**	**0.573 $m^3/s/m$**	**1**	**0.541 $m^3/s/m$**	1	0.636 $m^3/s/m$
45.0°	1	0.761 $m^3/s/m$	1	0.479 $m^3/s/m$	1	0.361 $m^3/s/m$	**1**	**0.644 $m^3/s/m$**

From calculations it becomes clear that the wind 0° direction and wave conditions as given in combination 1 form the governing scenario for sections 2, 3 and 4. For section 5 the governing situation is the scenario with wind coming at an angle of 45° however the difference with the 0° scenario is about 4%, since the error is small and for convenience the scenario used for the other sections is extended to 5.

In Figure 32 the significant wave height in the Havana bay is plotted. Other more detailed figures about wave height and wave setup can be found in Appendix N.

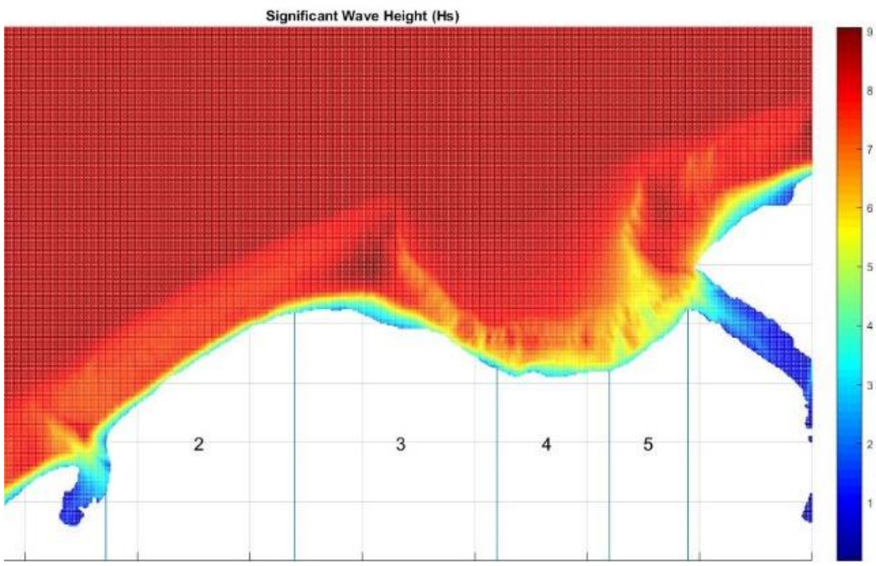

Figure 32: Significant wave height in Havana Bay

7.2. Calculation of the wave overtopping

In order to create a final design that takes into account local differences, the Malecón is divided in 25 subsections. Each subsection has a length of 250 meters and it is assumed that wave conditions, wall height and depth profile are constant over the subsection. In reality this is not the case, but in order to come up with a solution a certain resolution had to be assumed. Figure 33 shows this subdivision of sections.

Figure 33: Subdivision of the sections

To calculate the wave overtopping for the current state during design conditions the formula from the physical model tests is used. Combination 1, the governing one, gives a significant wave height of 6.52 meters offshore and a water level elevation of 1.95 m. With the use of SWAN the near shore significant wave height for each subsection was calculated and put into the calculations. In Table 7 the values of overtopping are given for the different subsections. More detailed results can be found in Appendix O.

Table 7: Wave overtopping volumes in the subsections

Section	Sub Section	q Overtop l/s/m
2	1	1944
	2	1593
	3	1721
	4	1619
	5	1265
	6	681
	7	953
	8	425
3	1	410
	2	215
	3	206
	4	248
	5	442
	6	1445
	7	1160
	8	372
4	1	311
	2	352
	3	422
	4	590
	5	744
5	1	414
	2	468
	3	452
	4	499

From Table 7 it becomes clear that section 2 is most critical with overtopping values reaching a maximum of almost 2 $m^3/s/m$. When considering the demand of 0.05 $m^3/s/m$, it can be concluded that serious reduction measures are needed in this section. One remark is that in section 2 the current wall is relatively low, which results in a rapid increase in the amount wave overtopping.

7.3. ANSYS model results

This paragraph contains the results of the structural analysis of the proposed curved wall. A detailed explanation of the setup of the model is described in Appendix R. The results for the structural analysis of the proposed solution at ultimate limit state are shown in the figures below. These will be used to design reinforcement and determine necessary structural properties of the dowels.

7.3.1. Maximum principal stresses

The maximum principal stresses shown in Figure 34 are the governing tensile stresses in the structure which will be used to determine the principal (longitudinal) reinforcement in the new wall and check strength of the steel dowels as well as the bonded connection with the epoxy grout used to fix them in place. The governing tensile stress in the new wall occurs between the old wall and the concrete with a value of 5.3 MPa due the bending of the crest around the concrete body.

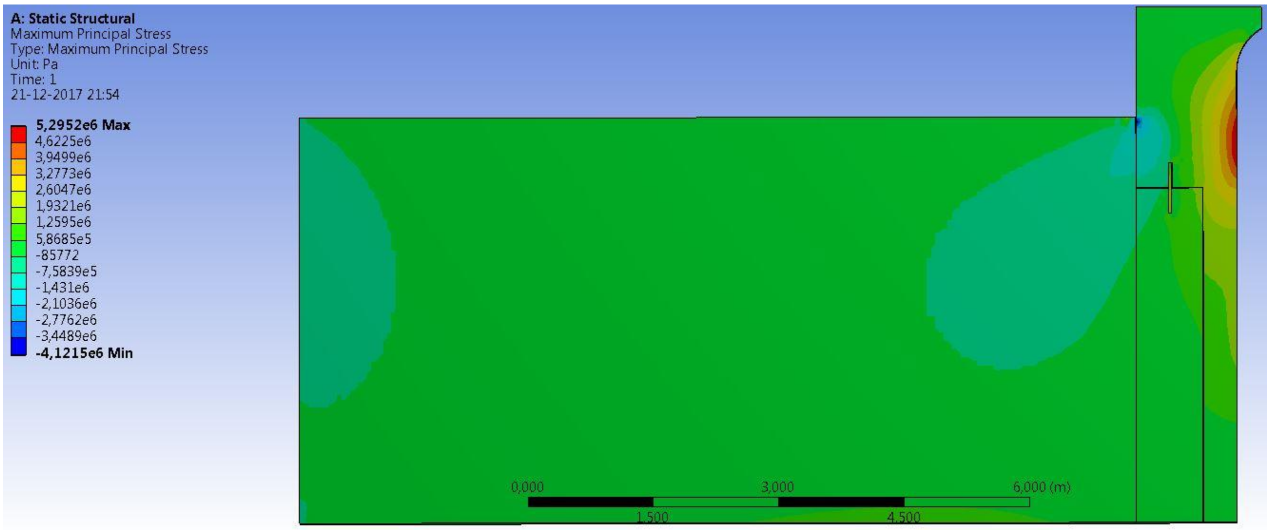

Figure 34: Maximum principal stresses in the structure at ULS

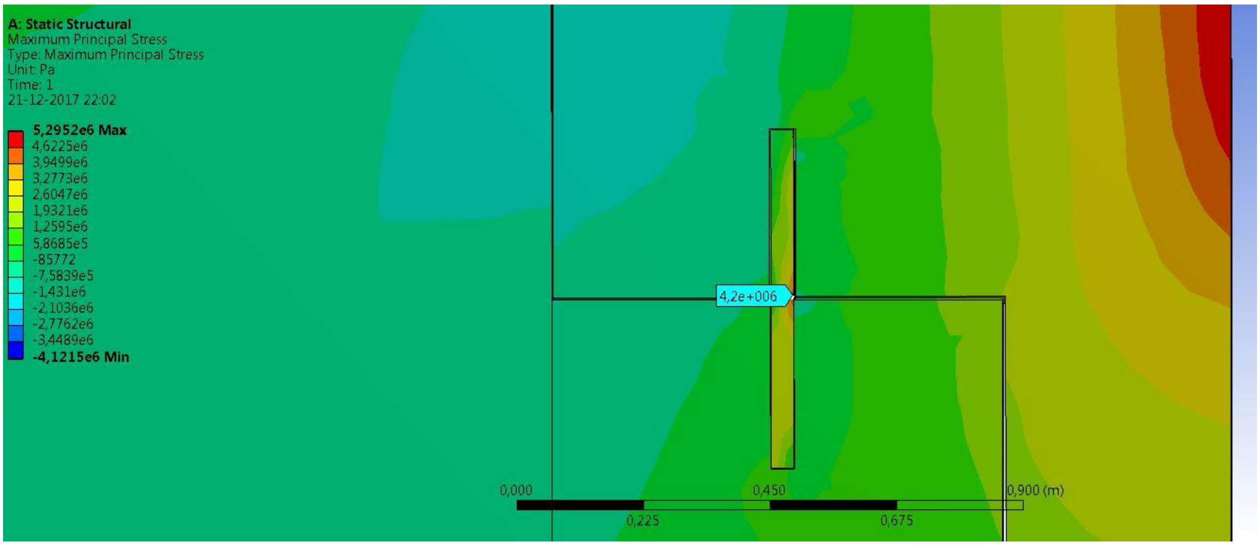

Figure 35: Maximum tensile stress in the dowel at ULS

The maximum tensile stress in the dowel, shown in

Figure 35, is 4.2 MPa which is far below the ultimate tensile strength of all grades of structural steel. This value will also determine the necessary number and dimensions of dowels to be used in order to ensure a strong bond between the dowels and the new wall. The bond will be made of epoxy grout.

7.3.2. Minimum principal stresses

The minimum principal stresses shown in Figure 36 are the governing compressive stresses in the structure which will be used to check that the ultimate strength of the concrete is not exceeded. The governing compressive stresses occur at the contact point between the new wall and the concrete body reaching values of 12.5 MPa and 12 MPa respectively. The allowable concrete compressive stress is not exceeded anywhere in the new wall but some crushing may occur around the edge of the concrete body. It is recommended to remove this top layer due to existing damage and to replace it with a higher strength concrete, a weak concrete such as C20/25 is sufficient to reach the necessary compressive stress.

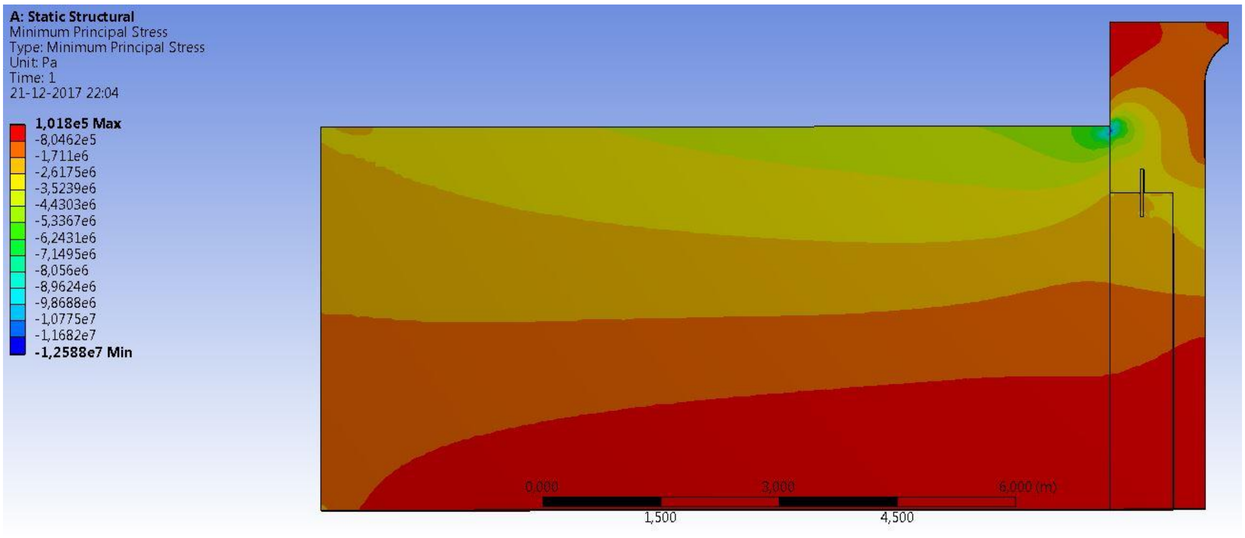

Figure 36: Minimum principal stresses in the structure at ULS

7.3.3. Maximum shear stresses

The governing shear stresses are 4.8 MPa and 5.1 MPa in the new wall and the concrete body respectively, these occur in the contact area between the concrete body and the new wall as shown in Figure 37.

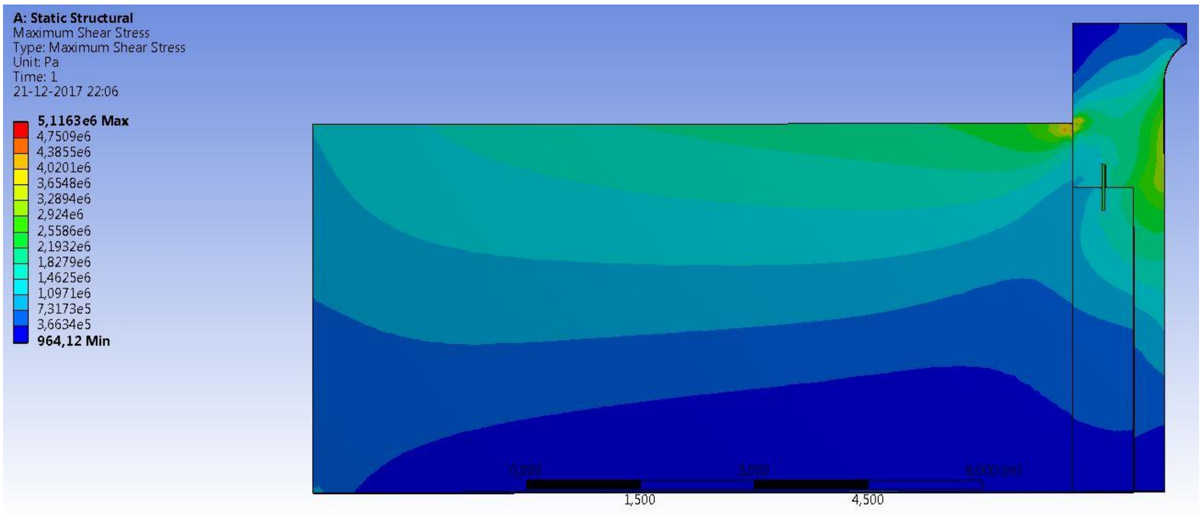

Figure 37: Maximum shear stresses in the structure at ULS

The shear stress capacity of the concrete body is 0.15 MPa and that of the new wall is 0.43 MPa. It is clear that shear stresses will cause significant issues in the design if proper measures are not taken. The amount of reinforcement necessary depends on the primary reinforcement applied in the new wall and will be discussed in the chapter 'Detailed design'.

7.3.4. Vector principal stresses

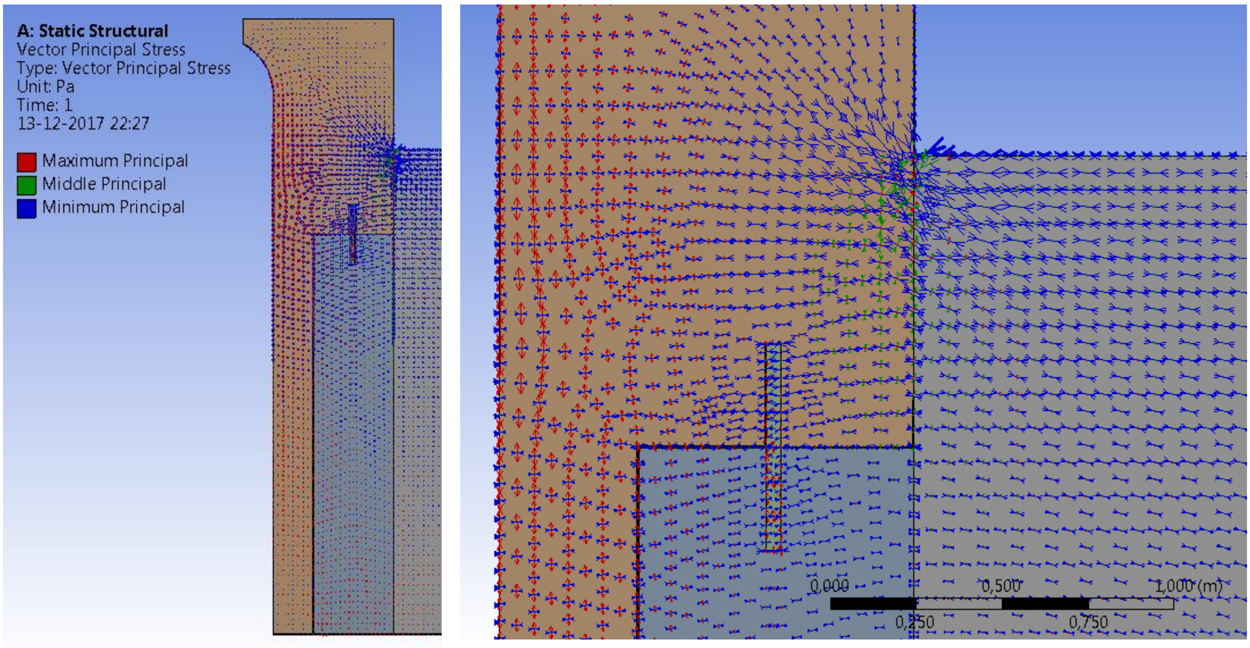

Figure 38: Vector principal stresses in the structure and a detail of the stresses above the old wall

The vector principal stresses in the structure give an indication of how the stresses flow within the structure; this is useful for determining where reinforcement can be placed most effectively. The vector principal stresses are given in Figure 38, blue arrows indicate compressive stresses and red arrows indicate tensile stresses. In areas where more and larger red arrows can be seen reinforcement must be applied, in the same direction, to take tensile forces.

8. Design alternatives

Since 1995, numerous studies have been conducted on the different parts of the Malecón in order to find solutions to the problem of wave overtopping. In these studies four different types of alternatives surface as feasible solutions, namely; structural fortification of the vertical wall, a curved wall, a berm, and a breakwater. These solutions are however not ready for execution and require further detailing. Sub-options will be discussed in this chapter and a decision will be made with the use of a multi criteria analysis. The sub-options will entail construction method of the wall, type of berm, and type of breakwater.

8.1. Structural fortification

The first alternative for strengthening the Malecón is to improve the current wall with structural fortification. In 2015, TU Delft Students concluded that a partial replacement method is the best alternative to fortification the structure (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015). With this method the top part of the old wall will be replaced with a new and higher part, together with an additional segment in front. While this alternative addresses the issue of overtopping the new wall would be substantially higher and with a bigger cross section than the current wall. Therefore it does not meet the boundary conditions, particularly for allowable added height from the sidewalk, and is not feasible.

8.2. Curved Wall

The second alternative is constructing a wall with recurve. The recurve in the wall directs the waves backwards to the sea instead of up and over the wall. This solution has been studied extensively by Professor Córdova and the CIH by physical model tests. In these studies, both single curved and double curved walls were tested. Both versions have the same effect on reducing overtopping however the single curved seawall is easier to construct and requires less material. Therefore this option is more effective.

In 2016 Professor Córdova performed another study for the wall with recurve (Córdova López, et al., 2016). This time the vertical wall of the current situation of section 4 and 5, with a height of 3.96m, and a wall with recurve which was 0.5 meter higher than the current situation were compared. The result of the study was that the curved wall significantly reduces the overtopping with a relatively small increase in the height of the wall. The seawall with recurve was chosen by previous research groups as a viable option for section 2 of the Malecón due to its relatively low production costs (Muilwijk, Versmissen, Meijer, Groenendaal, & Veenstra, 2003). The wall can be constructed in three ways: in situ, prefabricated or a combination of both.

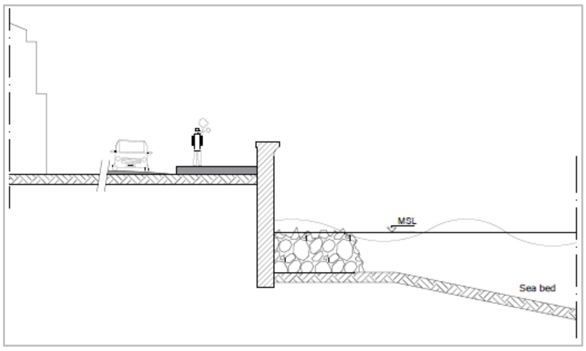

Figure 39: Cross-section with wall with recurve

Constructing in situ entails that the wall with recurve will be constructed on site with moulds in which concrete will be poured; this is similar to how the original wall was constructed between 1900 and 1950. Advantages to this method are its ability to cope with variations in site conditions and geometric flexibility. Disadvantages are fluctuations in quality, required working conditions and high labour demand. If prefabricated, the curved wall will be produced offsite, transported to the site and placed at the required position. Advantages of this method are cost reduction for repetition, relative fast construction time, less vulnerable to water impact, and continuous quality and reduction of errors. Disadvantages are related to the large variations in site conditions, required dimensions, and a cold connection between the existing structure. A combination of constructing the wall in situ and partly prefabricating elements beforehand offers the opportunity to exploit the large amount of repetition while being able to cope with the variations on site conditions and dimensions. The goal would be to standardise as much as possible to take full advantage of prefabrication with as few moulds as possible. With the limitations put on the seawall for the maximum allowable height of 1.25 meters above the sidewalk solutions for measures behind the wall have to be included.

8.3. Berm

The third alternative is the construction of a berm near the shore. Waves will break on the berm instead of breaking against the seawall thus reducing wave impact. In 2015, DUT students concluded that for sections 4 and 5 a berm is more economical viable than a breakwater (la Gasse, van Rooij, Smits, Ton, & Velhorst, 2015). Three permeable berms with different height and width were studied by (Córdova López, et al., 2016). The three berms were modelled with a vertical wall and a wall with recurve. The result of the test was that a wall with recurve without berm is more effective than a vertical wall with any of the berms tested. Furthermore, the shortest berm is the most viable option as it is only slightly less effective than the other types and needs less material and is therefore cheaper to produce.

Table 8: Types of berms tested by Professor Córdova in 2016

Type	Height	Width	Slope
1	3.28m	5m	1:1.5
2	2.28m	20m	1:1.5
3	1.73m	30m	1:1.5

Berms can be constructed using concrete, rubble, or a combination, and be made permeable or impermeable. A permeable structure has a higher wave reduction while an impermeable structure is easier to construct and is more robust. Construction of rubble berms can be done from two sides, so both land-based and waterborne equipment can be involved (Schiereck & Verhagen, 2012). Where concrete is to be used a dry dock is necessary for construction. It is also possible to construct a rubble berm with concrete cubes. Concrete is relatively cheap and easy to obtain in Cuba, whereas rubble is harder to obtain. The top layer requires large elements which would be especially challenging to obtain. For this reason only permeable and impermeable concrete berms will be examined in this study. Figure 40 schematically illustrated the berm according to the dimensions of Type 1.

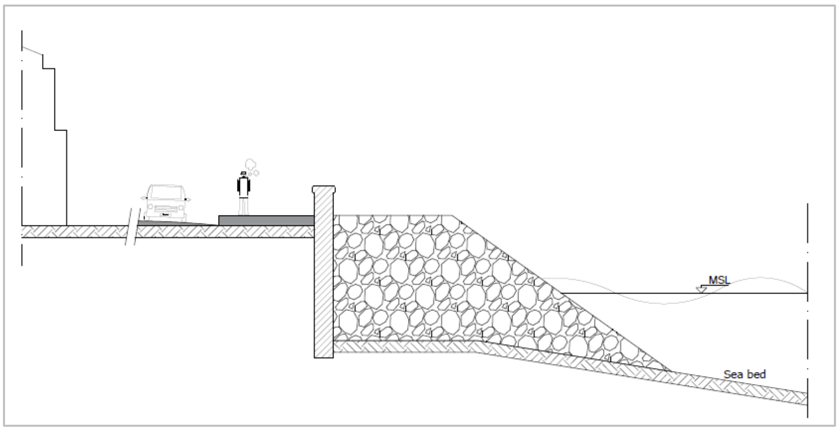

Figure 40: The berm alternative dimensioned in according to Córdova (2016)

8.4. Breakwater

Previous studies conclude that the construction of a breakwater is the most expensive alternative of the four. In 2006, DUT students concluded that a submerged breakwater may be more expensive than a berm, but it also matched better with the list of demands and therefore has a higher value than a berm (Baart, van Kruchten, McCall, & van Nieuwkoop, 2006). The 2016 study of Professor Córdova also contained a study of breakwaters in combination with a wall with recurve (Córdova López, et al., 2016). Similarly as for berms, a breakwater in combination with a wall with recurve is more effective than with a vertical wall. Several options regarding the construction of breakwaters are available.

The first assessment for the type of offshore breakwater to be applied is the choice between submerged or emerged. In previous studies it was concluded that an emerged breakwater is a more effective option (la Gasse, van Rooij, Smits, Ton, & Velhorst, 2015). Secondly, the construction type of the breakwater has to be taken into account. Options include a monolithic breakwater consisting of a rectangular shaped caisson filled with water, sand, or rock and with a rubble, rocks with various sizes, foundation filled. This functions as a vertical impermeable block. The second option is a rubble mound type breakwater consisting of loose elements of various layers and sizes. Lastly, a combination between a monolithic element, caisson, and a berm of loose elements is possible. Due to its positive effect on water exchange and higher stability a permeable breakwater is favourable.

A rubble mound low crested breakwater is concluded as the best option by research from (Baart, van Kruchten, McCall, & van Nieuwkoop, 2006) and (la Gasse, van Rooij, Smits, Ton, & Velhorst, 2015), this is schematically illustrated in Figure 41.

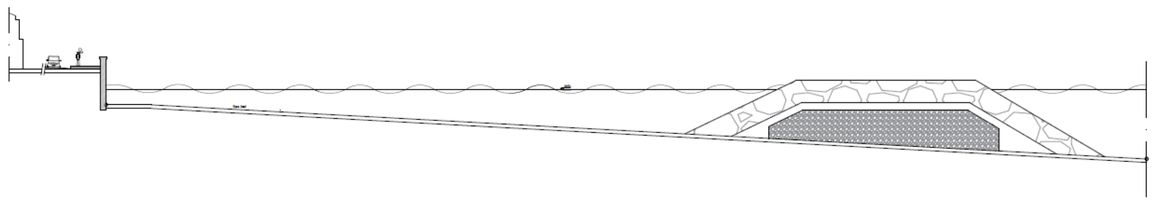

Figure 41: The low crested emerged breakwater alternative

The other possibility is to place emerged breakwater closer to shore. The last consideration is between the material of the elements which consist either of rock, concrete, or a mixture of both elements. This would depend on the availability and required volume of the material in the vicinity of Havana. Any type of breakwater would require large elements in the armour layer which are not easy to obtain. A final alternative is the use of shipwrecks as a breakwater, which is not taken into account in this assessment due to lack of information. It could be an addition for the surrounding for the view and diving possibilities for new opportunities in tourism.

9. Multi Criteria Analysis

In order to compare the alternatives, a multi criteria analysis will be used. In order to find the best alternative, a well-designed multi criteria analysis needs to be conducted. In order to so, the following process was used.

In order to conduct a MCA, alternatives need to be compared based on criteria with a certain ranking system. To determine these criteria, weightings and ranking system, a draft version was first created. This draft version was then evaluated by individual project members, which provided feedback. This feedback was incorporated into the MCA, after which the MCA design was send for a last evaluation by Prof. Córdova.

The analysis itself was done two times levels independently by two team members. Hereafter, the results were compared. Each result was debated and in case of uncertainty, an expert team member regarding the subject was consulted.

In order to get an optimal result, two MCA's were conducted on different levels. The first MCA: Alternatives compares alternatives of a certain category with each other. Curved wall type alternatives were compared with each other, as well as berm and breakwater alternatives. this was done in order to find the best alternative of each type.

The second MCA compares integrated solutions for the whole Malecón instead of single alternatives. The reason for this is that with the current boundary conditions, none of the alternatives can meet the safety requirements alone.

9.1. Criteria and weighting

This paragraph will evaluate the several options of these previously established solutions with a multi criteria analysis. The criteria are derived from the design criteria listed in Chapter 5. The alternatives are rated on a 5-point scale for each criterion where 5 and 1 indicate the best and worst scores respectively. Overall, there are four criteria. Each of these criteria is divided into two or three sub-criteria. Each criterion counts for a certain percentage of the total mark an alternative can get. The first paragraph will further elaborate the criteria and weighting of these factors.

9.1.1. Safety

Protection against wave overtopping is the primary goal of the project therefore it has a high weighting with safety accounting for 25% of the total score. The criterion safety is divided into the sub-criteria reduction wave overtopping and structural integrity of the structure. The reduction of wave overtopping is a key part of the study; therefore it has a weight of 70% in the criterion safety. Reduction of wave overtopping is measured as a percentage reduction compared to the current situation.

Table 9: MCA scoring sub criterion: wave overtopping

Score	Reduced overtopping
5	81-100%
4	61-80%
3	41-60%
2	21-40%
1	0-20%

Structural integrity is defined as the integrity of the Malecón coastal defence system over the design lifetime. The current wall is damaged and shows signs of severe deterioration. This affects the safety of the hinterland as the structure might not provide sufficient protection during extreme weather events. The structural integrity of the seawall has a weight of 30% of the criterion safety.

Table 10: MCA scoring sub criterion: structural integrity

Score	Structural Integrity
5	Very positive effect on the structural integrity
4	Positive effect on the structural integrity
3	No effect on the structural integrity
2	Negative effect on the structural integrity
1	Very negative effect on the structural integrity

9.1.2. Costs

There is not a fixed budget put out by the parties responsible for the Malecón and costs will be calculated on the basis of estimates provided by (Centro de Información de la Construcción, 2005). Financial resources in Cuba in general are scarce; alternatives which are relatively cheap to produce are therefore favoured. The costs of the project count for 25 % of the score. Project costs can be divided into two separate types: direct building costs and maintenance costs. Both these sub criteria will be scaled on a scale of very inexpensive to very expensive based on estimations. Building costs have a weighting of 70% and maintenance of 30%. After the MCA a detailed cost estimation will be conducted on the selected alternative(s).

Table 11: MCA scoring sub criterion: direct costs & maintenance costs

Score	Building costs/maintenance costs
5	Very inexpensive
4	Inexpensive
3	Normal
2	Expensive
1	Very Expensive

9.1.3. Social and environmental factors

There are several social and environmental aspects related to the Malecón coastal defence system, which are mainly of great importance to the Historian office. As this is one of the problem owners, these aspects are important criteria on which the alternatives can score. A weight of 25% is given to the social and environmental criterion. This criterion is divided into three sub-criteria: social attractiveness, characteristic view and effect on environment.

As stated before, the Malecón serves as a social meeting place for both locals and tourists. The wish of the Historians office is that the Malecón stays attractive for such social activities. The social attractiveness can be determined by the amount of stimulation or hindrance of social and/or tourism activities as a result of an alternative. This sub-criterion has a weight of 40% on the criterion social and environmental.

Table 12: MCA scoring sub-criterion social attractiveness

Score	Social attractiveness
5	Very attractive
4	Attractive
3	No effect
2	Not very attractive
1	Not attractive

Both for the Historians Office and UNESCO, the characteristic view of Havana Vieja and the Malecón is an important aspect and therefore has a weight of 40% of the criterion social and environment. This characteristic view might diminish due to the construction of alternatives. None of the alternatives in this study have a positive effect on the characteristic view. Therefore, only the degree of the negative effect will be scored.

Table 13: MCA scoring sub-criterion characteristic view

Score	Effect on characteristic view
5	No effect on the characteristic view
4	Slightly negative effect on the characteristic view
3	Slightly negative on the characteristic view
2	Negative effect on the characteristic view
1	Very negative effect on the characteristic view

Environmental criteria are difficult to quantify and are not a main priority to the involved parties. However a severe reduction in water quality is unacceptable for instance due to build-up of sewage water as a consequence of breakwaters. Furthermore, negative effects on the environment may then affect the social attractiveness and characteristic view. None of the alternatives in this study have a positive effect on the environment. Therefore, only the degree of the negative effect will be scored. The effect on the environment receives a weight of the remaining 20% of the criterion social and environment.

Table 14: MCA scoring sub criterion: effect on environment

Score	Effect on environment
5	No effect on the environment
4	Slightly negative effect on the environment
3	Slightly negative on the environment
2	Negative effect on the environment
1	Very negative effect on the environment

9.1.4. Implementation

The implementation of the alternatives makes up 15% of the total score. The feasibility of an alternative depends on the limitations of the project site and the limitations of available materials and equipment in Cuba. Site limitations include tidal patterns, storms, high water, the current wall and berm. For instance, tidal patterns and storms might make it more difficult to construct a breakwater, and the existing berm might make the construction of seawall harder. Site limitations have a weight of 50% on the criterion implementation.

Table 15: MCA scoring sub criterion: site limitations

Score	Site limitations
5	No site limitations
4	Slight number of limitations on site
3	Some site limitations on site
2	Many limitations on site
1	High number of site limitations

As a result of the embargo, some equipment and materials are harder to obtain in Cuba, this can affect the implementation of certain alternatives. The future of the embargo remains uncertain and may continue to affect the project. Furthermore, the Cuban culture prefers using its own labour, equipment and materials, which will lower the costs of the project. Therefore, alternatives which can be constructed with local equipment and materials are preferred. The availability of materials and equipment has a weight of 50% on the criterion implementation.

Table 16: MCA scoring sub criterion: Availability of materials and equipment

Score	Availability of materials and equipment in Cuba
5	Equipment and materials very easy obtainable in Cuba
4	Equipment and materials easy obtainable in Cuba
3	Equipment and materials obtainable with some effort in Cuba
2	Equipment and materials difficult to obtain in Cuba
1	Equipment and materials very difficult to obtain in Cuba

9.1.5. Construction

The last criterion is the criterion construction. Construction is divided into the sub-criteria construction time and reduced safety during construction. Longer construction time leads to a higher probability of severe weather affecting construction. It is preferred to select an alternative which takes less time to construct and implement. The criterion will be measured by estimated construction time compared to other options. Construction time has a weight of 50% on the criterion construction.

Table 17: MCA scoring sub criterion construction time

Score	Construction time of alternative to each other
5	Short construction time
4	Relative short construction time
3	Mediate construction time
2	Relative long construction time
1	Long construction time

The second sub-criterion in this category is reduced protection during construction. This counts for the remaining 50% of the criterion construction. Construction of alternatives might affect the protection of the Malecón during this construction period.

Table 18: MCA scoring sub criterion reduced protection

Score	Reduced protection during construction
5	No reduced protection during construction
4	Slight reduction of protection during construction
3	Some reduced protection during construction
2	Reduced protection during construction
1	Greatly reduced protection during construction

In Figure 42 the result of the weightings is shown. The weighting of the criteria multiplied by the weighting of the sub criteria gives the weight of the sub-criteria on the total score.

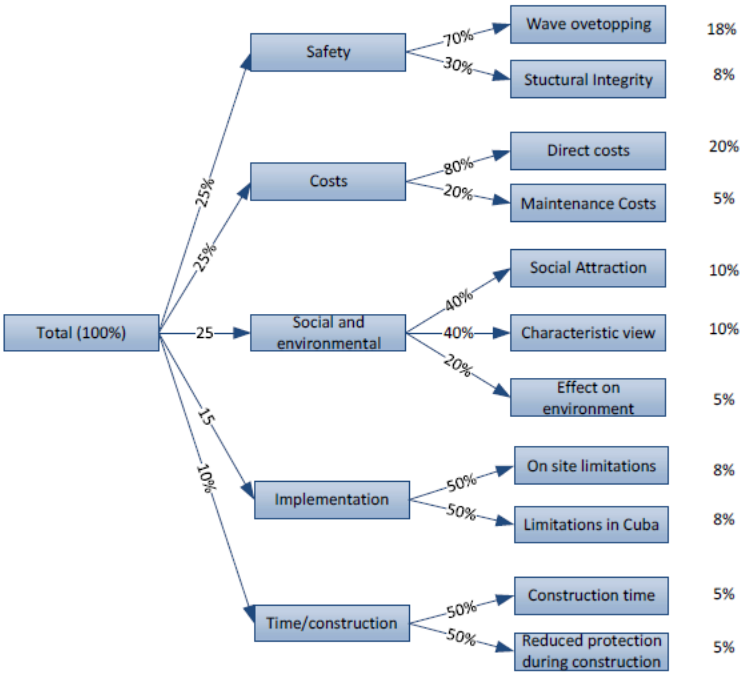

Figure 42: Weighting of criteria for the MCA

9.2. Results of the Multi Criteria Analysis: Alternatives

From the multi criteria analysis it can be concluded that the favourable alternatives are: a wall with recurve combination construction, a permeable berm and, an emerged breakwater. The results of the MCA are shown in Appendix S. The sub options differ only slightly for some criteria because they are relatively comparable. For the wall with recurve this depends for instance on quality and construction limitations whereas for the breakwater the costs and safety influence the decisions due to variation in size, effectives, appearance, and positioning. Further detailing the design and modelling of wave overtopping will be conducted to determine the reduction per option.

9.3. Integration of solutions

The research on the existing situation, hydraulic- and structural conditions, design options, costs, and modelling of wave overtopping for various situations results in the integration of the solutions. On the basis of the outcomes from the previous sections a detailed consideration has been made as to which options have to be applied in which areas of the Malecón coastal defence system. These decisions are based upon the decision tree illustrated in Figure 43. Since many factors have to be taken in to account it is decided to work out four alternative proposals. These will be tested in a MCA, giving the most favourable solution.

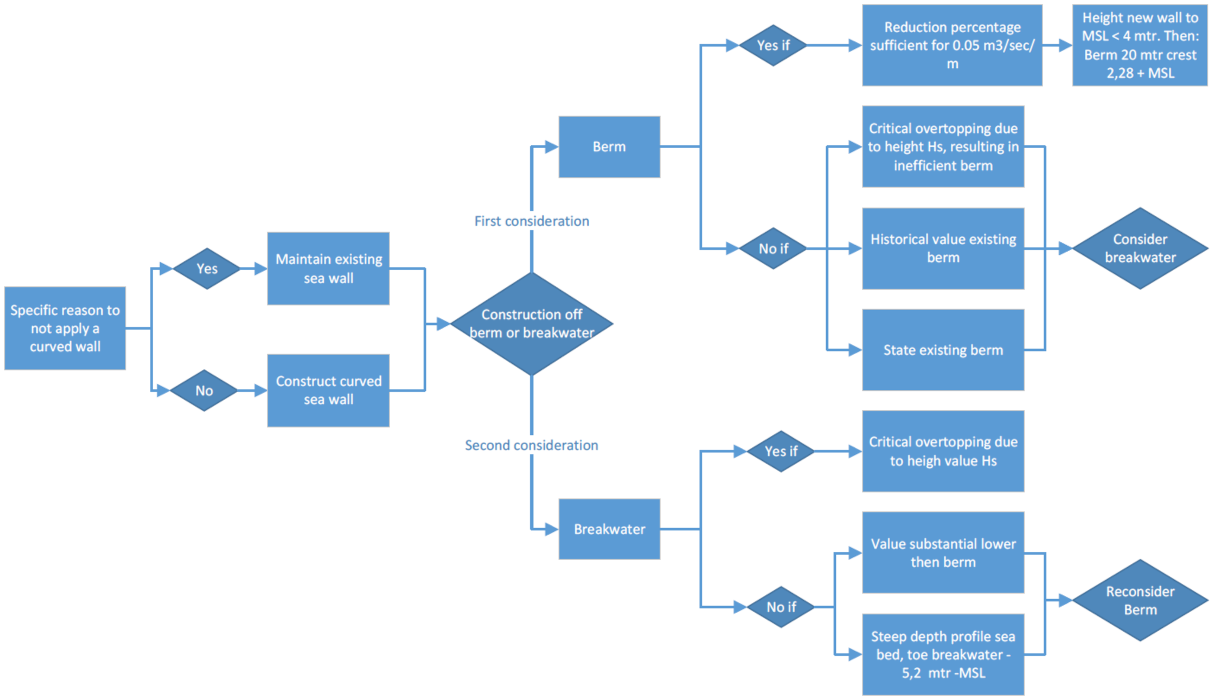

Figure 43: Decision tree for deciding which measure to apply

Not applying the curved wall is only considered if there is a very specific reason not to. Since it is within the boundary conditions and its effectiveness in reduction, there is enough reason to apply it in any section. Two boundary solutions have been determined which have been assessed as not feasible or insufficient but serve to better focus the analysis.

The first solution is the application of only the raised curved sea wall, preserving the characteristics and view of the sea as much as reasonable possible. This results in an average reduction of 47.1 % over the Malecón and a total average overtopping of 0.312 m^3/sec/m compared to 0.670 m^3/sec/m existing overtopping. This value is deemed insufficient in comparison with the goal of 0.05 m^3/sec/m. The second boundary solution fulfils the stated goal of overtopping reduction, resulting in an average of 0.038 m^3/sec/m. This solution realises maximum reduction by constructing a breakwater and curved wall over the entire 6 km stretch of the Malecón from section 2 till 5. This solution is not financially feasible, due to the high costs and impact of a breakwater and has a severe impact on the characteristic view of the sea and the environment. Therefore, it is concluded that a solution between these 2 boundaries has to be found but the goal of 0.05 m^3/sec/m cannot reasonably be met. The other alternative solutions are explained further.

9.3.1. Critical wave overtopping

The first alternative focuses primarily on reducing the wave overtopping at the sections with the highest overtopping values. This is realised by constructing berms and breakwaters at all critical sections. Where a berm does not have sufficient effect, overtopping values over 0.2 m^3/sec/m, a breakwater is constructed. This approach results in a reduction of 84.3 % in comparison to the current situation. This does not take into account the site limitations while constructing a breakwater, such as steep sea bed slopes. Therefore, this is an expensive solution which also does not focus on the preservation of the characteristics.

9.3.2. Alternating berm and breakwater

This alternative is based on the principle that of alternating between a berm and a breakwater. By alternating it is assumed that the breakwaters still influence wave energy dissipation, diffraction and reduction in the sections in between. By alternating the costs will be lower in comparison to a full breakwater and have less impact on the view and environment. This results in a 79.1 % reduction from the existing situation but is relatively expensive as well.

9.3.3. Lowest costs for highest reduction (Value)

This alternative aims to maximise the value of the solution meaning achieving the highest amount of reduction for the lowest amount of costs and maximising use of the existing berm. The focus lies mainly on the overall average amount of overtopping by applying measures only where their effect is essential. In sections with a relatively low significant wave height it is more effective to place berms in terms of value. If the berm option with respect to the curved wall realises an additional reduction of $0.08 m^3/sec/m$ it is applied. This results in an average reduction of 61 % in wave overtopping with a relatively low impact on characteristics and view of the ocean at the lowest costs.

9.3.4. Combining forces

The fourth alternative consists of compromises on the first three options. It is a good representation of the decision tree and making compromises to fulfil the boundary conditions. It achieves a relatively high average reduction of 79.8 % compared to the existing situation and aims to tailor a solution per section taking into account the value, depth profile, characteristics, and maximise use of existing situation. This entails four breakwaters with a total length of approximately 2.5 km and up to 2.5 km of artificial berms.

9.4. Multi criteria analysis: integrated solutions

To compare the possible proposed solutions a multicriteria analysis is conducted. The criteria and weight of the analysis is comparable to that used in paragraph 9.1. Some adjustments have been made to emphasise the safety criteria, while removing less relevant criteria and adding new ones of interest. This section will elaborate on these adjustments and the scoring of the alternatives.

The most critical criterion for evaluating the design alternatives is wave overtopping, this is given a weighting of 35% and will have the largest influence on the proposed solution. In order to categorise the alternatives the two boundary solutions are used to set up a scale, the minimum overtopping reduction that can be achieved applying only curved walls is 53.4% while the maximum is obtained applying breakwaters everywhere resulting in a reduction of 94.3%. The scores, starting at 1 for the lowest reduction, increase by 1 for an increment in reduction of 10%. As expected the critical alternative has the most positive effect as it tackles the key sections and the value option scores lowest given the lack of measures applied.

The direct costs are approximated by giving an initial value to the basic option of only applying a wall with recurve, while the berms and breakwaters will be factors two and five times as expensive. The lowest achievable value for relative cost is 25, applying only curved wall, up to 150 for the breakwater boundary solution. This results in a scale from 25 to 150, subdivided into five steps of 25 points each. The direct costs are lowest, and therefore score best, for the value alternative as the lowest possible number of measures is applied. Costs of the alternating alternative are relatively high due to the large amount of material and work involved. The critical section received the same score because despite using fewer breakwaters the depth at which these are to be applied would significantly raise costs. The combination option is somewhere in between as it more carefully considers depth profiles and makes use of the existing berm. The number of breakwaters and berms is also expected to influence the amount of required maintenance. While the alternatives are designed to be maintenance free the possibility of some maintenance cannot be excluded. Therefore, the more measures are applied the more unfavourable the costs score.

The added economic value category aims to give an indication of the possible investment opportunities that may arise from improving the safety and appearance of the Malecón seawall. The alternatives providing the highest level of safety are likely to invite more investment as the area directly behind the seawall becomes a more secure investment with less likelihood of flooding. For this reason the critical and combination alternative score best, with the alternating option slightly below due to the loss of the characteristic view which may affect businesses near the seawall.

The social attractiveness is used to gauge the impact on how appealing the Malecón remains for locals and tourists. The alternating alternative scores lowest due to the large number of measures applied which hinder the view, similarly the large number of berms and breakwaters applied in the critical alternative reduce the appeal of the area. The value alternative scores best due to the implementation of a minimal number of measures while the combination alternative scores in between these proposals. The characteristic view pertains mainly to features of the Malecón such as the old pools in the berm and the location of the measures, placing more breakwaters around the traditional Malecón is seen is detracting more from the view than in other sections. The critical and alternating alternatives score the worst due to the amount and placement of measures. The value alternative, while modifying relatively little, does the least to protect the area behind the seawall and therefore it scores less than the combination alternative. The creation of employment opportunities is also seen as a social aspect of the proposed solution. The critical and alternating alternatives score best due to the large amount of construction needed, followed by the combination and then value alternatives. The effect on the environment is quantified mainly by how easily water can flow around the shore and to which extent marine life will be affected by the construction. The critical, alternating and combination alternatives make significant use of breakwaters and therefore significantly impact the coastal environment.

The on-site limitations relate primarily to the challenges related to constructing a particular alternative, the critical alternative scores lowest here as many breakwaters are set to be placed and several of these in areas with steep slopes which increase e probability of errors. For similar reasons the alternating alternative scores poorly while the combination alternative more carefully considers the issues of placement and challenges in construction. In terms of availability of materials each alternative scores similarly as the materials used are largely identical, the value alternative scores slightly better as the quantity of material used will be lower. Construction time largely depends on the number of measures applied, particularly breakwaters therefore alternating, critical, combination, and value alternatives scoring worst to best respectively. Figure 44 illustrates the scoring per alternative and the results from the MCA.

						Selected Alternatives			
					Weight	Critical	Alternating	Value	Combination
Criteria	Safety	35%	Wave Overtopping	100%	35%	4,2	3,91	2,1	3,98
			Structural Integrity	0%	0%				
	Cost	25%	Direct cost	60%	15%	2	2	5	3
			Maintenance costs	20%	5%	2	1	4	3
			Economic added value	20%	5%	4	3	1	4
	Society and environmental	25%	Social attractiveness	40%	10%	2	1	4	3
			Characteristic view	30%	8%	2	2	3	4
			Employment	20%	5%	4	4	2	3
			Effect on environment	10%	3%	2	2	4	2
	Implementation	10%	On site limitations	50%	5%	1	2	4	3
			Availability materials and equipment in Cuba	50%	5%	3	3	2	3
	Construction	5%	Construction Time	100%	5%	2	1	4	3
	Total	100%			100%	2,97	2,6685	3,06	3,443

Figure 44: Results of MCA final design solution

The most favourable solution is the combination alternative with a score of 3.44 followed by the value alternative with a score of 3.06. The result is to be expected since it aims to take in account most critical factors while realising a high reduction in wave overtopping. The combination alternative consists of a wall with recurve, berms, and offshore breakwaters placed based on consideration of the characteristics of the area. The following chapter will work out the proposed solution in a section-by-section detailed design which will include the specific characteristics of these structural elements.

10.Detailed design

This chapter contains the final design of the integrated solution resulting from the MCA in paragraph 8.8. It will include a section-by-section design based upon characteristics, wave height and applicability of measures. The proposal includes four breakwaters along the coast with a combined length of approximately 2.5 km as well as 2.5 km of newly constructed berms. A structural analysis and design verification of the wall is performed to propose a viable solution for the seawall. The berm and breakwater are dimensioned and tested in previous studies but require final computations for local conditions. Where after an overview of the solution is given including cross-sections along the Malecón. Finally, a cost estimation for the measures of the combined solution and construction methods is given and final analysis of the breakwaters in SWAN is performed.

10.1. Detailed design curved sea wall

The first step in the final design of the proposed solution is the design of the curved seawall. In order to perform a structural analysis of the seawall that was designed by Professor Cordova the results from the ANSYS model were used to set up a detailed design for the reinforcement. The forces to be taken by these bars, and the necessary number and diameter, are described in this chapter. Eurocode 2 for concrete structures (NEN-EN 1992-1-1) will form the basis for the calculations as it gives accurate and reliable information for the numerous aspects which must be taken into account for the proposed solution.

10.1.1. Cover

The previous investigation by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) and in accordance with Professor Córdova a cover of 100 mm was chosen. Before designing the reinforcement, the cover was checked according to the Eurocode (NEN, 2005) to verify the validity of this assumption. The exposure class is XS3 due to the risk of corrosion from chlorides and the location being a tidal, splash and spray zone. While the design life is set to 50 years it is likely that the structure will have to perform longer for this reason it was chosen to use the conservative estimate of a 100-year design life, raising the structural class by 2. The assumption of slab geometry lowers the structural class by 1, resulting in a structural class of S5. The minimum required cover is 70mm which the chosen of 100mm amply satisfies as well as adding a safety margin for the unknown negative influence of seawater.

10.1.2. Principal reinforcement

The principal (longitudinal) reinforcement is designed to take the tensile forces in the structure. These are concentrated where the pressures on the wall are highest and run vertically along the seaward face of the structure. In order to determine the amount of reinforcement required the tensile force must be determined, this is done by summing together all the stresses over the area multiplied by the areas over which they act as shown in the schematic drawing in Figure 45.

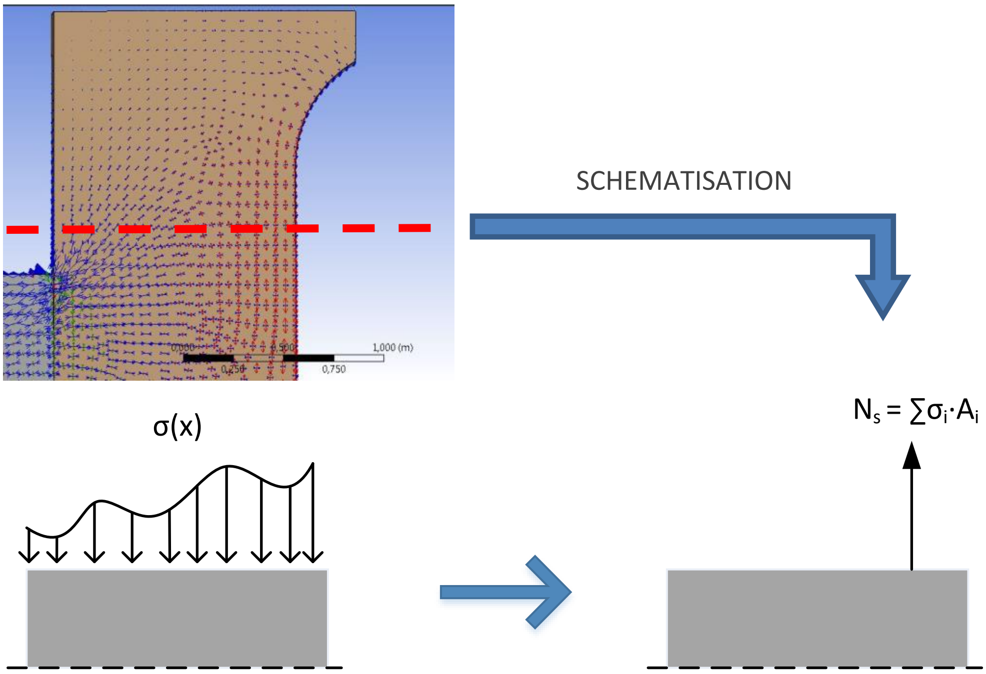

Figure 45: Schematisation to determine tensile force

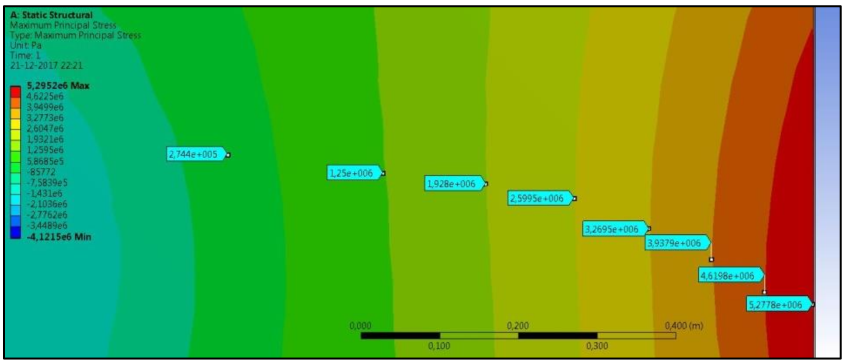

Figure 46: Tensile stresses in the ANSYS model used for determining forces

Table 19: Stresses and resultant forces in the structure

Average stress over area [MPa]	Area of the segment [m²]	Tensile force [N]
(5.3 + 4.63) / 2	0.06	307800
(4.63 + 4) / 2	0.07	302050
(4 + 3.3) / 2	0.08	280000
(3.3 + 2.6) / 2	0.09	265500
(2.6 + 1.93) / 2	0.11	249150
(1.93 + 1.26) / 2	0.125	199375
(1.26 + 0.6) / 2	0.14	130200
TOTAL (N_s)		1,734,075

The required area of reinforcement to be taken by the steel is:

$A_{s,req} = N_s / f_{yd} = 1{,}734{,}075 / 435 = 3986{,}4\ mm^2$.

It is important to note that the calculations will be performed per meter width of the structure. In order to satisfy this requirement 5 bars of 32mm diameter will be applied (i.e. 32mm diameter bars spaced 140mm along the width). This results in a steel reinforcement area of $A_s = 4020\ mm^2$.

10.1.3. Minimum reinforcement check

According to Chapter 7.3.2 of NEN 1992-1-1 the minimum reinforcement area is given by:

$$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct} \tag{7.1}$$

where:

- $A_{s,min}$ is the minimum area of reinforcing steel within the tensile zone
- A_{ct} is the area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack
- σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack width limits according to the maximum bar size or spacing (see 7.3.3 (2))
- $f_{ct,eff}$ is the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur:

  $f_{ct,eff} = f_{ctm}$ or lower, ($f_{ctm}(t)$), if cracking is expected earlier than 28 days
- k is the coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces

  = 1,0 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm

  = 0,65 for webs with $h \geq 800$ mm or flanges with widths greater than 800 mm

  intermediate values may be interpolated
- k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:

  For pure tension $k_c = 1{,}0$

  For bending or bending combined with axial forces:

  - For rectangular sections and webs of box sections and T-sections:

$$k_c = 0{,}4 \cdot \left[1 - \frac{\sigma_c}{k_1(h/h^*)f_{ct,eff}}\right] \leq 1 \tag{7.2}$$

$\sigma_s = N_s / A_s = 1{,}734{,}075 / 4020 = 431\ N/mm^2$

A_{ct} is determined from the ANSYS model as the area in which in the principal stress is tensile: 360*1000 = 360000 mm^2

$f_{ct,eff}$ = f_{ct} = 3.2 N/mm^2 because cracking is not expected earlier than 28 days

k = 0.65 because h ≥ 800 mm

σ_c is determined from the average stress along the cross-section and is equal to 3.7 MPa (compressive)

k_1 = 1.5 for compressive stresses

h = 1200 mm

h* = 1000 mm because h ≥ 1,0 m

k_c = 0,143 (≤ 1,0)

This results in a minimum reinforcement area:

$A_{s,min}$ = (0,143*0,65*3,2*360*1000)/431 = 249 mm^2, the provided area is larger than this value therefore it suffices.

10.1.4. Secondary reinforcement

The secondary reinforcement runs along the width of the structure, perpendicular to the longitudinal reinforcement. In Chapter 9.3.1.1 (2) of Eurocode 2 it is stated that the secondary reinforcement in a slab should be at least 20% of the larger area of A_s and $A_{s,min}$. A_s is largest in this case meaning the minimum secondary reinforcement to be provided is: 0.2*4020 = 804 mm^2.

In order to satisfy this requirement 4 bars of diameter 16mm are provided resulting in an area of $A_{s,secondary}$ = 804.2 mm^2. The placement of these bars will be discussed in further detail in chapter 10.1.5.

10.1.5. Shear reinforcement

The first step in calculating the shear capacity is determining the shear capacity of the concrete and principal reinforcement. According to Chapter 6.2.2 the shear capacity without shear reinforcement is given by:

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \tag{6.2.a}$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \tag{6.2.b}$$

where:

f_{ck} is in MPa

$k \quad = 1+\sqrt{\dfrac{200}{d}} \leq 2{,}0$ with d in mm

$\rho_l \quad = \dfrac{A_{sl}}{b_w d} \leq 0{,}02$

A_{sl} is the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]

$\sigma_{cp} \quad = N_{Ed}/A_c < 0{,}2\,f_{cd}$ [MPa]

N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_E may be ignored.

A_C is the area of concrete cross section [mm^2]

$V_{Rd,c}$ is [N]

Note: The values of $C_{Rd,c}$, v_{min} and k_1 for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0{,}18/\gamma_c$, that for v_{min} is given by Expression (6.3N) and that for k_1 is 0,15.

$$v_{min} = 0{,}035\,k^{3/2} \cdot f_{ck}^{1/2} \tag{6.3N}$$

$C_{Rd,C}$ = 0,18 /γ_c = 0.18/1.5 = 0.12

d = 1200 – 100 – 32/2 = 1084 mm

k = 1 + √(200/d) = 1,43

f_{ck} = 35 MPa

k_1 = 0.15 (recommended)

b_w = 1000 mm (width per meter width)

σ_c 1,5 for compressive stresses

h = 1200 mm

h* = 1000 mm because h ≥ 1,0 m

This results in a shear capacity of $V_{Rd,c}$ = 2,572,812 N = 2572 kN

The minimum value of which must be:

$v_{min} = 0.0035*k^{3/2}*f_{ck} = 0.35$, $v_{Rd,c}$ is larger than this value and can therefore be used for the resistance.

The design shear force acting on the structure is; $V_{Ed} = 3.018 * 10^6$ N = 3018 kN, and is derived using the same method as the normal tensile force. Figure 47 shows the distribution of shear stresses in the cross section.

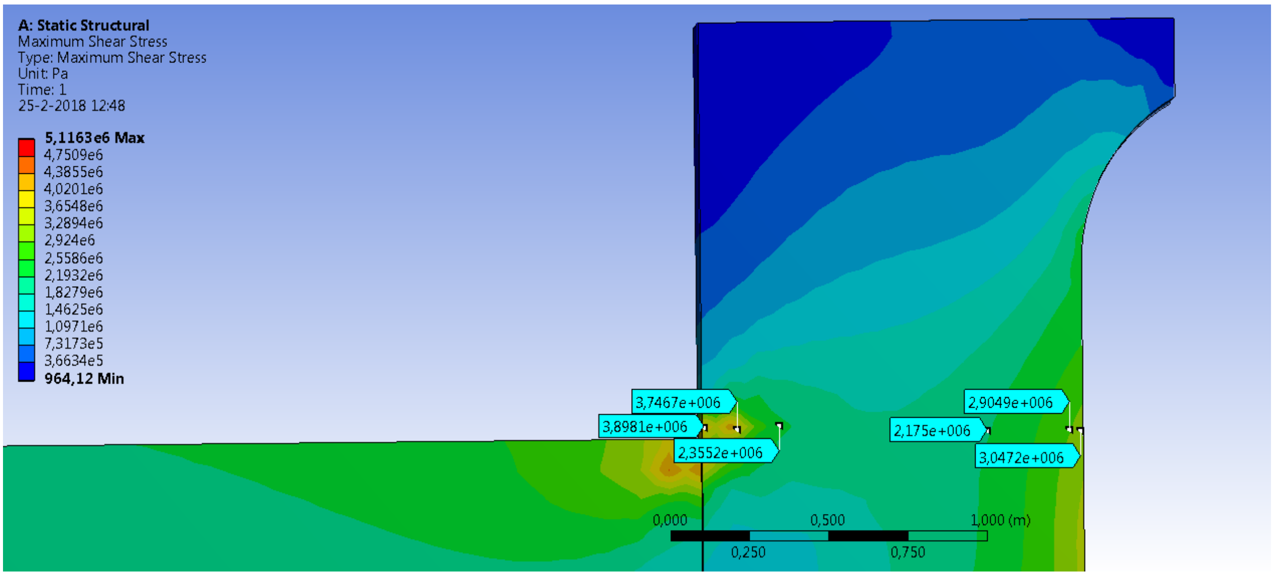

Figure 47 Shear stress distribution in the ANSYS model

$V_{Ed} > V_{Rd,c}$ therefore additional shear reinforcement is need. The shear force that must still be compensated is: 3,018,000 – 2,572,812 = 445 * 10^3 N = 445 kN.

According to Chapter 6.2.3 of Eurocode 2 the shear capacity of members with shear reinforcement is given by:

$$V_{Rd,s} = \frac{A_{sw}}{s} z\, f_{ywd} \cot\theta \qquad (6.8)$$

Note: If Expression (6.10) is used the value of f_{ywd} should be reduced to 0,8 f_{ywk} in Expression (6.8)

and

$$V_{Rd,max} = \alpha_{cw}\, b_w\, z\, \nu_1\, f_{cd}/(\cot\theta + \tan\theta) \qquad (6.9)$$

where:

- A_{sw} is the cross-sectional area of the shear reinforcement
- s is the spacing of the stirrups
- f_{ywd} is the design yield strength of the shear reinforcement
- ν_1 is a strength reduction factor for concrete cracked in shear
- α_{cw} is a coefficient taking account of the state of the stress in the compression chord

Note 1: The value of ν_1 and α_{cw} for use in a Country may be found in its National Annex. The recommended value of ν_1 is ν (see Expression (6.6N)).

Note 2: If the design stress of the shear reinforcement is below 80% of the characteristic yield stress f_{yk}, ν_1 may be taken as:

$\nu_1 = 0,6$ for $f_{ck} \leq 60$ MPa (6.10.aN)

$\nu_1 = 0,9 - f_{ck}/200 > 0,5$ for $f_{ck} \geq 60$ MPa (6.10.bN)

Note 3: The recommended value of α_{cw} is as follows:

1 for non-prestressed structures

$(1 + \sigma_{cp}/f_{cd})$ for $0 < \sigma_{cp} \leq 0,25 f_{cd}$ (6.11.aN)

1,25 for $0,25 f_{cd} < \sigma_{cp} \leq 0,5 f_{cd}$ (6.11.bN)

$2,5 (1 - \sigma_{cp}/f_{cd})$ for $0,5 f_{cd} < \sigma_{cp} < 1,0 f_{cd}$ (6.11.cN)

where:

σ_{cp} is the mean compressive stress, measured positive, in the concrete due to the design axial force. This should be obtained by averaging it over the concrete section taking account of the reinforcement. The value of σ_{cp} need not be calculated at a distance less than 0.5d cot θ from the edge of the support.

Note 4: The maximum effective cross-sectional area of the shear reinforcement, $A_{sw,max}$, for $\cot\theta = 1$ is given by:

$$\frac{A_{sw,max} f_{ywd}}{b_w s} \leq \tfrac{1}{2}\, \alpha_{cw} \nu_1 f_{cd} \qquad (6.12)$$

A_{sw} = 4*π/4*16² = 804 mm², this is assuming that the secondary reinforcement will be placed as stirrups would be in a beam us providing tensile strength in both x- and y-directions

s = 700 mm, this is the vertical distance between the 'stirrups' as the wall is being modelled as a beam rotated by 90 degrees (or placed vertically rather than horizontally)

z = 0.9*d = 0.9*1084 = 976 mm

f_{ywd} = 435 MPa

cot(θ) = 1 (Eurocode recommends a value between 1 and 2.5) (θ = 45 degrees)

This results in an additional shear strength of: $V_{Rd,s}$ = 474,092 N = 474 kN. This must be lower than the maximum allowable resistance $V_{Rd,max}$ which is calculated using the parameters:

α_{cw} = 1 for non-prestressed structures

b_w = 1000 mm

v_1 = 0.6 because $f_{ck} \leq 60$ MPa

f_{cd} = 23.3 MPa

This yields: $V_{Rd,max}$ = 6,832,000 N = 6832 kN > $V_{Rd,s}$ which passes the check. The maximum effective cross-sectional area of the shear reinforcement $A_{sw,max}$ must also not be exceeded. Using equation

(6.12) above and the presented parameters $A_{sw,max}$ = 11,248 mm^2 which is not exceeded by the provided area of 804 mm^2.

The total shear resistance of the concrete is: $V_{Rd} = V_{Rd,c} + V_{Rd,s} = 2572 + 474 = 3046$ kN > $V_{Ed} = 3018$ kN.

10.1.6. Detail design

The diagram below indicates the design for the reinforcement mesh. On the left an overview is show with dimensions indicated while on the right key aspects of the design are highlighted and explained.

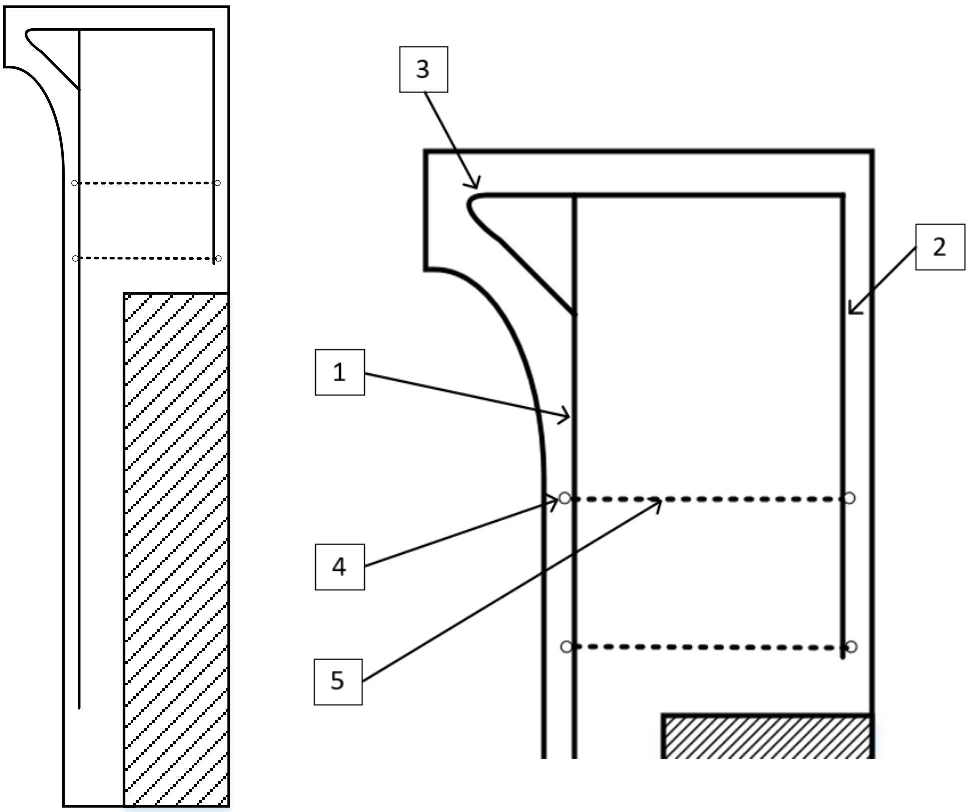

Figure 48: Overview of reinforcement mesh and key details

1. Principal reinforcement: bars placed vertically to take the tensile stresses in the structure. The bars have a diameter of 32mm and run from the top of the wall, minus the cover of 100mm, to 1050mm from the bottom of the wall where the tensile stresses approach 0.
2. This bar forms the other side of the principal reinforcement and matches the bar at the front
3. The bar at the back of the structure highlighted in point 2 will be bent backwards at the front of structure to take the tensile stresses in the recurve.
4. Secondary reinforcement: provided by 4 bars of 16mm with a distance of 700mm. These will be placed like stirrups in a beam element and form a complete loop to provide stability to the reinforcement mesh.
5. Shear reinforcement: the dotted lines indicate where the secondary reinforcement will loop back. These elements spanning from the front to the back of the wall also serve as shear reinforcement. Each stirrup must form a complete loop in every meter width in order to provide the required reinforcement area.

1.1.1. Crack width control

According to Chapter 7.3.4 of Eurocode 2 the crack width, w_k, is given by:

(1) The crack width, w_k, may be calculated from Expression (7.8):

$$w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm}) \tag{7.8}$$

where

- $s_{r,max}$ is the maximum crack spacing
- ε_{sm} is the mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered
- ε_{cm} is the mean strain in the concrete between cracks

(2) $\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from the expression:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}} \left(1 + \alpha_e \rho_{p,eff}\right)}{E_s} \geq 0{,}6 \frac{\sigma_s}{E_s} \tag{7.9}$$

where:

- σ_s is the stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\sigma_p$ the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level.
- α_e is the ratio E_s/E_{cm}
- $\rho_{p,eff}$ $(A_s + \xi_1^2 A_p')/A_{c,eff}$ (7.10)
- A_p' and $A_{c,eff}$ are as defined in 7.3.2 (3)
- ξ_1 according to Expression (7.5)
- k_t is a factor dependent on the duration of the load

k_t = 0,6 for short term loading
k_t = 0,4 for long term loading

The maximum crack spacing should be calculated according to:

$$s_{r,max} = k_3 c + k_1 k_2 k_4 \phi / \rho_{p,eff} \tag{7.11}$$

where:

ϕ is the bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, ϕ_{eq}, should be used. For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2, the following expression should be used

$$\phi_{eq} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2} \tag{7.12}$$

c is the cover to the longitudinal reinforcement

k_1 is a coefficient which takes account of the bond properties of the bonded reinforcement:
= 0,8 for high bond bars
= 1,6 for bars with an effectively plain surface (e.g. prestressing tendons)

k_2 is a coefficient which takes account of the distribution of strain:
= 0,5 for bending
= 1,0 for pure tension
For cases of eccentric tension or for local areas, intermediate values of k_2 should be used which may be calculated from the relation:

$$k_2 = (\varepsilon_1 + \varepsilon_2)/2\varepsilon_1 \tag{7.13}$$

Where ε_1 is the greater and ε_2 is the lesser tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section

This is because the bonded reinforcement is spaced 147 mm < 5(c + Ø/2) = 5(100+16) = 580 mm.

σ_s = $\sigma_{c,tensile}$*$A_{c,tensile}$ / A_s = 1.5*360*1000 / 4020 = 131 N/mm^2, this is the average tensile stress in the concrete (in SLS) multiplied by the area of concrete in tension and divided by the area of steel reinforcement

k_t = 0.6 for short term load

z = 0.9*d = 0.9*1084 = 976 mm

$f_{ct,eff}$ = f_{ctm} = 3.2 MPa

$h_{c,eff}$ = min{2.5(h-d);(1/3)*(h-x);h/2)
= min{2.5(1200-1084);(1/3)*(1200-840);1200/2}
= min{290;120;600)
= 120 mm

$\rho_{p,eff}$ = A_s / $A_{c,eff}$ = 4020 / (120*1000) = 0.0335

α_e = E_s/E_{cm} = 210 / 35 = 6

This gives $\varepsilon_{sm} - \varepsilon_{cm}$ = 2.9*10^{-4}, however this must have a minimum value of 0.6*(σ_s/E_s) = 3.7*10^{-4} therefore $\varepsilon_{sm} - \varepsilon_{cm}$ = 3.7*10^{-4}.

For the calculation of $s_{r,max}$ the following parameters are used:

k_1 = 0.8, high bond bars are used

k_2 = $(\varepsilon_1 + \varepsilon_2) / 2\ \varepsilon_1 = (0.00016 - 2.3*10^{-6}) / (2*0.00016) = 0.493$

k_3 = 0.54, recommended by Eurocode 2

k_4 = $1.25\ (0.6 + 0.0014 / \varepsilon_{cu2})$
$= 1.25\ (0.6 + 0.0014 / 3.5*10^{-3})$
$= 1.25$

Ø = 32 mm

$\rho_{p,eff}$ = $A_s / A_{c,eff} = 4020 / (120*1000) = 0.0335$

This results in a maximum crack spacing $s_{r,max}$ = 67 mm.

The resulting crack width is w_k = 0.025 mm.

No direct requirements were established for the maximum allowable crack width however preventing corrosion from sea water is a clear necessity. Sub-heading (5) of Chapter 7.3.1 of Eurocode 2 states that in the absence of specific requirements a w_{max} = 0.2mm is generally satisfactory. This requirement is met without any problems.

Other references (Concrete Design Guide. No. 1: Guidance on the design of liquid-retaining structures., 2015)suggest that the crack width for water retaining structures should be limited according to the ratio of the hydraulic head (h_d) to the thickness of the wall (h).

Taking the hydraulic head as the height of the incoming wave gives a value for h_d = 9.52m and the wall thickness is h = 1.2m. This results in a ratio h_d/h = 7.93.

Table 6: Limiting values of $w_{k,1}$							
h_d/h	≤5	10	15	20	25	30	≥35
$w_{k,1}$ (mm)	0.200	0.175	0.150	0.125	0.100	0.075	0.050

Figure 49: Limiting values of w_k according to h_d/h

Taking a conservative value for the ratio of h_d/h as 10 it can be seen that allowable crack width is 0.175 mm which is still well above the calculated crack width.

The main reason for these small cracks is that despite the large loads the structure is subjected to the dimensions are very large meaning the self-weight limits the tensile stresses in the cross-section being considered.

1.1.2. Dowels and epoxy grouting

(Sing, Azraai, Yahaya, & Noor, 2015) state that the bond strength for bonding dissimilar materials, such as steel and concrete in this case, is between 7-35 MPa. Assuming the most unfavourable situation for the connection between the new wall and the dowel in which the tensile force to the foundation must be taken exclusively by the grout around the dowel the force to be taken is $1.35*10^6$ N for the top half of the dowel. The top half of the dowel is governing for the shear stress for this reason it has been chosen to analyse only this part. Taking the lower bound for the bond strength of the epoxy grout the required surface area for bonding is $1.35*10^6/7 = 192857$ mm^2.

The surface area of a half dowel is 28274 mm^2 meaning 7 dowels must be applied per meter width of the structure. The spacing is of 99mm is sufficient that it should not significantly hinder the integrity of the structure, care must be take however to accurately offset the position of the dowels and the reinforcement to avoid damaging either during construction.

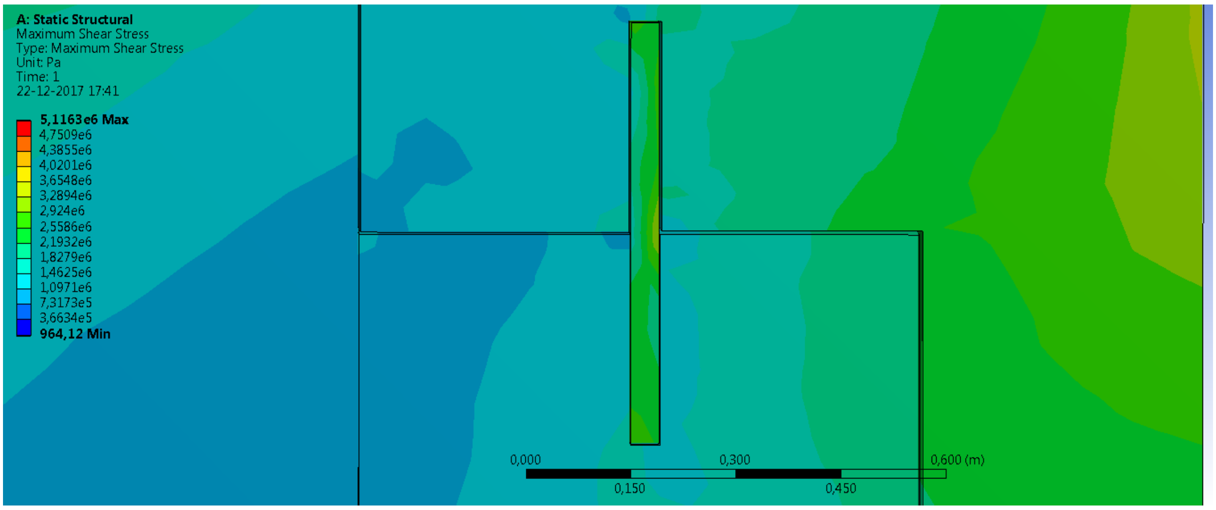

Figure 50: Shear stresses used to determine the governing force to be taken by the epoxy grout

1.1.9 Connection between the old and new wall

Apart from the dowels there is no bond set to be created between the old and new wall. The outer face of the old wall will be sand-blasted and cleaned before the new wall is mounted on top of it.

10.2. Unit determination berm and breakwater

A previous physical model test provides information regarding the optimal dimensions for an emerged breakwater. The test conducted involved emerged breakwaters close to the coast and were performed at the Laboratorio di Ingegneria Costiera of the University of Naples, Federico II. The slope is constructed with armour layered concrete cubes weighing 30 tons. The main result of this test with a 1/50 year return period storm and Hurricane Wilma conditions was the failure of the slope. Due to (Córdova L. F., 2017):

- The low porosity of the armour layer;
- The absence of interlocking among the cubes;
- The location of the structure, which exposes it to the strikes of breakers

10.2.1. Choice of armour elements

As a result of the failure of the slopes during the experiments an alternative to concrete blocks is proposed in the form of a single layer concrete unit-system. This has a higher porosity and creates a better interlocking between the elements. The units to be applied for the berms and breakwaters are ACCROPODE II and ECOPODE units produced by Concrete Layer Innovations (CLI). ECOPODE units are more suited for berms due to their more natural appearance while ACCROPODE units are best applied for the offshore breakwaters subjected to higher wave impact which require larger unit sizes. Due to the limited size of 10 m^3 for ECOPODE, CLI provides design tables used to create an initial design for possible applications using its units (Concrete Layer Innovations, 2012). These are used extensively in this chapter and can be found in Appendix W.

10.2.2. Design input parameters

In order to determine the required size of the units the slope of the seabed where each measure will be applied must be determined. The governing slope is that at the toe of the seaward facing side of the structure which is most critical for issues of stability. The seabed slopes for the berms and first breakwater in Section 2 are taken from (Muilwijk, Versmissen, Meijer, Groenendaal, & Veenstra, 2003) and have a value of 8.3%. The slope for the second breakwater which starts in Section 2-7 is of 11.2% and is taken from the same report. It should be noted that these slopes are an average taken over a distance from the coast varying from 50 to 80m. The slopes chosen however represent extreme values higher than what is determined directly from bathymetry data and it is sure to lead to a conservative design. The slope for the berms applied in Section 3 is estimated from the bathymetry data and is estimated at 13%. The seabed slopes for the berms are retrieved from the cross-section in previous work (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015). For the breakwaters in Sections 4 and 5 the bathymetry is again used to estimate the seabed slope and is approximately 9%.

The other input parameter to determine the required size of armour units is the significant wave height. For the berms the significant wave height at the wall as determined by SWAN models will be used while for the breakwaters the significant wave height at the distance at which the depth is 5.2m is used. This is done in order to better integrate the existing breakwater designs by Professor Córdova, which are standardised to a water depth of 5.2m. The highest wave height that occurs over the length of the defence measure is taken to provide a more conservative but also more realistic design given that it is unlikely that a particular wave height will only occur in one specific part of a structure.

10.2.3. Determining necessary unit sizes

The design graph shown in Appendix W uses the input parameters described previously and gives a minimum volume of the armour units to be applied. The graphs provided by CLI were recreated and extrapolated in order to determine the unit sizes for seabed slopes which are not given explicitly (Figure 51), while this method carries a significant degree of uncertainty the unit sizes are all rounded up to larger volumes which adequately covers this aspect.

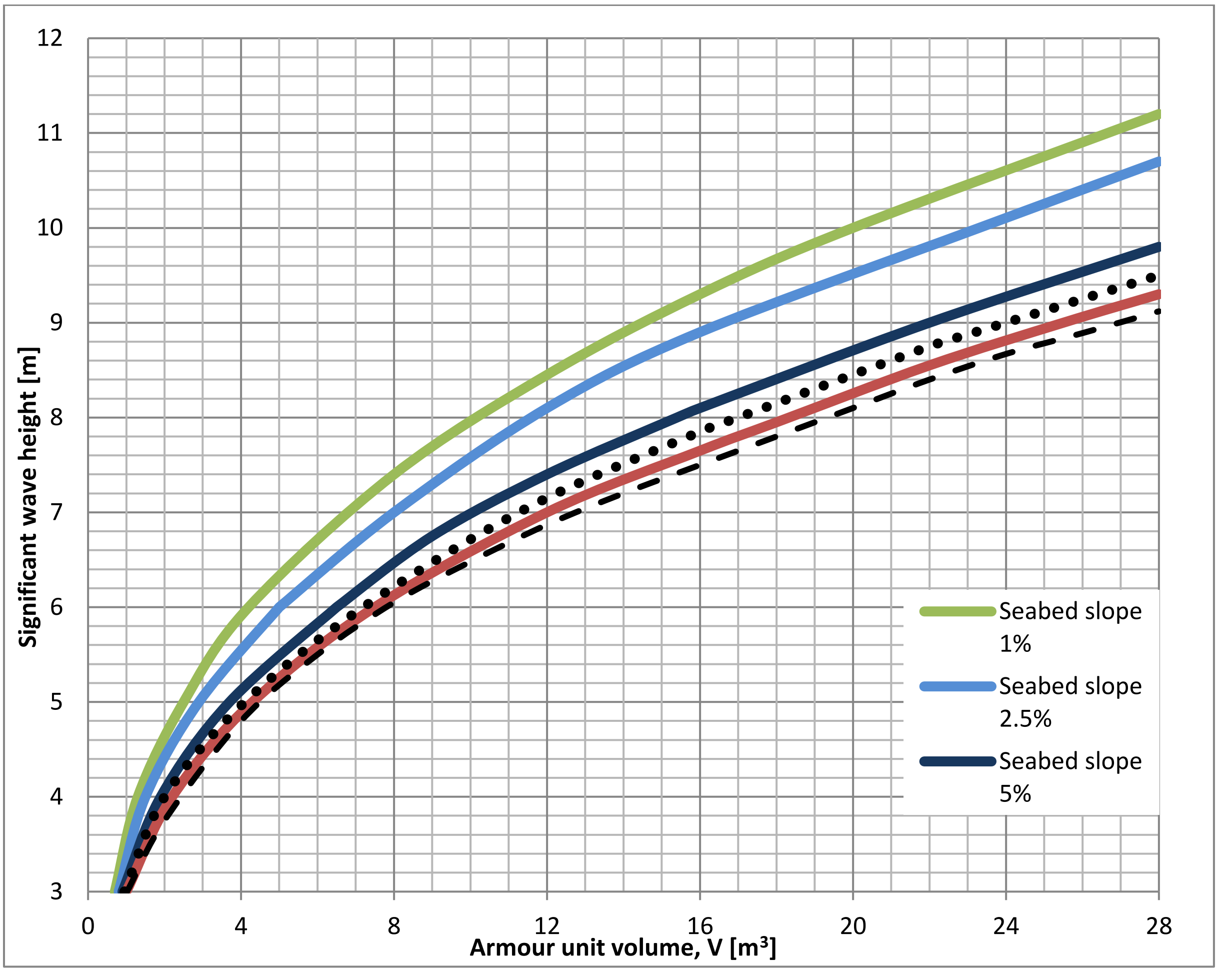

Figure 51: Design graphs by CLI with extrapolated lines for required seabed slopes

(van der Meer, 1988) describes the following stability condition for the armour units used in the structures: $N_s = {}^{H_s}/_{\Delta D_n}$

Where Hs is the significant wave height and D_n is the equivalent cube size (given by the cube root of the volume, V1/3). For *Ns < 3.7* no damage will occur to the elements in the structure while (Córdova López L. F., 2017) in accordance with Deltares, the company tasked with performing the proposed study, recommend a 'design value' of 2.5. For each structure the minimum armour unit volume is determined from the graph provided by CLI using the seabed slopes and significant wave heights. This is rounded up to the next available unit size. The unit size is increased until the design value condition of 2.5 is met for every structure. An overview of these calculations and resulting unit sizes is given in Appendix W.

From these considerations it is clear that the 'no damage' criteria for the armour units is far more favourable and can lead to significant reductions in material costs and construction time. Meanwhile, the design value option is preferred for its low maintenance qualities. The feasibility of this proposal with respect to its more cost-effective counterpart must be discussed further should it reach executions phase. Particularly the berm in Section 2-1 and 2-2 which is longer than the others provides an opportunity for reduction in costs.

10.2.4. Breakwater modelling in SWAN

To verify and test the proposed design regarding the breakwaters, a new SWAN model is made incorporating the breakwaters. For this model the 0 degrees wind direction and loading combination 1 are used. The breakwater is modelled using the DANGREMOND function in SWAN, using a crest height of +3.28MSL, a crest width of 12 meters and an angle of 56 degrees. After running the model, the significant wave height and wave setup in each subsection are determined and used to calculate the wave overtopping in each section. See Figure 52 for the results of these calculations. The graphical results can be found in Appendix X.

			Input from SWAN		New prediction	Old prediction	
Section	Wind Direction	Subsection #	Hsig	Zeta m	q Overtop m3/s/m	Q Predicted m3/s/m	Difference %
2	0	1	2.885	0.3505	0.130	0.116	-11%
		2	3.322	0.3901	0.226	0.210	-7%
		3	2.735	0.3270	0.211	0.094	-55%
		4	2.678	0.3194	0.198	0.083	-58%
		5	3.395	0.3303	0.226	0.279	23%
		6	3.908	0.3180	0.367	0.169	-54%
		7	2.492	0.2993	0.186	0.072	-61%
		8	2.513	0.3290	0.173	0.041	-77%
3	0	1	3.464	0.3636	0.438	0.188	-57%
		2	2.436	0.5860	0.231	0.154	-33%
		3	2.741	0.4048	0.236	0.147	-38%
		4	2.227	0.4153	0.148	0.180	22%
		5	4.916	0.1488	0.678	0.590	-13%
		6	4.078	0.1357	0.343	0.407	18%
		7	2.301	0.3350	0.057	0.079	39%
		8	2.839	0.2823	0.113	0.059	-48%
4	0	1	3.129	0.2803	0.159	0.116	-27%
		2	3.132	0.3108	0.166	0.191	15%
		3	2.875	0.2944	0.120	0.135	13%
		4	2.262	0.2568	0.115	0.043	-63%
		5	2.237	0.2757	0.115	0.070	-39%
5	0	1	3.305	0.1117	0.242	0.230	-5%
		2	3.549	0.1589	0.304	0.264	-13%
		3	2.053	0.1817	0.088	0.029	-67%
		4	2.777	0.1935	0.180	0.035	-80%

Figure 52: Results after implementing breakwaters

From Figure 52 it becomes clear that the breakwaters certainly have influence on the wave climate in front of the wall. The overtopping is reduced in a great amount; however, some discrepancies are found between the new and old predictions. This is partly due to the fact that zero wave setup was assumed in the old prediction, whereas the new SWAN model returns wave setup at these locations. Secondly, the breakwaters in section 2 and 5 are located to dissipate the most energy from waves coming from the North-West. In the new model only the North direction is modelled, which is probably the reason for the high waves behind the breakwaters in these sections. For this reasons it is recommended to further model breakwaters and their influence on the wave climate.

10.3. Overview solution Malecón coastal defence system

This paragraph aims to give an overview of the proposed solution for the Malecón coastal defence system, comparison with the solution described after the MCA in paragraph 8.8 some adjustments have been made. Due to an additional cost analysis and in accordance with Professor Córdova the breakwater length is reduced to a total of 2 kilometers by shortening the two breakwaters in section 2. Both have a length of 500 meters and in section 2-5 and 3-1 berms are applied instead. Figure 53 illustrates the Malecón area for sections 2 to 5 with the applied measures. Detailed illustrations from each section are provided in Appendix Z.

Figure 53: Overview solution coastal defence system Malecón

This adjustment results in an average overtopping reduction of 77.3 % compared to the current situation which translates to an average resulting overtopping of 0.152 m^3/sec/m over the entire study area. The results for each section are given in Table 20 expressed in volumes of overtopping, over the entire Malecón the overtopping during governing storm conditions is 18.95 m^3/sec/m which is reduced to 3.82 m^3/sec/m. Appendix Y provides an overview of the wave overtopping calculation per sub-section with the resulting overtopping values after the applied measures.

Table 20: Existing overtopping compared to new overtopping per section

Section	Existing overtopping m^3/sec/m	After measures m^3/sec/m	Reduction in %
2	1,11	0,12	89,28
3	0,56	0,22	61,17
4	0,48	0,11	77,62
5	0,46	0,12	73,23

Detailed cross-sections of the final design are presented for sections with distinct characteristics. These cross-sections include the measures applied as they would appear in the practice, an example of which is shown in Figure 54.

Cross section for sections 4-4, 4-5, 5-3 and 5-4 both have a breakwater of approximately 500 meters in front of the Malecón Tradicional. The breakwaters are designed with concrete ACCROPODE 12 m^3 units with a crest width of 12m and a crest height of 3.28m above MSL. The seabed for the first breakwater has an average slope of 9.2 % and the second 9.0 %. The breakwater emerges and becomes visible at approximately 30 meters from the wall with normal sea conditions.

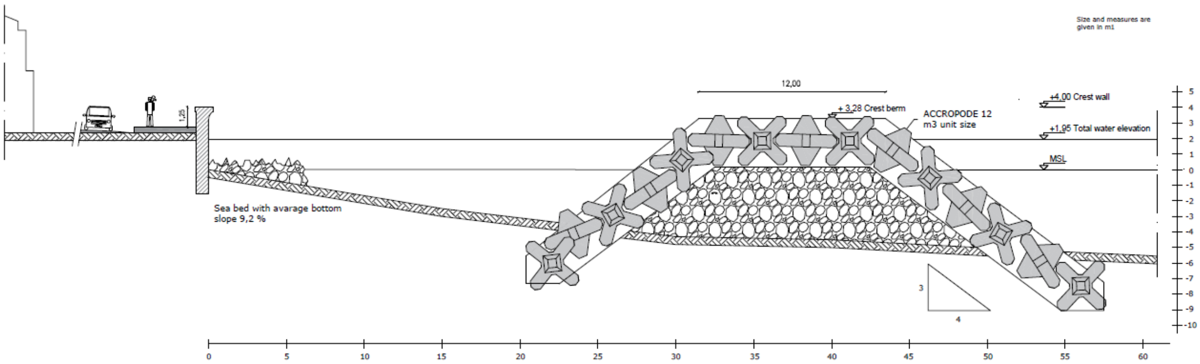

Figure 54 Detailed design for section 4.4, 4.5, 5.3 and 5.4

A complete overview of the cross sections is given in Appendix AA

10.4. Cost estimates

Information regarding construction cost estimates is retrieved from the Cuban Ministry of Construction (Centro de Información de la Construcción, 2005) from a document referred to as Precons which contains reference projects and estimates by the author. According to Precons the costs are classified into two categories: primary costs and secondary costs. The first is directly related to the construction and the secondary costs have an indirect relation. Table 21 and Table 22 show the composition of the primary and secondary costs. Prices are given in CUC ($), Cuban Convertible Pesos.

Table 21: Primary costs division

Subject	Includes	Determined
Direct costs of material (C1)	Construction material, which forms an integral part of the construction (concrete, concrete elements, bars of steel, cables, pipes, etc.). Supporting materials, which are used during the work (wood, molds, etc.). Semi-manufactured parts (the elements that arrive at the construction site in a partial state). Prefabricated materials (construction of concrete, construction of wood). Costs of the use of water during the fabrication of concrete.	According to Precons or determined from reference projects
Direct costs of work by hand (C2)	Design, Technical preparations (office, calculations, communication), Wages and Water (not used for concrete).	Precons or estimations
Direct costs of equipment (C3)	Fuel, lubricant, oil, electricity. Wages for the permanent operators of the material. Reparation and maintenance of the material. Interest of the use of capital, and Taxes.	Precons or estimations
Direct costs of means of support and small material (C4)		3% of C1+C2+C3
Total direct costs (C5)		C1+C2+C3+C4

Indirect costs (C6)	This includes for instance the design, preparation works and general overhead costs	11% of C5
Total costs (C7)		C5 + C6
Profit (C8)		20 % of C7
Total primary costs (C9)		C7 + C8

Table 22: Secondary costs division

Subject	Includes	Determined
Temporary facilities (P1)	Toilets, material warehouses, etc.	Precons
Transport (P2) Other additional costs (P3)	Transportation of materials by land or sea	Precons or estimation
Banking (P4)	Risk of price-changes during the project. Risks will not be quantified in this stage of the project, assumed to be included under unpredictable costs.	
Security (P5)	Protection of material and tools during and after work hours	1 % of C7
Unpredictable costs (P6)	Unpredictable costs have a high change of occurrence in this phase of a project	10 % of C9
Total secondary costs (P7)		P1 t/m P7

With these figures the aim is to give a cost estimate for a section of 100 meters for each alternative with the goal to create an overall cost estimation for the final design.

10.4.1. Curved wall cost estimation

The cost estimate is partly derived from the final design of the seawall in 2015 (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015). A cost estimate was made of \$450 for 1 m^3 of seawall, this includes concrete, reinforcement, dowels and framework. This was a design for a straight seawall, the recurve adds additional material which is added in the calculation. Constructing the recurve requires more effort in terms of framework and reinforcement and will therefore cost an additional \$30 per m^3. One meter of curved sea wall consists of 4,305 m^3 C34/45 concrete. A section of 100 meter curved sea wall has a total cost of approximately 0.78 million CUC. Detailed cost estimation is given in Appendix AB.

10.4.2. Berm and breakwater cost estimation

To determine the price of the ACCROPODE and ECOPODE used in the construction of the berms and breakwaters a reference project is used (Schepers, 1998). Prices are from 1997 in the former Dutch currency Gulden which is adjusted to Euros, current price level, and then to CUC. The price of the Accropode was 400 gulden/m^3 and core material (rocks from quarries) cost 15 gulden/1000 kg. For the Accropode this includes production of concrete, labour, the framework and placing. It is assumed that due to lower production and labour costs compared to the Dutch market a 50% reduction in prices for the Cuban market is realistic. For core material this includes the overall costs in the quarry meaning production costs include labour.

There are two types of berms included in the design which vary in price due to the quantities of material used. The first has a crest width of 5 meters and a height of 3.28 meters above MSL, the second has crest width of 20 meters and crest height of 2.28 meters above MSL. The quantities for both types of berms and the breakwater are given in Table 23, Table 24 and Table 25. The costs for

100 meters of berm type 1 are around 4.7 million CUC and of 16.7 million CUC for type 2, while for 100 meters of breakwater is approximately 66.3 million CUC. Detailed cost estimations for both structures are given in Appendix AB.

Table 23: Material quantities berm type 1

Berm 1 (5 meter crest)	Surface in m2	Width	Units	Volume in m3	Weight in tons
Armoured layer	25,9	100	789,95	2369,85	5687,64
Core	17,4	100		1740	4524
Excavation	5,5	100		550	

Table 24: Material quantities berm type 2

Berm 2 (20 meter crest)	Surface in m2	Width	Units	Volume in m3	Weight in tons
Armoured layer	59,2	100	1805,6	5416,8	13000,32
Core	62	100		16120	41912
Excavation	5,5	100		550	

Table 25: Quantities breakwater

Breakwater	Surface in m2	Width	Units	Volume in m3	Weight in tons
Armoured layer	148,7	100	1159,86	25516,92	61240,608
Core	59,5	100		15470	40222
Excavation	41,5	100		4150	

All the costs derived and assumptions used are calculated to current prices in CUC, a 2 % inflation rate is applied for every year. The calculations are presented in Table 26.

Table 26: Currency calculations

Part	Unit volume	Price in Gulden	Current €	Current $
Accropode	m3	400	1307,63	1169,73
Core material production	1000 kg	15	49,04	43,64
Placing core from land	1000 kg	7	22,88	20,37
Placing core from Sea	1000 kg	9	29,42	26,19
Transport land <300 kg	1000 kg/km	0,25	0,82	0,73
Transport sea <300 kg	1000 kg/km			0,36
Transport land >300 kg	1000 kg/km	0,4	1,31	1,16
Transport sea >300 kg	1000 kg/km			0,58

10.4.3. Combined cost estimation for the Malecón coastal defence system

With the costs calculated for each measure a combined cost estimate can be given for the final design solution. This includes all four types of measures applied in the different section, the total project costs will be approximately 1.14 billion CUC, taking into account that this is an estimate with prices derived only in part using Cuban standards. The cost overview is illustrated in Figure 55.

Total costs Malecón coastal defence system				
	Quantity	Prite Per Unit	Subtotal	Total
Total project costs				
Total design costs(hrs)	9600	40	$ 384.000,00	
Curved sea wall (100 mtr)	58	$ 785.435,90	$ 45.555.282,16	
Berm type 1 (100 mtr)	25	$ 4.686.006,37	$ 117.150.159,33	
Berm type 2 (100 mtr)	5	$ 16.758.411,65	$ 83.792.058,25	
Breakwater (100 mtr)	18	$ 49.875.974,45	$ 897.767.540,08	
Total costs				$ 1.144.265.039,82

Figure 55: Total costs for the Malecón coastal defence system

While it is difficult for the authors to make an objective statement about the costs it is clear that the proposed solution presents a significant investment from Cuban authorities. This can partly be explained by the high costs of breakwaters and the large increase in costs for higher levels of protection that these come with. This is illustrated in Figure 56; the horizontal axis represents the level of protection in percentage of wave overtopping reduction while the vertical axis shows the costs in million CUC for each percentage point of reduction in wave overtopping.

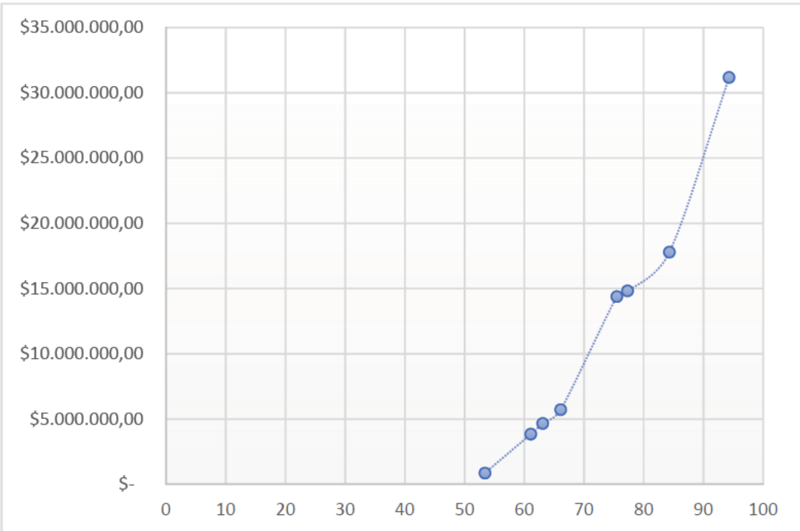

Figure 56: Relative cost increase for level of protection

Figure 56 shows that to achieve a higher wave overtopping reduction the costs grow exponentially. While costs can be expected to be lower in reality with a calculation by a Cuban specialist it does illustrate the large additional investment needed for relatively smaller improvements in overtopping reduction. An economic optimum can be found if the costs of damage due to flooding and growth in investment due to protection are calculated.

10.5. Schedule and phasing

Time estimates for Cuban work conditions deal with many unknown variables such as availability of equipment, material (production capacity from the concrete factory), financial resources and labour force. Therefore, within the timeframe of the project a realistic execution schedule is not produced. General statements regarding scheduling, materials and transportation can be made, including a phasing plan of the sequence of measures.

As stated by previous groups, planned work should be avoided during hurricane season, between July and November in Cuba. To avoid damage to unfinished constructions, which remain valuable, as well as the equipment to be used. Outside of the hurricane season severe weather conditions can still arise which hinder construction works. Wind speeds and wave overtopping values have to be determined for safe working conditions. Important factors that influence working conditions are currents, waves, wind, water level, water depth and available construction space.

Construction works to the wall from land can continue if cautionary measures are taken in minimizing consequences. With a clear evacuation plan of the construction site the damage can be minimized, for example by removing of construction site objects such as cranes, working docks, storage containers, fences and other material to a higher area. Construction of the breakwater and berm should be done in relatively small sections so core material, which is of smaller size, will not be moved during construction. Core material should not be directly exposed to waves or strong currents whenever possible.

The main aim would be to finish sections, berms, or breakwaters within one working season. So construction work of a breakwater should be executed within the 7 months between the hurricane seasons. This way maximum reduction is achieved after one season thus limiting the probability of damage to the unfinished structure.

General gathering of materials

As previously mentioned there is a low availability of rocks in Cuba, especially the large format rock which is required for the armoured layer, this is one of the main reasons for constructing berms and breakwaters using concrete. The smaller material for the core is available from quarries in Cuba. Due to the high amount of concrete required to construct these solutions it is advised to calculate production capacity of nearby concrete factories taking into account amounts consumed from reference projects. It may be necessary to expand existing plants or build a new plant considering the large quantities of concrete used, approximately 680,000 m^3, and extensive duration of the project. Transportation of these elements will be a continuous process, on-land elements will be transported with trucks to the site over the Malecón. Two lanes will be out of service during the construction on land for mobile cranes and delivery of material.

Phasing

The phasing can be determined based on three criteria. The first option is based on maintaining the cultural and economic value of the hinterlands. The second takes the existing condition of the Malecón as critical aspect together with current overtopping values. Finally, the economic value is taken into consideration in terms of efficiency of the applied measure compared to the overtopping reduction.

It is decided to execute the phasing based primarily on the current wave overtopping and critical wall sections while also carefully considering related costs. This results in a preference to construct the curved sea wall first, followed by the berm and lastly the breakwater. Based on the current situation analysis in chapter 3 section two is most critical. The following sequence of construction is therefore proposed:

- Curved wall section 2
- Curved wall section 3
- Curved wall section 4 and 5
- Breakwater section 2/3
- Berm section 2
- Berm section 3
- Berm sections 4 and 5
- Breakwater sections 4 and 5

10.6. Post-proposal reflection

The detailed design is discussed with Professor Córdova in order to better evaluate the feasibility and estimated costs of the project with respect to the other alternatives. The relative cost of the breakwaters is much higher compared to other measures due to the type and size of element used, this means a cheaper alternative appears far more cost-effective as the initial reduction in overtopping is easily achieved with basic measure. An important aspect herein is that the cheaper alternative cannot reach the same level of overtopping reduction such that the target overtopping value can be achieved. The result of purely economic considerations was that the proposal, in its current state, represents unacceptable costs for the current administration and a deeper analysis and discussion of the qualities of the proposal is necessary.

The solution lies in placing more emphasis on the long-term aspects of such a project. The regularity of cold fronts and even Hurricanes, which can cause flooding up to several blocks inland, carries significant costs linked to damage to property, hindrance of transport and reduced tourist activity. The damage caused by the lack of overtopping reduction can be seen as to be added direct maintenance cost, which as discussed early in the report, is less favourable for project proposals. Furthermore, as overtopping is reduced the boulevard of the Malecón will become more attractive to investors which in turn generate revenue for local authorities in the form of subsidies, sale of land or permits. These profits can then be subtracted from projects costs, which makes a project more attractive.
Both these facets are complex phenomena and difficult to adequately quantify. However, it highlights the notion that a solution which performs better over a longer period of time should be preferable. Construction phasing will play an important role in this process; if the more expensive alternative is applied starting with the cheapest structural elements it demonstrates a commitment from Cuban authorities to the improvement of the Malecón and its boardwalk. Consequently this will inspire more confidence from a larger pool of investors who then directly or indirectly contribute to the Cuban economy. This money can then be used to further develop and construct the sea defences around the Malecón and create an upward spiral for Cuban authorities, local inhabitants and investors. The challenge in this approach lies in ensuring a sustained expansion and refinement of the sea defence system stemming from collaboration between local authorities and construction companies.

Another point of discussion raised by Professor Córdova was the length of the breakwaters used. During summer the water circulation around the coast is lower, meaning the breakwaters are likely to hinder the disposal of waste which runs from the city into the sea. In order to improve upon the circulation the longer breakwaters in Section 2 are already reduced from 750 to 500m each, which also contributes to reducing costs. The most critical areas in Section 2 will remain protected for the governing wind direction such that the increase in overtopping is minimised. Applying alternating breakwaters was also considered; this would comprise a similar total length of breakwaters as before but improve water circulation as well cause strong disruption of wave fronts. The cost of an end section of a breakwater however is prohibitive and frequently interrupting the structure would lead to a significant increase in cost.

Based on the points discussed above the proposed solution remains the preferred alternative, assuming a continued effort in careful planning and phasing of construction, while the breakwaters will be modified to accommodate for reduced water circulation during summer. The following chapter will contain the final conclusion of the report and recommendations toward further research.

11. Conclusions and recommendations

11.1. Conclusions

In the current situation the Malecón seawall defence system is not able to withstand the wave forcing and wave overtopping during storm conditions. Unacceptable flooding of the city and damage to the wall are the result. The current situation at the wall differs significantly per section: berm widths can vary from 0 to 12 meters, if drainage pipes are present this can result in extra wave loading at the wall and large sections of the wall are critically damage or cracked. A new storm event or even hurricane event can cause severe damage to the wall and hinterlands.

Looking at the stakeholders in this project, several government and administrative authorities have overlapping roles and the responsible parties must be clearly defined and approached to form a single organisational body. Furthermore, research institutes as the CIH are essential for knowledge and experience on the subject of the Malecón and cooperation should be fostered. Other important stakeholders are the users of the Malecón, however their demands are primarily in line with those of local administration and are therefore likely to be satisfied. Finally, the role of foreign engineering firms remains complex and unclear given the available resources and political climate.

The maximum allowable overtopping over the wall is set at 0.05 $m^3/s/m$, this will be the main focus of the project. Costs must also be minimised in order to propose a feasible solution. From the design criteria and boundary conditions it also follows that the characteristic view and social function of the Malecón are important and must be preserved as much as possible.

The hydraulic boundary conditions with a return period of 50 years are established and the loading parameters are defined using a probabilistic design approach. The main loading parameters are the significant wave height and the water elevation. The significant wave height with a return period of 50 years is 9.52m and the water elevation is 1.95m. The correlation between the significant wave height and the water elevation is still uncertain and is taken in a conservative way, using the Ditlevsen boundary method to calculate the combined return period. After comparing the EurOtop formulas for overtopping with the physical model tests, it can be concluded that the EurOtop manual in case of the Malecón underestimates on average the amount of wave overtopping by 40%.

In order to make a more detailed design, the sections are subdivided in a total of 25 sections. Which are assumed to have same wave conditions, bathymetry and wall height. Running 21 different setups of the SWAN model, it is concluded that combination 1 and a wind coming from the North are the governing conditions in terms of wave overtopping. This combination has an offshore wave height of 6.52m and a water level elevation of 1.95m which are used in later calculations. After using the SWAN output in ANSYS, it can be concluded that the tensile stresses in the top of the new wall and the shear stresses near the sidewalk determine the design conditions for the reinforcement. The compressive stresses present in the structure are not critical, but improvement of the sidewalk will likely be necessary.

To finally arrive at a solution six alternatives are evaluated. These alternatives vary mainly in their application of curved walls, berms and breakwaters. Two extreme alternatives are used as boundary options, used to delimit values for the multi criteria analysis. When looking at the MCA, the 'combination' alternative scores the best with a value of 3.44. However, the 'economic' option also scores very high in the MCA and is probably a good solution when looking at the available resources in Cuba. A post-proposal discussion is made and resulted in the decision to modify the length of the breakwaters in the combination option and to consider a longer time scale to include possible investments and benefits of reduced flood risk.

Using the results coming from the ANSYS program, a reinforcement mesh is created for the new curved wall. In this all different forces and stresses are taken into account, and the amount of steel needed per meter curved wall is calculated to use in the cost estimation.

Figure 57 Overview applied measures Malecón

The positioning and arrangement of the measures applied in the proposed solution is shown in Figure 2Figure 57. At the end of sections 2 and 3 breakwaters are applied to reduce overtopping and over the remainder of these sections berms are placed. The second half of section 3 and first parts of section 4 are protected by a breakwater while the last sections of 4 and 5 will have breakwaters placed offshore. A curved wall will be applied in all sections. The berms will be constructed using

Ecopode units in the armour layer while Accropode units will be applied for the breakwater armour due to the requirements for stability.

Applying these measures results in an average reduction in wave overtopping of 77.3%, the table below represents average values per section. These values do not meet the required values, as this was almost impossible to achieve.

Section	Existing overtopping $m^3/sec/m$	After measures $m^3/sec/m$	Reduction in %
2	1,11	0,12	89,28
3	0,56	0,22	61,17
4	0,48	0,11	77,62
5	0,46	0,12	73,23

The costs of the proposed solution are 1.14 billion CUC, the main contributor to this are the breakwaters which require large amounts of material and cautious construction processes. Phasing is based primarily on reducing overtopping during construction and will be important in ensuring economic feasibility of the project.

11.2. Recommendations

In this chapter recommendations are made for further research for the Malecón coastal defence system.

- Hurricane Event

In this project only three different wind directions are modelled and only one at a time. However, during a hurricane the direction and the force of the wind will change over time, and so will the angle of wave attack. It is recommended to model the solution under hurricane conditions, to better understand the behaviour of the wall, berms and especially the breakwaters in such conditions.

- Breakwater Modelling

Out of the physical model test, it was assumed that the wave setup behind the breakwaters would be zero. However, when modelling the breakwaters in SWAN, a significant wave setup was found present behind the breakwaters resulting in higher amount of wave overtopping than expected. Other aspects that remain uncertain are phenomena of shadowing, energy redistribution due to diffraction and behaviour under a varying wind direction. The model used during this project was a first basic attempt to model the breakwaters in 2D and more research is needed to further investigate the effects of breakwaters in the case of the Malecón.

- Division into subsections

The sections are divided in subsections of 250 meters. It is assumed that wall height, bathymetry and wave climate are constant over the entire subsection. In reality this is not the case and the most important varying parameter will be the bathymetry. To come up with an even better design for the

Malecón it is therefore recommended to use a division with a higher resolution and mainly based on the local bathymetry.

- Solutions for specific sections

In sections 3-5 and 3-6 of the Malecón a breakwater turned out to be not an option, while one was certainly needed. Further research is needed to come up with alternative solutions for this specific point. This research could focus on solutions offshore, near shore or inland. Another point of attention is the heightening of the wall. In the calculations made during this project, the wall height is assumed to be everywhere the desired +4.46m MSL. However, due to the limitations set by the Historian Office, this is not possible at all locations. Further research is needed to come up with solutions for these sections or model the reduction for lower values then +4.46m.

- Verify forcing and structural model

With regard to the calculation of the pressure profiles a number of assumptions were made which should be verified. First and foremost, it is recommended to continue physical model testing of the wall with recurve and include the depth profiles for sections 2 and 3. This allows for verification, or possible correction, of the non-dimensional pressure applied from sections 4 and 5 to the rest of the Malecón. While the approach applied is conservative a modified design per section would reduce material use and therefore costs. It is further recommended to perform physical tests using a sensor in the recurve; this would require a larger scale test setup but would have the added benefit of being able to more accurately map the pressure distribution due to the greater number of sensors.

For the ANSYS model the most important aspect to verify are the assumptions regarding the support conditions at the bottom of the structure. As shown in the analysis of the ANSYS model the stress distributions are significantly affected by the support conditions, as the construction method becomes better defined the connection between the new wall and the berm must be adjusted in the model according to expected strength and elasticity. It is also recommended to create a more accurate 3D model of the design placing dowels at regular intervals along the width and with a bonded connection to the two wall elements to verify the assumptions made regarding epoxy grouting. This should also give a better impression of the stresses which will occur in the dowels and confirm whether the applied dimensions are sufficient. Overall more cooperation with local contractors and engineers is advised in order to evaluate the feasibility of the designed reinforcement mesh and better assess the potential risks of the proposed construction methods.

- Other aspects concerning the project

At last there are some aspects of the project that are only studied briefly or not at all in the past. First thing that should be researched further is the amount of rainfall, the working of the drainage system and flow of water during storm events. As briefly mentioned in this report the drainage system is not fully functional during storm events. This could affect the calculations of wave overtopping and it will affect the amount of flooding.

Secondly, the economic value of the protected area should be determined and the damage due to flooding and severe weather conditions should also be assessed. In this way, when considering the design possibilities, a more accurate estimation of the value of reducing overtopping can be produced to better motivate design choices. If cost of flooding, advantage of protection level in terms of increasing investments and costs are determined an economic most optimal investment level can be determined.

12. Reflection

This chapter will reflect on the period prior, in and after Cuba from the perspective of the students. With the goal to give insight in the process for the supervisors and future groups of students that would continue doing research in Cuba. In the period prior to going to Cuba the amount of research was limited to previous reports by students from the TU Delft. A clear project description with the goal of the research received from Professor Cordova was limited and the project group was formed relatively late. We should have steered to a more extensive proposal so data and literature gathering could have been executed with more accuracy. This would have been a major advantage in Cuba since information gathering is very limited there due to major internet and communication limitations.

As a group overall ambitions and goals were set for the project in terms of quality, quantity and work ethics. These are kept during the execution in Cuba resulting in a conflict free period where our agreed upon expectations from each other were critical. The overall group process can be described as good. Team members were up to date on the progress of others and questions/discussions between disciplines was a daily phenomenon. With the unfortunate exception of Frank his wellbeing forcing him to return to the Netherlands. This made the group process at the end difficult, since as earlier stated communication was limited so collaboration from the Netherlands was not possible for Frank. The results presented in the conceptual report and the presentation were perceived as a success from the guiding professor, committee of the assessor professors and other parties such as members from the Dutch embassy, van Oord and Deltaris. The grade was a ‘excellent’ which equals a 5 at the scale from 1 to 5. As a group this was above our expectations and were pleased with the results. Returned in the Netherlands some minor additional research was conducted, rewriting/finishing of some chapters, addition of the methodology and reflection chapter. With the goal of putting the dots on the i and preparing for the presentation of our works to the TU Delft supervisors.

Due to the limited amount of information upfront and communication restrictions in Cuba the input and guidance from the TU Delft supervisors was minimal. On the other hand the guidance and input from the supervisor in Cuba, professor Cordova, was frequent for Dutch terms. Meetings varying from 10 minutes to 2 hours two till three times a week was common, either individual or as a group. This was in most occasions useful but also sometimes meant an abundance of information, time consuming, raising of more questions and mistimed information. This can best be characterised as chaotic. We can’t say for sure if this would be an cultural difference but this was challenging for the group, which resulted sometimes in frustration and delays. Final point of reflection is the underestimation of the language barrier. The level of English, both spoken and written, is limited in Cuba therefore we were dependent on the interpretation of professor Cordova and Luca his basic Spanish.

The overall group opinion: That this was a very interesting, educational and memorable experience from both educational and personal point of view. Collaborating on such a level for 2 months, an unknown country and culture, learning from different specialisations brought us a lot for future endeavours. And we hope to see some of our results being used/implemented in Havana.

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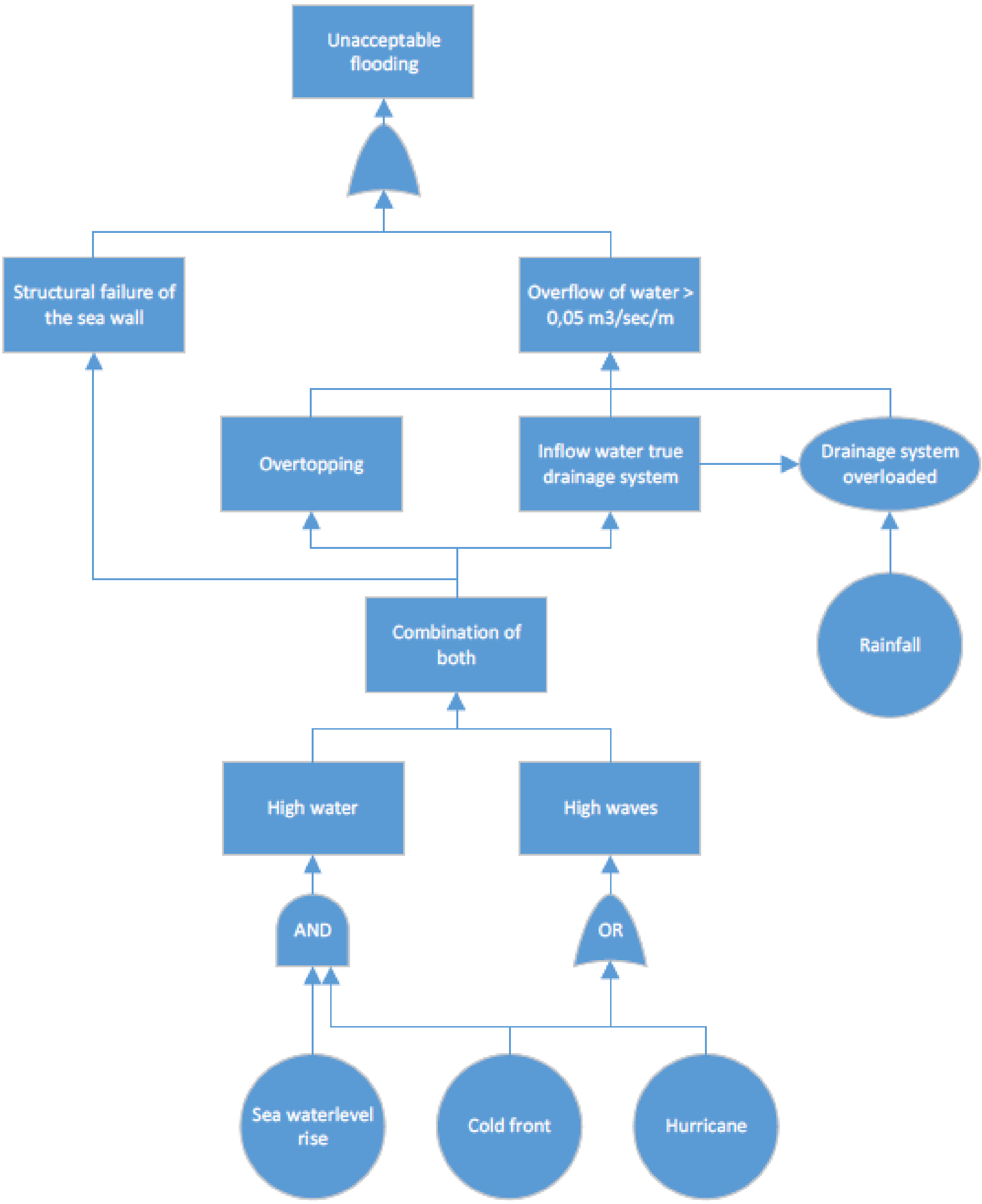
Appendix A. Characteristics current situation

Section	Street part	Height wall MSL	Street level	Berm y/n	notes on berm	Lanes
1	Calle 12					
2	Calle 12 - Calle 10	2,99	0,91	2 - 4 mtr	low	3
	Calle 10 - Calle 8	3,22	0,9	2 - 4 mtr	low	3
	Calle 8 - Calle 6	3,35	0,88	2 - 4 mtr	low	3
	Calle 6 - Calle 4	3,5	0,89	2 - 4 mtr	low	3
	Calle 4 - Calle 2	3,8	0,88	2 - 4 mtr	low	3
	Calle 2 - Paseo	3,9	0,9	2 - 4 mtr	low	3
	Paseo - Calle A	4,07	0,9	3,00	Stairs constuction	3
	Calle A - Calle B	3,31	0,9	2 - 4 mtr		3
	Calle B - Calle C	3,71	0,89	No	Large parts missing	3
	Calle C - Calle D	4,15	0,92	1-2 mtr	Large parts missing	3
	Calle D - Calle E	3,79	0,9	2,00	Big drain	3
	Calle E - Calle F	4,32	0,89	2,00		3
	Calle F - Calle G	3,89	0,88	4 - 2 mtr		3
	Calle G - Calle H	4,35	0,93	4 to 3 mtr	Parts missing	3
	Calle H - Calle J	4,43	1 - 0,90	4,00		3
3	Calle J - Calle K	4,3	1,13	4 to 6 mtr	Build like a ramp	3
	Calle K - Calle L	4,3	1,12	4 to 6 mtr	Build like a ramp	3
	Calle L - Calle M	4,3	1,13	2 to 8 mtr	high	3
	Calle M - Calle N	4,3	1,12	6 to 12 mtr	Several parts missing	3
	Calle N - Calle O	4,3	1,1	4 to 10 mtr	Relatifly low	3
	Calle O - Calle P	4	0,84	1-2 mtr	Relatifly low	3
	Calle P - Calle 23	4	0,81	4 to 10 mtr	Enterance drainage	3
	Calle 23 - Calle 25	4	0,83	4 to10 mtr	Parts missing of ca. 10 mtr	3
	Calle 25 - Principe	4	0,81	6 to 12 mtr	Parts missing of ca. 10 mtr	3
	Principe - Calle Marina	4	0,81	6 to 12 mtr	Parts missing of ca. 10 mtr	3
4	Calle Marina (start section)					
	Parque Antonio Maceo	3,98	0,85	2 to 3 mtr	Natrual berm	3
	Belascoain - Gervasio	4,33	1,2	2 mtr	Big squere rocks	3
	Gervasio - Escobar	4,33	1,2	2 mtr	Natural berm	3
	Escobar - Lealtad	4,33	1,2	2 mtr	Natural berm	3
	Lealtad - Perseverancia	3,97	0,75	3 - 4 mtr	Natural berm	3
	Perseverancia - Campanario	3,97	0,75	4 - 6 mtr	Natural berm	3
	Campanario - Manrique	3,97	0,7	4 - 6 mtr	Natural berm, some holes	3
	Manrique - San Nicolas	3,97	0,7	4 - 6 mtr	Natural berm, some holes	3
	San Nicolas - Calle Galiano	3,97	0,55	4 - 6 mtr	Natural berm, some holes	3
5	Calle Galiano - Blanco	3,97	0,7	8 mtr	Swimmingpools	3
	Blanco - Aguila	3,97	0,68	8 mtr	Swimmingpools	3
	Aguila - Crespo	3,97	0,68	8 - 12 mtr	Swimmingpools	3
	Crespo - Genios	3,94	0,72	15 -20 mtr	Swimmingpools	3
	Genios - Castillo de la Punta	3,94	0,72	10 - 12 mtr	Natrual berm, some holes	3
6	Castillo de la Punta					
	Between La Punta					

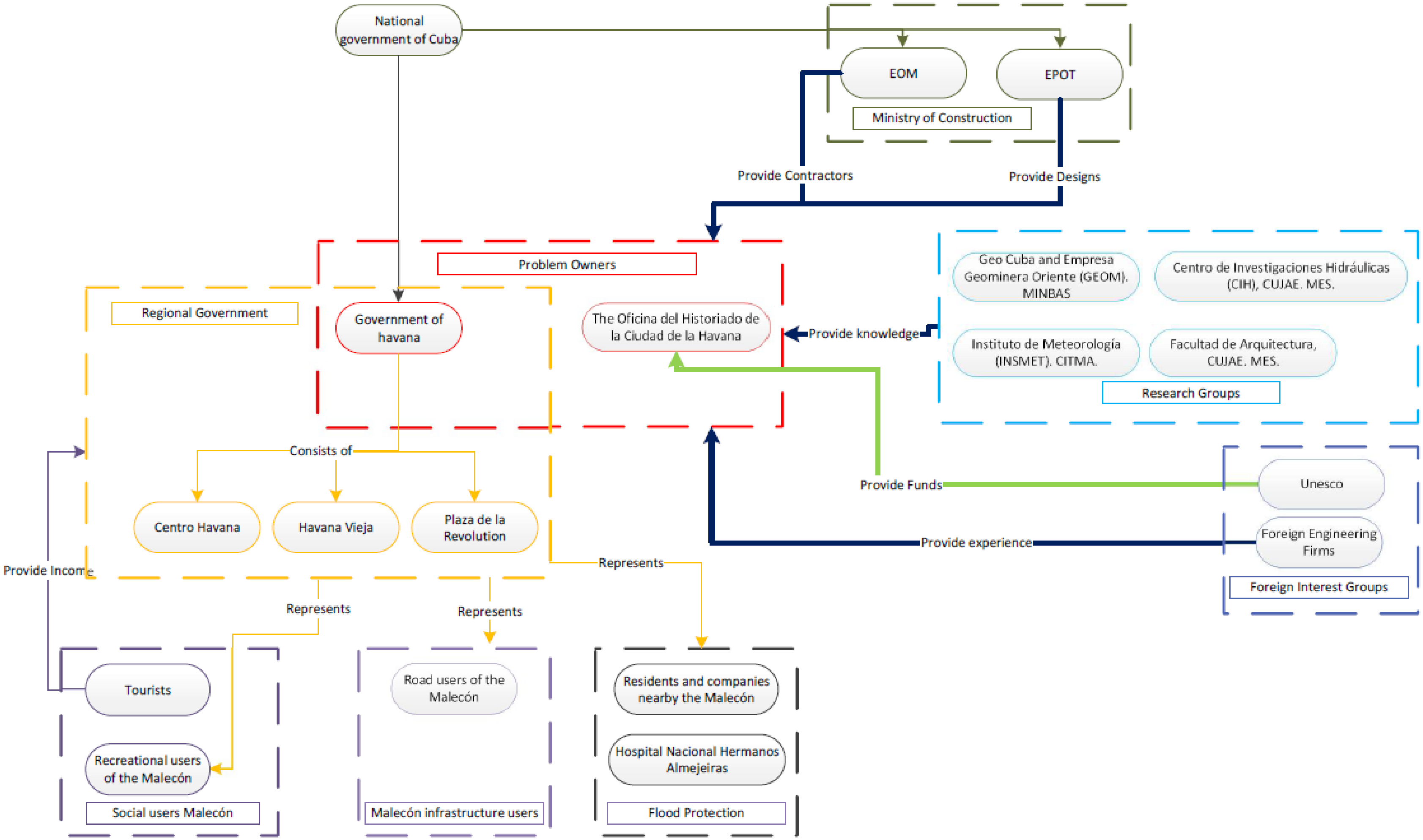
Appendix B. Structural assessment current situation

Section	street part	Top parts broken off	Reinforcement steel visual	Concrete corrosion	Severe cracks	Sum
1	**Calle 12**					
2	Calle 12 - Calle 10	0	0	0	0	0
	Calle 10 - Calle 8	0	0	0	0	0
	Calle 8 - Calle 6	0	0	0	0	0
	Calle 6 - Calle 4	0	0	0	0	0
	Calle 4 - Calle 2	0	0	0	2	2
	Calle 2 - Paseo	0	0	0	0	0
	Paseo - Calle A	1	1	1	1	4
	Calle A - Calle B	0	0	0	2	2
	Calle B - Calle C	1	1	1	1	4
	Calle C - Calle D	1	1	1	1	4
	Calle D - Calle E	0	0	0	2	2
	Calle E - Calle F	0	0	0	2	2
	Calle F - Calle G	1	1	1	1	4
	Calle G - Calle H	0	0	0	1	1
	Calle H - **Calle J**	1	1	1	1	4
3	**Calle J** - Calle K	0	0	0	1	1
	Calle K - Calle L	0	0	0	2	2
	Calle L - Calle M	0	0	0	1	1
	Calle M - Calle N	0	0	0	2	2
	Calle N - Calle O	0	0	0	2	2
	Calle O - Calle P	1	0	0	0	1
	Calle P - Calle 23	1	1	1	0	3
	Calle 23 - Calle 25	0	0	0	0	0
	Calle 25 - Principe	0	0	0	1	1
	Principe - **Calle Marina**	0	0	0	1	1
4	**Calle Marina (start section)**					
	Parque Antonio Maceo	0	0	0	0	0
	Belascoain - Gervasio	0	0	0	0	0
	Gervasio - Escobar	0	0	0	1	1
	Escobar - Lealtad	0	0	0	1	1
	Lealtad - Perseverancia	0	0	0	0	0
	Perseverancia - Campanario	0	0	0	0	0
	Campanario - Manrique	0	0	0	1	1
	Manrique - San Nicolas	0	0	0	0	0
	San Nicolas - **Calle Galiano**	0	0	0	0	0
5	**Calle Galiano** - Blanco	0	0	0	0	0
	Blanco - Aguila	0	0	0	0	0
	Aguila - Crespo	0	0	0	0	0
	Crespo - Genios	0	0	0	0	0
	Genios - **Castillo de la Punta**	0	0	0	0	0
6	**Castillo de la Punta**					
	Between La Punta					

Appendix C. Fault Tree Analysis



Appendix D. Stakeholder map



Appendix E. Stakeholders: Power and Interest

	Actors	Power High - low	Interest High - low	Interest
Governmental	Government of Havana	High	Medium	Protection of Havana against flooding, touristic development, traffic and policy
	Oficina del Historiado de la Ciudad de la Havana	High	High	Preserve and restore the Malecón while maintaining its Characteristic view
	Enterprise of Projects of Transport Works (EPOT)	Medium	Medium	Design company for the government who will execute the final design
	Enterprise of Maritime Works (EOM)	Medium	Medium	Party that will execute the works, interest in design meeting their construction abilities
	National government of Cuba	High	Medium	Policy, financing and high interest in touristic development
Research	Centro de investigaciones Hidráulicas (CIH), CUJAE	Medium	High	Conduction of research regarding design, hydraulic boundary conditions and tests
	Geo Cuba and Empresa Geominera Oriente (GEOM)	Low	Medium	Geotechnical research
	Instituto de Meteorologica	Low	Medium	Hydraulic boundary conditions
	Facultad de Arquitectura, CUJAE	Low	Medium	Preservation of the view and esthetical design part
Users	Recreational users of the Malecón	Low	Medium	Enjoy the social aspects of the Malecón, view of the ocean
	Road users of the Malecón	Low	Medium	Use the infrastructure on the Malecón
	Residents and companies nearby the Malecón	Medium	High	Protection against water and maintaining view on the ocean
Other	Hospital	Low	High	Protection against water, keeping operationalizable during high water
	Engineering firms	Low	Medium	Make a profit by renovating the Malecón, investing in durable relation Cuba
	UNESCO	Medium	High	Preserve the interesting architecture of Havana Vieja, protection of cultural heritage

Appendix F. Stakeholders: Critical Actor assessment

	Actors	Important Resources	Replaceability (high/low)	Dependency (low,moderate,high)	Critical Actor (yes/no)
Governmental	Government of Havana	Yes	Low	High	Yes
	Oficina del Historiado de la Ciudad de la Havana	Yes	Low	High	Yes
	Enterprise of Projects of Transport Works (EPOT)	Yes	Low	High	Yes
	Enterprise of Maritime Works (EOM)	Yes	Low	High	Yes
	National government of Cuba	No	Low	High	Yes
Research	Centro de investigaciones Hidráulicas (CIH), CUJAE	Yes	Low	Moderate	Yes
	Geo Cuba and Empresa Geominera Oriente (GEOM)	Yes	Low	Moderate	No
	Instituto de Meteorologica	Yes	Low	Moderate	No
	Facultad de Arquitectura, CUJAE	No	Low	Low	No
Users	Recreational users of the Malecón	No	Medium	Low	No
	Road users of the Malecón	No	Low	Low	No
	Residents and companies nearby the Malecón	No	Low	Low	No
Other	Hospital	No	Low	Low	No
	Engineering firms	Yes	High	Low	No
	UNESCO	Yes	Low	Medium	Yes

Appendix G. Stakeholders: Engagement Plan

	Actor	Level of Involvement	Concrete steps
Governmental	Government of Havana	Collaborate	Government of Havana is a problem owner. They must be part of the collaboration. They need to be activated. Check funding possibilities
	Oficina del Historiado de la Ciudad de la Havana	Collaborate	Collaborate with them, check funding possibilities, designing solution with their requirements in mind
	Enterprise of Projects of Transport Works (EPOT)	Involve	Involve them later in the process, when initial design is finished
	Enterprise of Maritime Works (EOM)	Involve	Involve them later in the process, when initial design is finished
	National government of Cuba	Inform	Regular updates in order to keep them informed. If not final design may not be accepted by them and project cannot be completed/long delays as a result of new designs needed
Research	Centro de Investigaciones Hidráulicas (CIH), CUJAE	Collaborate	Collaborate to use their extensive knowledge of the Malecón for optimal solutions. CIH is currently the expert on the Malecón Coastal defence system
	Geo Cuba and Empresa Geominera Oriente (GEOM)	Consult	Receive information on soil conditions around the Malecón
	Instituto de Meteorologica	Consult	Receive information on historic climate data and hurricanes
	Facultad de Arquitectura, CUJAE	Consult	Consult on aesthetical design options for the Malecón
Users	Recreational users of the Malecón	Inform	Inform of final design and construction period. Their Wishes are included in requirements of other stakeholders (reduce wave overtopping, keep social aspect available. Both in requirements of historical office
	Road users of the Malecón	Inform	Inform of construction period, this way they can adjust their routes if necessary
	Residents and companies nearby the Malecón	Inform	Inform of final design and construction period
Others	Hospital	Inform/Consult	Consult on requirements and take them into account while designing solutions
	Engineering firms	Consult/Involve/Collaborate	Depends on the necessary level of involvement. Their equipment may be necessary. As well as their skills/knowledge/expertise in flood defence
	UNESCO	Consult/Involve	Check requirements of UNESCO for the Malecón and possible funding the project could receive for maintaining the character of the Malecón

Appendix H. Meteorological data for Havana, Cuba

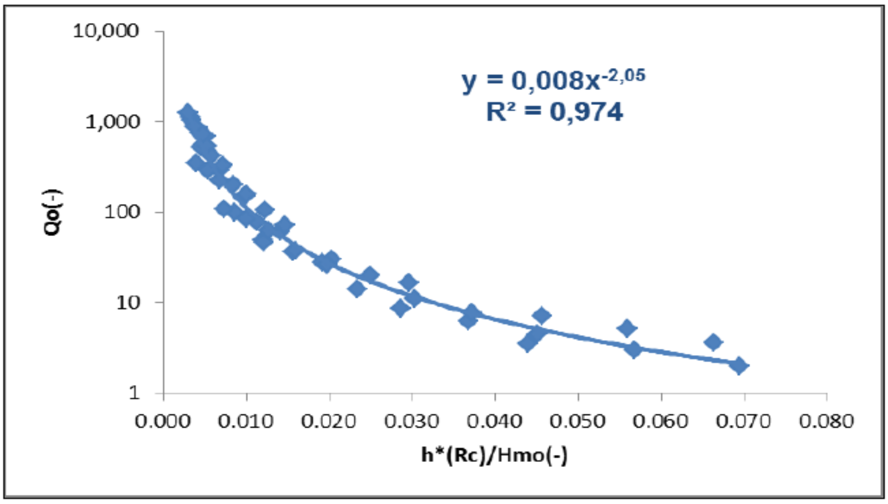
Date	Wind direction	Wind speed [m/s]	H_s [m]	T_p [s]
25-09-1975	-	-	5.4	8.2
28-10-1985	-	-	5.8	8.9
19-11-1985	NNE	18.2	5.0	10.7
12-10-1987	NW	20.5	5.2	9.4
14-11-1994	NNW	18.2	3.5	8.1
04-10-1995	-	-	3.3	6.3
28-09-1998	N	22.7	3.9	11.2
15-10-1999	NW	13.6	3.7	9.4
17-09-2000	N	22.7	3.9	11.2
04-11-2001	NE	25.0	4.5	11.8
26-08-2005	N	18.2	5.3	10.7
20-09-2005	N	13.6	3.7	11.1
24-10-2005	NW	22.7	5.8	11.3

Date	Wind direction	Wind speed [m/s]	H_s [m]	T_p [s]
03-02-1970	NW	18.2	5.8	10.0
19-01-1977	NW	20.5	5.5	9.6
03-01-1979	NNW	15.9	5.7	10.3
02-03-1980	NW	13.6	4.2	7.7
05-11-1982	NNE	9.1	2.5	6.2
17-03-1983	-	-	5.6	8.4
28-02-1984	NW	13.6	4.3	7.7
29-03-1984	NW	13.6	3.6	7.7
23-11-1984	N	11.4	3.1	7.0
04-01-1985	NNW	13.6	3.7	7.7
12-02-1985	NW	9.1	3.4	6.8
05-01-1987	NW	13.6	5.1	8.3
25-01-1987	NW	13.6	3.9	7.7
25-01-1988	NNW	11.4	3.5	7.0
12-04-1988	NW	13.6	4.5	7.7
15-02-1991	NW	13.6	4.0	7.7
06-02-1991	NW	13.6	5.3	9.2
13-03-1993	NW	13.6	5.3	9.2
03-03-1994	NW	13.6	4.6	7.7
23-12-1994	NW	11.4	3.8	8.3
08-01-1996	NW	13.6	3.6	7.7
04-02-1996	NNW	13.6	3.6	7.7
08-03-1996	NNW	15.9	3.4	8.4
20-03-1996	NW	11.4	3.8	8.3
14-12-1997	NW	13.6	3.6	7.7
27-12-1997	NW	11.4	3.4	6.2
04-02-1998	WNW	15.9	4.6	8.7
15-03-1999	NW	13.6	3.7	7.7
24-01-2000	NW	13.6	3.4	7.7
20-03-2001	NW	13.6	3.7	7.7
23-02-2002	NW	13.6	4.7	9.2
24-11-2005	NW	9.1	3.5	7.7

Appendix I. Detailed results of the physical model tests (L.F. Córdova et al, 2016)

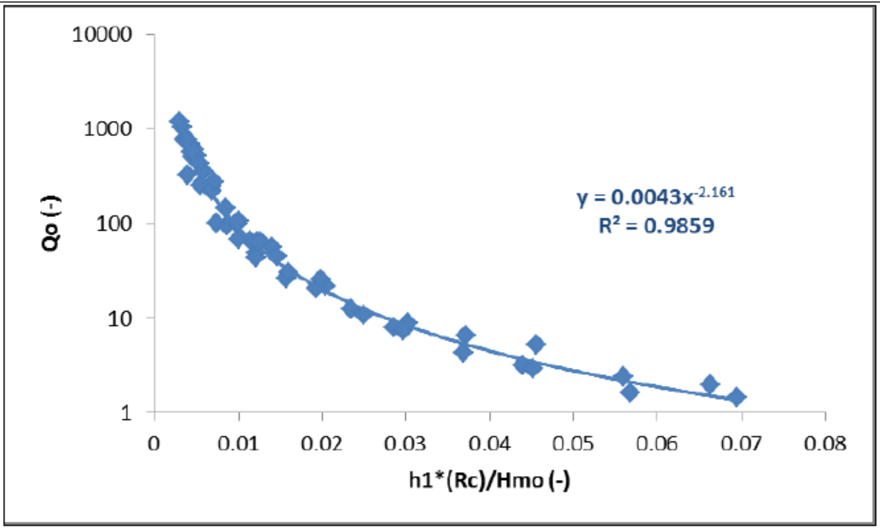
Vertical Wall

Physical model results for the vertical wall. The regression line is plotted with a correlation of 0,974.



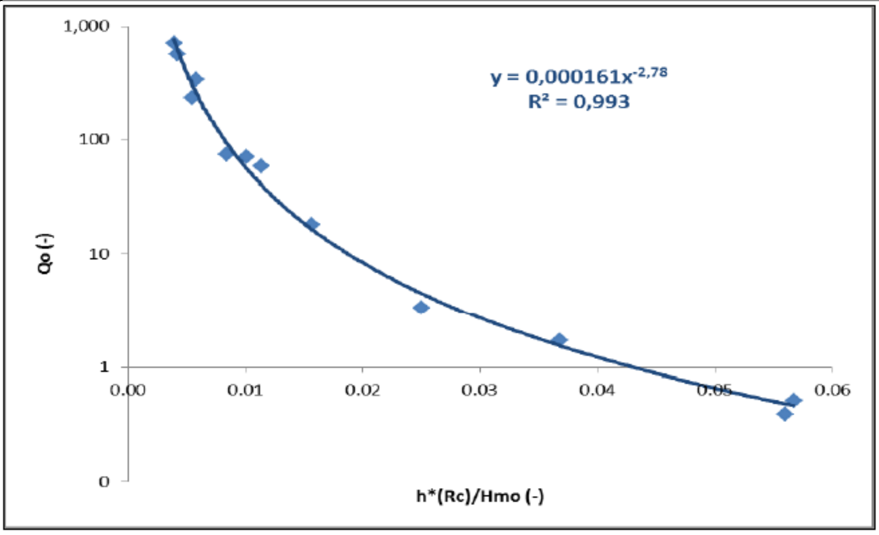
Curved Wall

Physical model results for the curved wall. The regression line is plotted with a correlation of 0,9859.



Curved Wall + Berm

Physical model results for the curved wall in combination with a berm. The regression line is plotted with a correlation of 0,993.



Appendix J. SWAN Model

In this table the different settings are briefly discussed.

Setting	Explanation
SET	This alters general parameters of the model. In this model the water level is increased.
MODE	SWAN is run in a stationary 2-D mode
COORD	The coordinate system used during the modelling is a Cartesian one
CGRID	The computational grid is defined as a rectangular grid. Input data requires X0, Y0, rotation, length, width, number of meshes, mesh size. The spectral directions cover the full CIRCLE and is divided in 36 meshes. Lowest frequency use is 0.03 Hz and the highest is 1 Hz.
WIND	The wind that is acting in de model. The speed is taken as 25 m/s and the angle differs between the models.
BOU SHAPE	The shape of the wave spectrum at the boundary is assumed a JONSWAP spectrum. A peak enhanced factor of 3.3 is used and the PEAK period is used as characteristic wave period. DSPR is used for expressing the width of the directional distribution and is expressed in DEGREES.
BOU SIDE	For all sides the boundary conditions are given by input Parameters. These are Significant wave height, characteristic period, peak wave direction and a coefficient of directional spreading.
GEN3	SWAN is run in a third generation mode. The mode is KOMEN.
WCAP	White capping of waves is included in the calculation.
QUAD	Quadruplet wave-wave interactions are included in the calculation.
BREAKING	Wave breaking is included in the calculation, using a constant breaker index.
FRICTION	Bottom friction is taken into account. The JONSWAP results for bottom friction dissipation are used.
TRIADS	Triad wave-wave interactions are taken into account.
SETUP	Wave induced set-up is computed and accounted for in the computations
NESTout	This gives a 2D wave spectra along a nest boundary.

SWAN Input file to compute wave transformation from the offshore to the near shore situation.

```
!****************DESCRIPTION****************
!
PROJECT  'Combination'  '453'
!
!     Nesting project Complete Malecon Seawall
!     Wave propagation towards the shore
!     Coarse Grid (1/3)
!     November 2017
!****************MODEL INPUT****************
!
SET 1.50 NAUT
MODE STAT TWOD
!
COORD  CART
!
CGRID  REG 3.209e+5 3.532e+5 0.0 75000. 100000. 75 100 CIRCLE 36 0.03 1. 31
INPGRID BOT REG 3.209e+5 3.532e+5 0.0 75 100 1000. 1000. EXC -1.00000000000e+003
READINP BOT 1. 'z-table 76x101.bot' 3 0 FREE
!
WIND 25.0 45
!
BOU SHAPE JON 3.3 PEAK DSPR DEGREE
BOU SIDE N CCW CON PAR 9.14 12.0 45 30
BOU SIDE W CCW CON PAR 9.14 12.0 45 30
BOU SIDE E CCW CON PAR 9.14 12.0 45 30
!
!****************PHYSICS****************
!
GEN3 KOMEN
WCAP
QUAD
BREAKING
FRICTION
TRIADS
SETUP
!
!****************OUTPUT****************
!
BLOCK 'COMPGRID' NOHEAD '1RunMalecon.mat' LAY 4 XP YP HS BOTLEV
NGRID 'MALECON' 3.434e+5 3.600e+5 0 30e+3 30e+3 300 300
NESTout 'MALECON' '1RunSpec.spc'
!
TEST 0 0
COMPUTE
STOP
```

SWAN Input file in order to generate depth profiles in a high resolution (1m x 1m).

```
!****************DESCRIPTION****************
!
PROJECT  'Depth profiles'  '404'
!
!     Nesting project Complete Malecon Seawall
!        Wave propagation towards the shore
!        December 2017
!
!****************MODEL INPUT****************
!
SET 0.0 NAUT
!
MODE STAT TWOD
COORD  CART
!
CGRID  REG 3.557e+5 3.68e+5 0.0 1000 1000 1000 1000 CIRCLE 36 0.03 1. 31
!
INPGRID BOT REG 3.557e+5 3.68e+5 0.0 1000 1000 1. 1. EXC -1.00000000000e+003
READINP BOT 1. 'z-table1.bot' 3 0 FREE
!
WIND  10.0 0
!
!
!
!****************PHYSICS****************
!
GEN3 KOMEN
NUM STOPC 0.005 0.01 0.005 99.5 STAT 0 0.00 0.1
WCAP
QUAD
BREAKING
FRICTION
TRIADS
SETUP
!
!****************OUTPUT****************
!
CURVE 'sec21' 356055 368000 500 355715 369000
CURVE 'sec22' 356959 368000 500 356065 369000
CURVE 'sec23' 356700 368520 500 356435 369000
!
TABLE 'sec21' HEADER 'sec21.tab' XP YP BOTLEV
TABLE 'sec22' HEADER 'sec22.tab' XP YP BOTLEV
TABLE 'sec23' HEADER 'sec23.tab' XP YP BOTLEV
!
TEST 0 0
COMPUTE
STOP
```

Appendix K. Comparison between EurOtop Manual and physical model test for section 4

			Cordova 2016 Physical Model Test					EurOtop Manual							
Section	Wind Direction	Combination	hs	h*	Rc	Q0	q Overtop	a	b	Valid	h*	Q0	q Overtop	Comparison
		#	m	-	m	m3/s	l/s/m			h*(Rc/Hm0)	m	m3/s/m	l/s/m	EurOtop - Physical Model
4	45	0	3.17980	0.0199	1.948	61.02	428	0.00027	-2.7	0.013	0.0199	35.06	246	-42.55%
		1	3.13150	0.0202	1.996	51.01	361	0.00027	-2.7	0.014	0.0202	27.69	196	-45.72%
		2	3.09200	0.0196	2.036	52.88	345	0.00027	-2.7	0.014	0.0196	29.03	189	-45.10%
		3	2.76610	0.0165	2.362	49.56	195	0.00027	-2.7	0.014	0.0165	26.65	105	-46.22%
		4	2.94430	0.0181	2.184	52.26	270	0.00027	-2.7	0.014	0.0181	28.58	147	-45.31%
		5	3.03270	0.0189	2.095	53.21	314	0.00027	-2.7	0.014	0.0189	29.27	173	-44.99%
		6	3.42720	0.0234	1.701	56.27	612	0.00027	-2.7	0.013	0.0234	31.51	342	-44.01%
	0	0	3.33160	0.0204	1.796	79.09	624	0.00027	-2.7	0.011	0.0204	49.33	389	-37.63%
		1	3.27150	0.0201	1.856	72.21	**541**	0.00027	-2.7	0.012	0.0201	43.76	328	-39.40%
		2	3.24040	0.0196	1.888	74.18	522	0.00027	-2.7	0.012	0.0196	45.34	319	-38.88%
		3	2.92060	0.0171	2.207	61.12	281	0.00027	-2.7	0.013	0.0171	35.13	161	-42.52%
		4	3.09960	0.0186	2.028	67.11	395	0.00027	-2.7	0.012	0.0186	39.73	234	-40.79%
		5	3.18200	0.0193	1.946	69.60	460	0.00027	-2.7	0.012	0.0193	41.69	275	-40.10%
		6	3.55750	0.0227	1.570	87.18	944	0.00027	-2.7	0.011	0.0227	56.08	608	-35.67%
	315	0	3.26230	0.0204	1.866	66.36	510	0.00027	-2.7	0.012	0.0204	39.15	301	-41.00%
		1	3.20060	0.0207	1.927	54.19	417	0.00027	-2.7	0.014	0.0207	29.98	231	-44.67%
		2	3.16700	0.02	1.961	58.21	409	0.00027	-2.7	0.013	0.0200	32.94	232	-43.41%
		3	2.85190	0.0171	2.276	52.82	233	0.00027	-2.7	0.014	0.0171	28.99	128	-45.12%
		4	3.02610	0.0185	2.102	57.38	323	0.00027	-2.7	0.013	0.0185	32.32	182	-43.66%
		5	3.10900	0.0193	2.019	58.60	374	0.00027	-2.7	0.013	0.0193	33.23	212	-43.28%
		6	3.49820	0.0237	1.630	63.56	730	0.00027	-2.7	0.013	0.0237	36.99	425	-41.80%

Appendix L. Physical model test data

Tabla 45. Resultados de las presiones para el sensor 0.

Pruebas	Altura de ola en aguas profundas Hso (m)	Período pico Ts0 (s)	Altura de ola a 20 m de profundidad Hs20 (m)	Altura de ola frente al muro Hs muro (m)	Nivel del mar (m)	S0 (Pmax/1025*Hs frente al muro)						Presión (T/m2)	Máximo evento S0
PORCIENTO DE EXCEDENCIA (%)						50	25	10	5	1	250		
1	4	12	2,7	2,51	2,28	1,66	3,80	7,07	10,70	22,20	**30,70**	**78,98**	**34**
2	6	12	4	2,99	2,28	1,92	3,77	6,14	8,43	20,90	**51,50**	**157,83**	**73**
3	8	12	5,4	3,24	2,28	1,71	3,32	5,11	7,15	12,50	**32,10**	**106,60**	**47**
4	10	12	6,5	3,55	2,28	1,79	2,91	4,50	6,80	11,10	**34,30**	**124,81**	**46**
5	4	10	2,7	2,44	2,28	1,42	3,14	6,12	9,89	16,70	**29,50**	**73,78**	**29**
8	10	10	6,5	3,36	2,28	1,54	2,97	5,25	7,33	18,30	**34,10**	**117,44**	**38**
9	4	12	2,7	1,82	1,73	1,66	4,09	8,58	12,60	31,70	**51,60**	**96,26**	**52,5**
10	6	12	4	2,09	1,73	1,71	3,40	6,58	10,70	18,60	**38,50**	**82,48**	**40**
11	8	12	5,4	2,32	1,73	2,07	3,56	6,42	10,10	19,30	**35,70**	**84,89**	**43**
12	10	12	6,5	2,63	1,73	1,92	3,56	5,33	6,47	12,90	**20,90**	**56,34**	**25,8**
13	4	10	2,7	1,65	1,73	1,57	3,87	8,15	12,20	24,00	**42,60**	**72,05**	**45**
16	10	10	6,5	2,28	1,73	1,61	2,90	4,99	6,60	14,40	**25,90**	**60,53**	**27,5**

Tabla 46. Resultados de las presiones para el sensor 1.

Pruebas	Altura de ola en aguas profundas Hso (m)	Período pico Ts0 (s)	Altura de ola a 20 m de profundidad Hs20 (m)	Altura de ola frente al muro Hsmuro (m)	Nivel del mar (m)	S1 (Pmax/1025*Hs frente al muro)						Presión (T/m2)	Máximo evento S1
PORCIENTO DE EXCEDENCIA (%)						50	25	10	5	1	250		
1	4	12	2,7	2,51	2,28	1,64	3,53	6,1	8,61	14,5	**18,1**	**45,431**	**19,8**
2	6	12	4	2,99	2,28	2	3,38	5,81	8,03	13,5	**47,7**	**142,62**	**52**
3	8	12	5,4	3,24	2,28	1,79	3,11	4,71	5,97	11,1	**15,4**	**49,896**	**17**
4	10	12	6,5	3,55	2,28	1,78	2,75	3,98	5,39	9,08	**13**	**46,15**	**15**
5	4	10	2,7	2,44	2,28	1,31	2,78	4,64	6,03	11,5	**17,3**	**42,212**	**19**
8	10	10	6,5	3,36	2,28	1,6	3,03	5,38	6,87	9,63	**11,5**	**38,64**	**12**
9	4	12	2,7	1,82	1,73	2,32	5,37	10,3	14,8	26,9	**50,9**	**92,638**	**55**
10	6	12	4	2,09	1,73	2,39	4,56	8,07	10,7	17,8	**25,6**	**53,504**	**25**
11	8	12	5,4	2,32	1,73	2,77	4,33	6,98	8,89	15,8	**33,7**	**78,184**	**37**
12	10	12	6,5	2,63	1,73	2,66	4,06	6,28	7,98	21	**35,3**	**92,839**	**37**
13	4	10	2,7	1,65	1,73	2,13	4,89	8,06	10,3	22,2	**31,8**	**52,47**	**33**
16	10	10	6,5	2,28	1,73	2,07	3,26	5,16	7,25	12,6	**46,9**	**106,93**	**72**

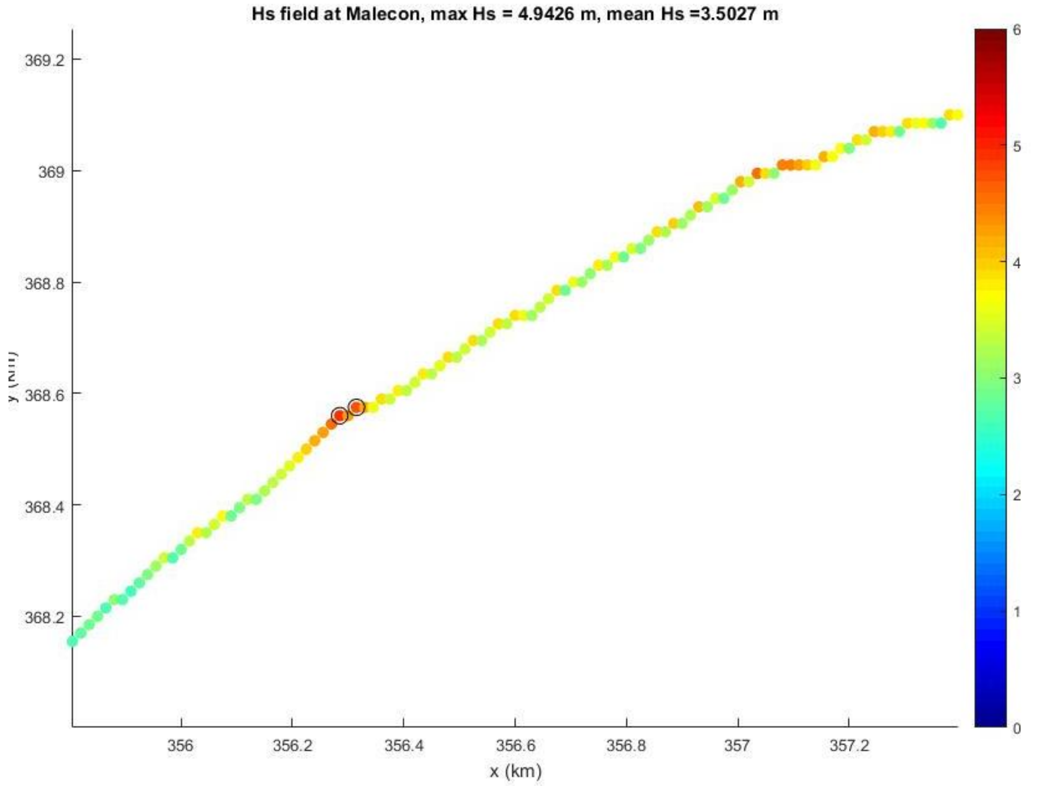
Appendix M. SWAN Results

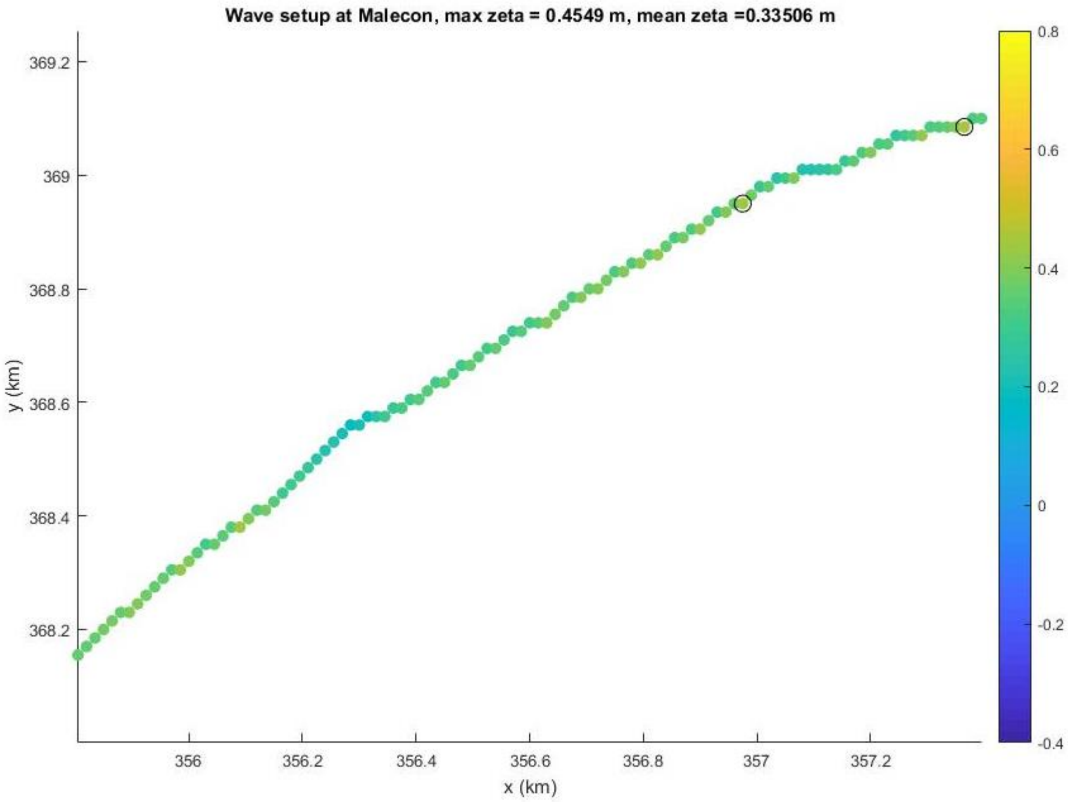
							Input from SWAN				Cordova 2016 Physical Model Test				
Section	Wind Direction	Combination	Height wall	Elevation	BotLev	Tp	Max SigH	MeanSigH	Max Zeta	Zeta Mean	hs	h*	Rc	Q0	q Overtop
		#	MSL +m	MSL +m	m	s		m	Max Zeta	m	m	-	m	m3/s	l/s/m
2	45	0	3.78	1.95	1.234	12.044	4.6910	3.4502	0.4230	0.2301	3.41409	0.0201	1.600	115.87	929
		1	3.78	1.95	1.234	12.044	4.3365	3.3308	0.3553	0.1547	3.33870	0.02	1.675	100.01	761
		2	3.78	1.88	1.234	12.044	4.4719	3.3329	0.3855	0.1985	3.31250	0.0196	1.702	100.32	730
		3	3.78	1.50	1.234	12.044	4.6310	3.1323	0.4671	0.2745	3.00850	0.0172	2.006	82.39	400
		4	3.78	1.70	1.234	12.044	4.5212	3.2662	0.4362	0.2439	3.17790	0.0184	1.836	93.64	564
		5	3.78	1.80	1.234	12.044	4.5290	3.3104	0.4179	0.2278	3.26180	0.0192	1.752	97.87	663
		6	3.78	2.28	1.234	12.044	4.3099	3.4206	0.2970	0.1123	3.62630	0.0229	1.388	116.94	1328
	0	0	3.78	1.95	1.234	12.044	5.0293	3.5964	0.5454	0.4232	3.60720	0.0216	1.407	142.70	1424
		1	3.78	1.95	1.234	12.044	4.9426	3.5027	0.4549	0.3351	3.51910	0.0211	1.495	125.13	**1149**
		2	3.78	1.88	1.234	12.044	5.0031	3.5117	0.5116	0.3915	3.50550	0.0209	1.509	126.12	1128
		3	3.78	1.50	1.234	12.044	4.6841	3.3117	0.5692	0.4681	3.20210	0.0185	1.812	98.72	604
		4	3.78	1.70	1.234	12.044	4.8340	3.4590	0.5778	0.4423	3.37630	0.0196	1.638	116.83	876
		5	3.78	1.80	1.234	12.044	4.8899	3.4671	0.5515	0.4186	3.45260	0.0205	1.561	118.69	1002
		6	3.78	2.28	1.234	12.044	4.9851	3.6943	0.4205	0.2723	3.78630	0.0231	1.228	172.67	2132
	315	0	3.78	1.95	1.234	12.044	4.9189	3.5824	0.5181	0.4131	3.59710	0.0215	1.417	139.99	1387
		1	3.78	1.95	1.234	12.044	4.6962	3.4445	0.4072	0.3050	3.48900	0.0211	1.525	116.17	1052
		2	3.78	1.88	1.234	12.044	4.8329	3.4836	0.4750	0.3684	3.48240	0.0208	1.532	121.54	1065
		3	3.78	1.50	1.234	12.044	4.5987	3.2909	0.5669	0.4601	3.19410	0.0185	1.820	96.32	588
		4	3.78	1.70	1.234	12.044	4.7144	3.4059	0.5326	0.4275	3.36150	0.0198	1.653	109.60	828
		5	3.78	1.80	1.234	12.044	4.7721	3.4519	0.5056	0.4026	3.43660	0.0204	1.577	116.35	966
		6	3.78	2.28	1.234	12.044	4.7632	3.6047	0.3388	0.2470	3.76100	0.0234	1.253	153.91	1924
Section	Wind Direction	Combination	Height wall	Elevation	BotLev	Tp	Max SigH	MeanSigH	Max Zeta	Zeta Mean	hs	h*	Rc	Q0	q Overtop
		#	MSL +m	MSL +m	m	s		m	Max Zeta	m	m	-	m	m3/s	l/s/m
3	45	0	4.15	1.95	1.2544	12.044	5.3395	3.1237	0.5592	0.3269	3.53130	0.0238	1.873	48.59	572
		1	4.15	1.95	1.2544	12.044	4.9911	3.0212	0.4714	0.2544	3.45880	0.0236	1.946	42.68	479
		2	4.15	1.88	1.2544	12.044	5.1497	3.0258	0.5148	0.2972	3.43160	0.0232	1.973	43.12	462
		3	4.15	1.50	1.2544	12.044	5.1023	2.8302	0.5877	0.3659	3.12030	0.0205	2.284	35.85	260
		4	4.15	1.70	1.2544	12.044	5.1663	2.9405	0.5620	0.3392	3.29360	0.022	2.111	39.50	358
		5	4.15	1.80	1.2544	12.044	5.1966	2.9969	0.5478	0.3248	3.37920	0.0227	2.025	41.84	420
		6	4.15	2.28	1.2544	12.044	4.9744	3.1636	0.4130	0.2060	3.74040	0.0264	1.664	51.53	811
	0	0	4.15	1.95	1.2544	12.044	5.3800	3.1929	0.6620	0.4117	3.61610	0.0244	1.788	53.03	681
		1	4.15	1.95	1.2544	12.044	5.0740	3.0951	0.5858	0.3415	3.54590	0.0242	1.859	46.75	**573**
		2	4.15	1.88	1.2544	12.044	5.2203	3.1168	0.6379	0.3862	3.52060	0.0237	1.884	48.18	560
		3	4.15	1.50	1.2544	12.044	5.1623	2.9073	0.6969	0.4519	3.20630	0.0211	2.198	38.74	310
		4	4.15	1.70	1.2544	12.044	5.2453	3.0251	0.6775	0.4291	3.38350	0.0226	2.021	43.44	431
		5	4.15	1.80	1.2544	12.044	5.2388	3.0724	0.6548	0.4089	3.46330	0.0233	1.941	45.70	500
		6	4.15	2.28	1.2544	12.044	5.0578	3.2531	0.5270	0.2916	3.82600	0.0268	1.578	58.68	990
	315	0	4.15	1.95	1.2544	12.044	4.6102	2.8942	0.5215	0.3063	3.51070	0.0254	1.894	35.59	473
		1	4.15	1.95	1.2544	12.044	4.3182	2.7224	0.4216	0.2385	3.44290	0.026	1.962	27.91	376
		2	4.15	1.88	1.2544	12.044	4.4528	2.7793	0.4791	0.2783	3.41270	0.025	1.992	30.53	376
		3	4.15	1.50	1.2544	12.044	4.4493	2.6541	0.5622	0.3402	3.09460	0.0215	2.310	27.85	220
		4	4.15	1.70	1.2544	12.044	4.4921	2.7481	0.5325	0.3172	3.27160	0.0232	2.133	30.12	301
		5	4.15	1.80	1.2544	12.044	4.4862	2.7795	0.5112	0.3002	3.35460	0.0241	2.050	30.89	346
		6	4.15	2.28	1.2544	12.044	4.3384	2.8521	0.3755	0.2044	3.73880	0.0292	1.666	33.68	651

							Input from SWAN				Cordova 2016 Physical Model Test				
Section	Wind Direction	Combination #	Height wall MSL +m	Elevation MSL +m	BotLev m	Tp s	lax SigH	MeanSigH m	Max Zeta Max Zeta	'eta Mea m	hs m	h* -	Rc m	Q0 m3/s	q Overtop l/s/m
4	45	0	4.09	1.95	1.0379	12.044	3.7810	3.0325	0.3497	0.1919	3.17980	0.0199	1.948	61.02	428
		1	4.09	1.95	1.0379	12.044	3.6465	2.8940	0.2988	0.1436	3.13150	0.0202	1.996	51.01	361
		2	4.09	1.88	1.0379	12.044	3.6435	2.9110	0.3205	0.1741	3.09200	0.0196	2.036	52.88	345
		3	4.09	1.50	1.0379	12.044	3.5608	2.7609	0.3942	0.2282	2.76610	0.0165	2.362	49.56	195
		4	4.09	1.70	1.0379	12.044	3.6325	2.8625	0.3645	0.2064	2.94430	0.0181	2.184	52.26	270
		5	4.09	1.80	1.0379	12.044	3.6414	2.9009	0.3477	0.1948	3.03270	0.0189	2.095	53.21	314
		6	4.09	2.28	1.0379	12.044	3.7452	2.9941	0.2489	0.1093	3.42720	0.0234	1.701	56.27	612
	0	0	4.09	1.95	1.0379	12.044	4.1564	3.2502	0.4820	0.3437	3.33160	0.0204	1.796	79.09	624
		1	4.09	1.95	1.0379	12.044	4.0953	3.1733	0.4349	0.2836	3.27150	0.0201	1.856	72.21	**541**
		2	4.09	1.88	1.0379	12.044	4.1279	3.1902	0.4552	0.3225	3.24040	0.0196	1.888	74.18	522
		3	4.09	1.50	1.0379	12.044	3.8636	2.9660	0.5670	0.3827	2.92060	0.0171	2.207	61.12	281
		4	4.09	1.70	1.0379	12.044	4.0110	3.0870	0.5017	0.3617	3.09960	0.0186	2.028	67.11	395
		5	4.09	1.80	1.0379	12.044	4.0478	3.1318	0.4797	0.3441	3.18200	0.0193	1.946	69.60	460
		6	4.09	2.28	1.0379	12.044	4.1801	3.3230	0.3803	0.2396	3.55750	0.0227	1.570	87.18	944
	315	0	4.09	1.95	1.0379	12.044	4.0676	3.1075	0.3934	0.2744	3.26230	0.0204	1.866	66.36	510
		1	4.09	1.95	1.0379	12.044	3.8439	2.9494	0.3228	0.2127	3.20060	0.0207	1.927	54.19	417
		2	4.09	1.88	1.0379	12.044	3.9334	2.9955	0.3604	0.2491	3.16700	0.02	1.961	58.21	409
		3	4.09	1.50	1.0379	12.044	3.8258	2.8381	0.4507	0.3140	2.85190	0.0171	2.276	52.82	233
		4	4.09	1.70	1.0379	12.044	3.9265	2.9529	0.4108	0.2882	3.02610	0.0185	2.102	57.38	323
		5	4.09	1.80	1.0379	12.044	3.9486	2.9887	0.3905	0.2711	3.10900	0.0193	2.019	58.60	374
		6	4.09	2.28	1.0379	12.044	3.9457	3.0819	0.2817	0.1803	3.49820	0.0237	1.630	63.56	730
Section	Wind Direction	Combination #	Height wall MSL +m	Elevation MSL +m	BotLev m	Tp s	lax SigH	MeanSigH m	Max Zeta Max Zeta	'eta Mea m	hs m	h* -	Rc m	Q0 m3/s	q Overtop l/s/m
5	45	0	3.96	1.95	1.1765	12.044	4.2068	3.2104	0.2376	0.0842	3.21070	0.0191	1.926	75.86	501
		1	3.96	1.95	1.1765	12.044	4.0909	3.0536	0.1878	0.0563	3.18280	0.0198	1.954	62.17	432
		2	3.96	1.88	1.1765	12.044	4.0682	3.0842	0.2191	0.0748	3.13130	0.019	2.005	65.65	409
		3	3.96	1.50	1.1765	12.044	3.9445	2.9674	0.2609	0.1082	2.78470	0.0156	2.352	65.38	231
		4	3.96	1.70	1.1765	12.044	4.0473	3.0568	0.2430	0.0941	2.97060	0.0172	2.166	67.07	319
		5	3.96	1.80	1.1765	12.044	4.0782	3.0917	0.2397	0.0868	3.06330	0.0181	2.073	67.76	372
		6	3.96	2.28	1.1765	12.044	4.1100	3.1269	0.1468	0.0358	3.49230	0.0233	1.644	66.70	737
	0	0	3.96	1.95	1.1765	12.044	4.2551	3.5173	0.3717	0.2110	3.33750	0.0189	1.799	108.21	737
		1	3.96	1.95	1.1765	12.044	4.1394	3.3937	0.3142	0.1652	3.29170	0.019	1.845	93.93	**636**
		2	3.96	1.88	1.1765	12.044	4.2016	3.4471	0.3558	0.1765	3.23300	0.0181	1.904	101.10	601
		3	3.96	1.50	1.1765	12.044	3.9784	3.2750	0.4178	0.2463	2.92280	0.0155	2.214	90.94	344
		4	3.96	1.70	1.1765	12.044	4.0903	3.3633	0.3917	0.2260	3.10250	0.0171	2.034	94.46	471
		5	3.96	1.80	1.1765	12.044	4.1608	3.4067	0.3742	0.2112	3.18770	0.0178	1.949	97.26	548
		6	3.96	2.28	1.1765	12.044	4.3583	3.5265	0.2377	0.1319	3.58840	0.0218	1.548	110.55	1115
	315	0	3.96	1.95	1.1765	12.044	4.3344	3.5758	0.3875	0.2226	3.34910	0.0187	1.787	115.67	776
		1	3.96	1.95	1.1765	12.044	4.1087	3.4028	0.3169	0.1697	3.29620	0.019	1.840	94.91	644
		2	3.96	1.88	1.1765	12.044	4.2205	3.4693	0.3603	0.2018	3.25830	0.0182	1.878	103.32	633
		3	3.96	1.50	1.1765	12.044	4.0257	3.2841	0.4396	0.2636	2.94010	0.0157	2.196	91.23	355
		4	3.96	1.70	1.1765	12.044	4.1390	3.4007	0.4082	0.2390	3.11550	0.017	2.021	98.45	491
		5	3.96	1.80	1.1765	12.044	4.1758	3.4482	0.3860	0.2221	3.19860	0.0177	1.938	101.96	572
		6	3.96	2.28	1.1765	12.044	4.1731	3.5264	0.2359	0.1357	3.59220	0.0218	1.544	110.62	1122

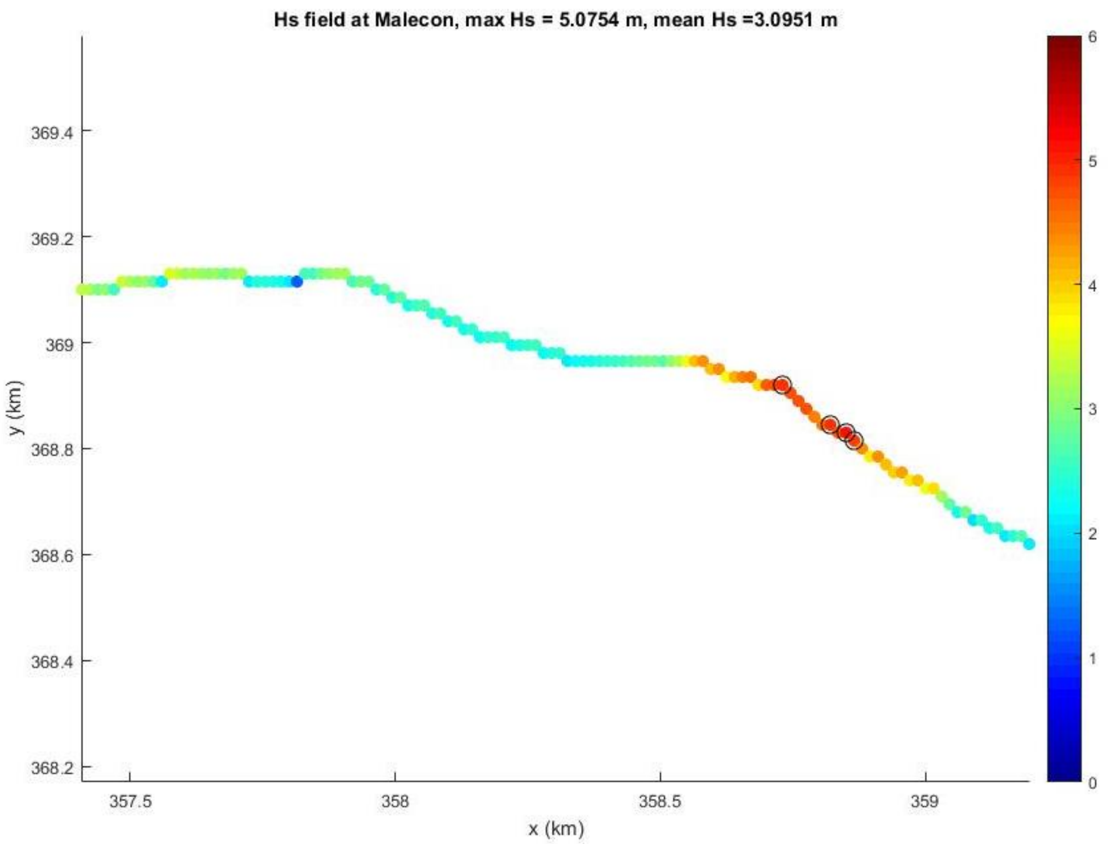
Appendix N. Significant wave height and wave set up per section

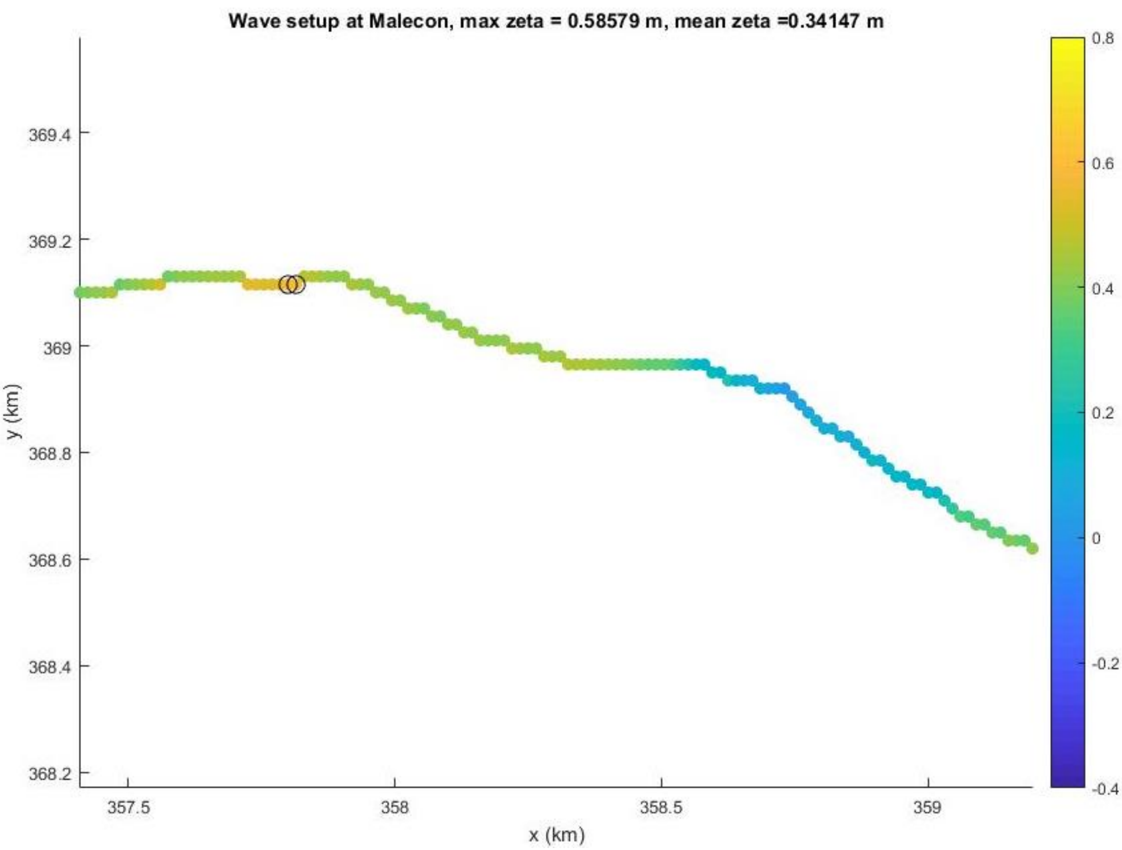
Section 2



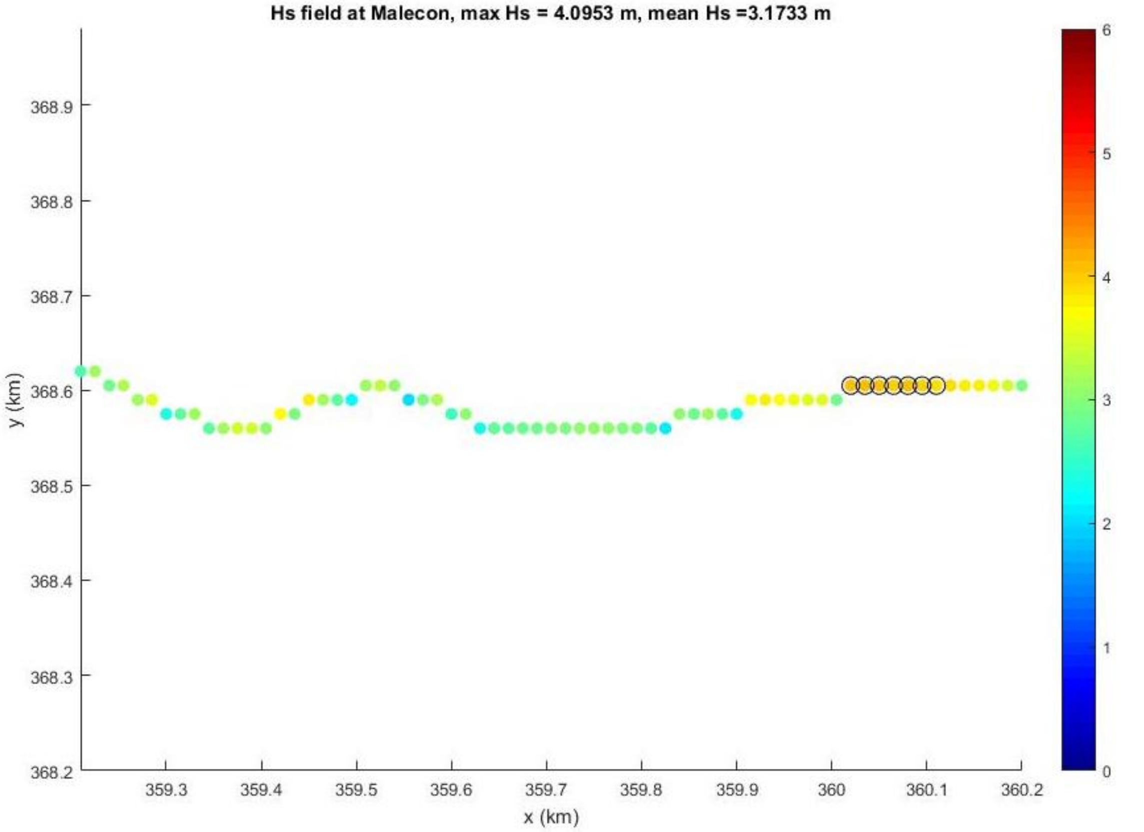


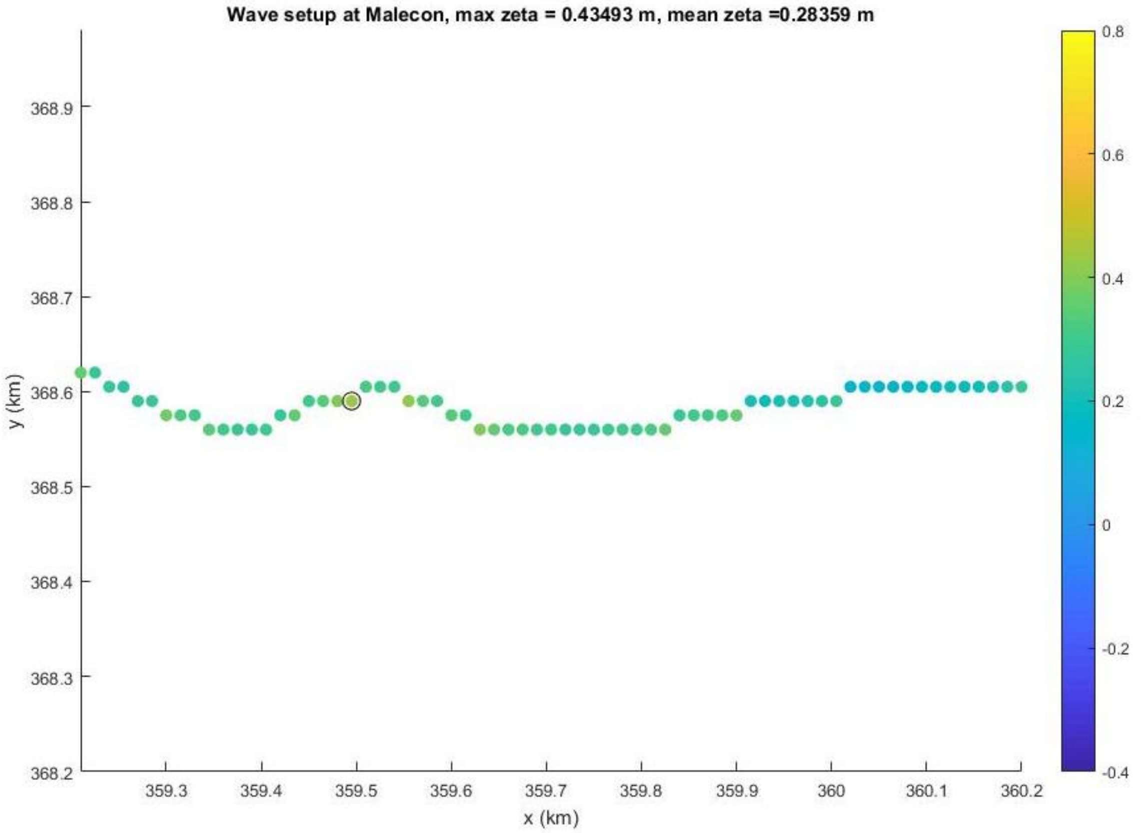
Section 3


Hs field at Malecon, max Hs = 5.0754 m, mean Hs =3.0951 m
y (km)
x (km)
369.4
369.2
369
368.8
368.6
368.4
368.2
357.5
358
358.5
359
0
1
2
3
4
5
6

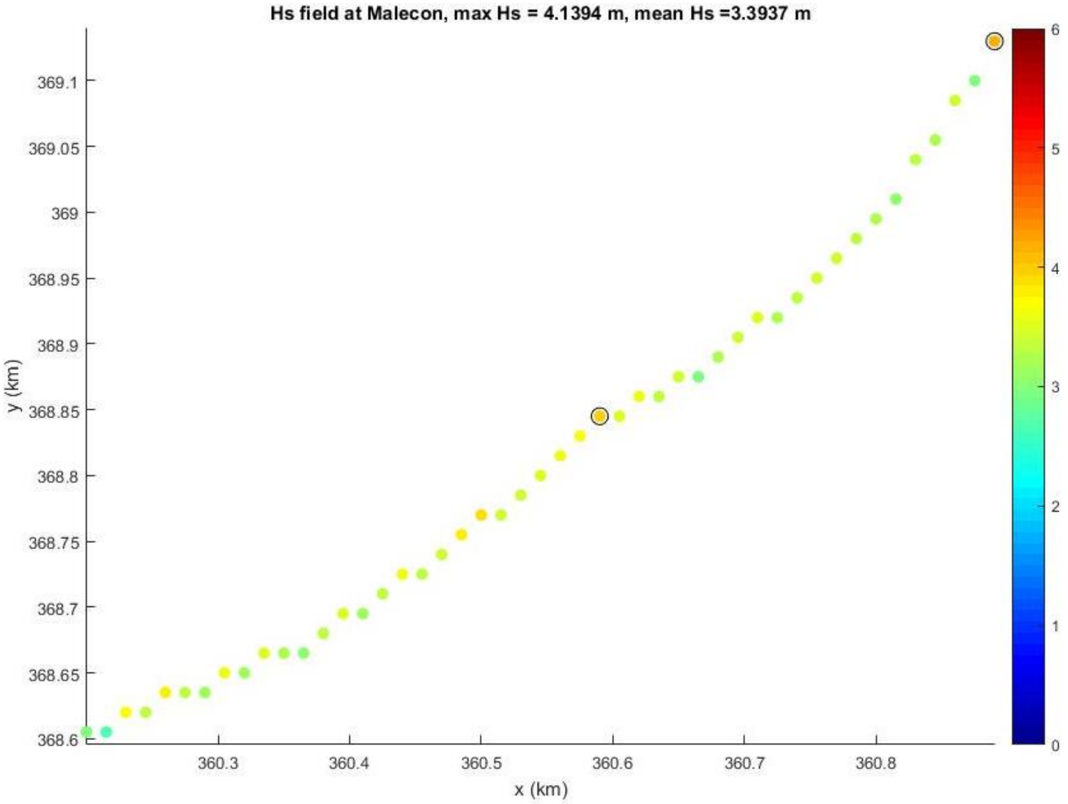

Wave setup at Malecon, max zeta = 0.58579 m, mean zeta =0.34147 m
y (km)
x (km)
369.4
369.2
369
368.8
368.6
368.4
368.2
357.5
358
358.5
359
0.8
0.6
0.4
0.2
0
-0.2
-0.4

Section 4

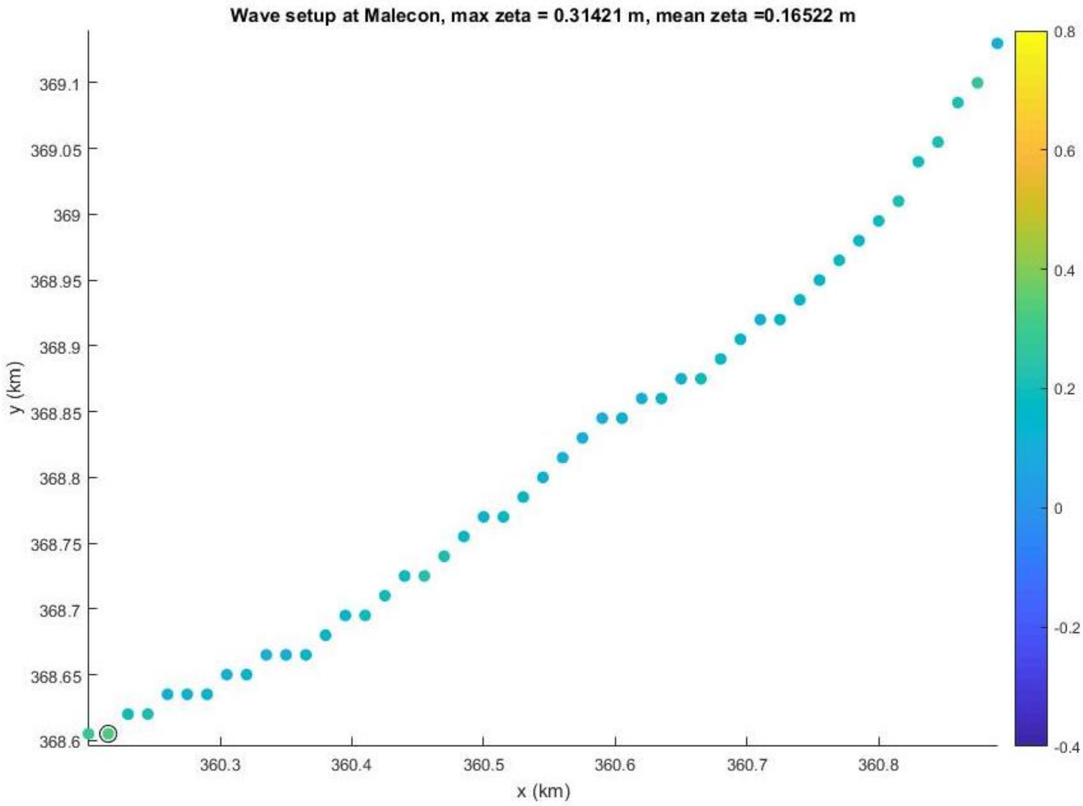

Hs field at Malecon, max Hs = 4.0953 m, mean Hs =3.1733 m
y (km)
x (km)


Wave setup at Malecon, max zeta = 0.43493 m, mean zeta =0.28359 m
y (km)
x (km)

Section 5



Hs field at Malecon, max Hs = 4.1394 m, mean Hs =3.3937 m
y (km)
x (km)

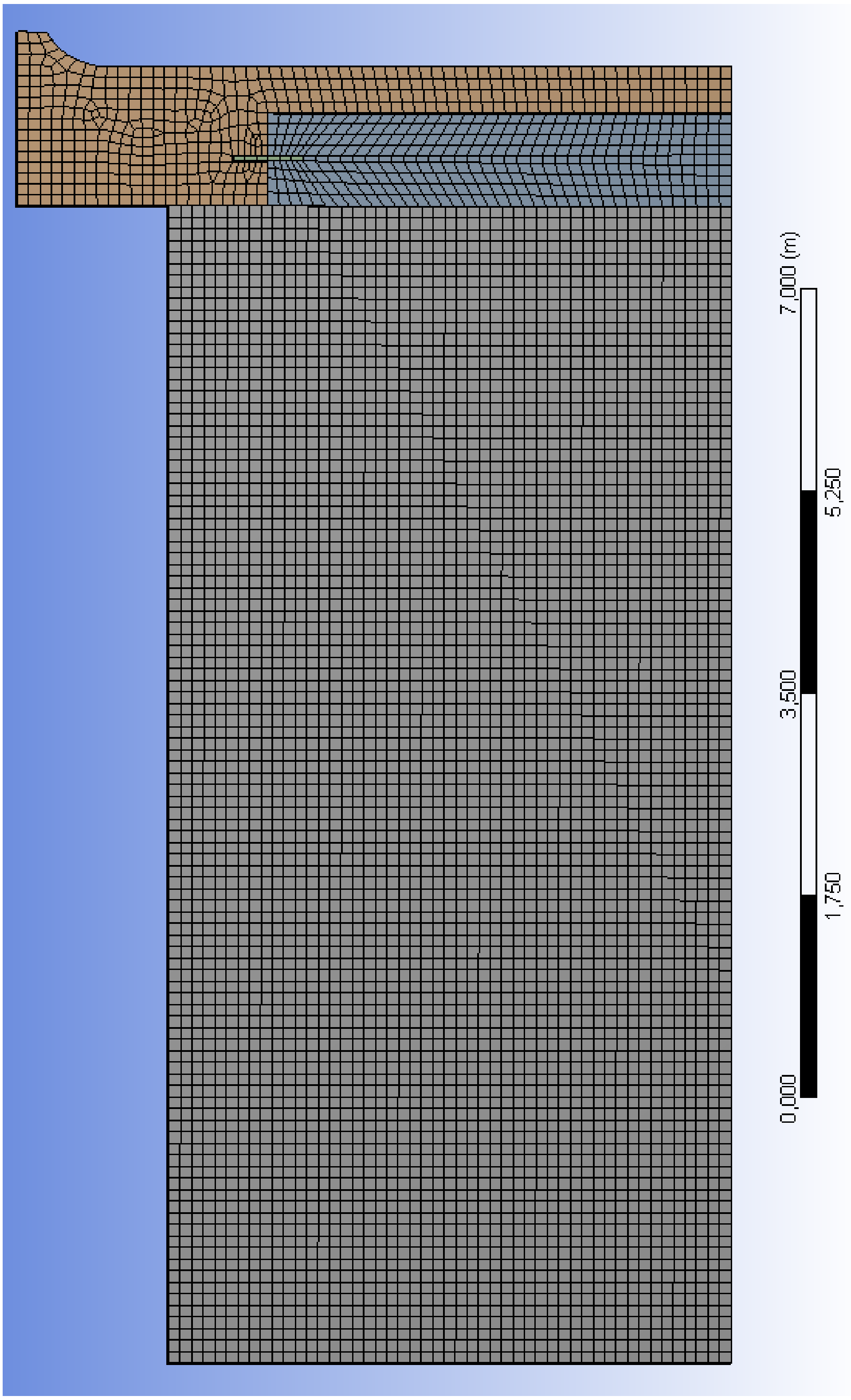


Wave setup at Malecon, max zeta = 0.31421 m, mean zeta =0.16522 m
y (km)
x (km)

Appendix O. Results of SWAN modelling and wave overtopping calculation

Section	SubSection	Height wall MSL +m	MeanSigH m	Zeta Mean m	Elevation m	Tp s	BotLev m	hs m	h* -	Rc m	H*0	Rc0	Q00	Q0 m3/s	q Overtop m3/s/m	q Overtop l/s/m
2	1	3,10	2,8100	0,33510	1,95	12,0435	1,2340	3,5191	0,026	0,815	0,026	0,815	154,771	154,771	2,209	1944
	2	3,35	3,3190	0,33510	1,95	12,0435	1,2340	3,5191	0,022	1,065	0,022	1,065	176,972	176,972	1,810	1593
	3	3,60	4,2300	0,33510	1,95	12,0435	1,2340	3,5191	0,017	1,315	0,017	1,315	310,472	310,472	1,955	1721
	4	3,60	4,1090	0,33510	1,95	12,0435	1,2340	3,5191	0,018	1,315	0,018	1,315	275,642	275,642	1,840	1619
	5	3,75	3,5940	0,33510	1,95	12,0435	1,2340	3,5191	0,021	1,465	0,021	1,465	144,957	144,957	1,265	1265
	6	3,80	3,1240	0,33510	1,95	12,0435	1,2340	3,5191	0,024	1,515	0,024	1,515	67,033	67,033	0,774	681
	7	4,10	3,8710	0,33510	1,95	12,0435	1,2340	3,5191	0,019	1,815	0,019	1,815	126,678	126,678	0,953	953
	8	4,43	3,5027	0,33510	1,95	12,0435	1,2340	3,5191	0,021	2,145	0,021	2,145	52,532	52,532	0,483	425
3	1	4,30	3,2130	0,34150	1,95	12,0435	1,2544	3,5459	0,023	2,009	0,023	2,009	40,897	40,897	0,465	410
	2	4,30	2,3630	0,34150	1,95	12,0435	1,2544	3,5459	0,032	2,009	0,032	2,009	11,597	11,603	0,244	215
	3	4,30	2,3170	0,34150	1,95	12,0435	1,2544	3,5459	0,032	2,009	0,032	2,009	10,699	10,704	0,234	206
	4	4,30	2,5280	0,34150	1,95	12,0435	1,2544	3,5459	0,030	2,009	0,030	2,009	15,294	15,302	0,281	248
	5	4,00	4,4280	0,34150	1,95	12,0435	1,2544	3,5459	0,017	1,709	0,017	1,709	241,044	241,188	1,445	1445
	6	4,00	3,9880	0,34150	1,95	12,0435	1,2544	3,5459	0,019	1,709	0,019	1,709	156,943	157,037	1,160	1160
	7	4,00	2,5200	0,34150	1,95	12,0435	1,2544	3,5459	0,030	1,709	0,030	1,709	23,899	23,914	0,442	442
	8	4,00	2,3190	0,34150	1,95	12,0435	1,2544	3,5459	0,032	1,709	0,032	1,709	16,997	17,007	0,372	372
4	1	4,16	2,8560	0,28360	1,95	12,0435	1,0379	3,2715	0,022	1,926	0,022	1,926	38,259	38,242	0,354	311
	2	4,33	3,2870	0,28360	1,95	12,0435	1,0379	3,2715	0,019	2,096	0,019	2,096	57,239	57,217	0,400	352
	3	3,97	2,9840	0,28360	1,95	12,0435	1,0379	3,2715	0,021	1,736	0,021	1,736	56,658	56,631	0,480	422
	4	3,97	3,4990	0,28360	1,95	12,0435	1,0379	3,2715	0,018	1,736	0,018	1,736	108,832	108,781	0,670	590
	5	3,97	3,9090	0,28360	1,95	12,0435	1,0379	3,2715	0,016	1,736	0,016	1,736	171,418	171,337	0,846	744
5	1	3,97	3,1400	0,16520	1,95	12,0435	1,1765	3,2917	0,021	1,855	0,021	1,855	59,431	59,444	0,471	414
	2	3,97	3,3280	0,16520	1,95	12,0435	1,1765	3,2917	0,019	1,855	0,019	1,855	75,431	75,448	0,532	468
	3	3,94	3,2230	0,16520	1,95	12,0435	1,1765	3,2917	0,020	1,825	0,020	1,825	68,388	68,404	0,514	452
	4	3,94	3,3790	0,16520	1,95	12,0435	1,1765	3,2917	0,019	1,825	0,019	1,825	83,013	83,032	0,568	499

Appendix P. ANSYS model mesh



Appendix R. ANSYS model overview and schematization

Dimensions of the structure. The first thing that must be established for the structural analysis is the geometry of the structure, this is based primarily on the analysis of the current Malecón structure made in the report by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) and the proposed design solution by (L.F. Córdova et al, 2016). In the former it was stated that the structure consisted of two main parts; firstly the current wall and secondly the area behind it, referred to as the concrete body. The concrete body is the result of the construction method, dating from the early 20th century, whereby the shore was blocked off by caissons and the area inland was subsequently filled with a mixture of concrete and rocks. This area will continue to be modelled as a weak concrete, with a modulus of elasticity of 25 GPa, as was done by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) given its effectiveness. The construction method for the proposed solution defines the dimensions of the old wall, which will have the new wall mounted onto it using dowels and an epoxy grout fill to withstand tensile forces. In order to mount the new wall structure it was recommended by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) to cut the old wall at 2.3m from mean sea level (MSL) as the tensile stresses are lowest. An ANSYS model was used to verify this assumption, the results of which are shown in Figure 58.

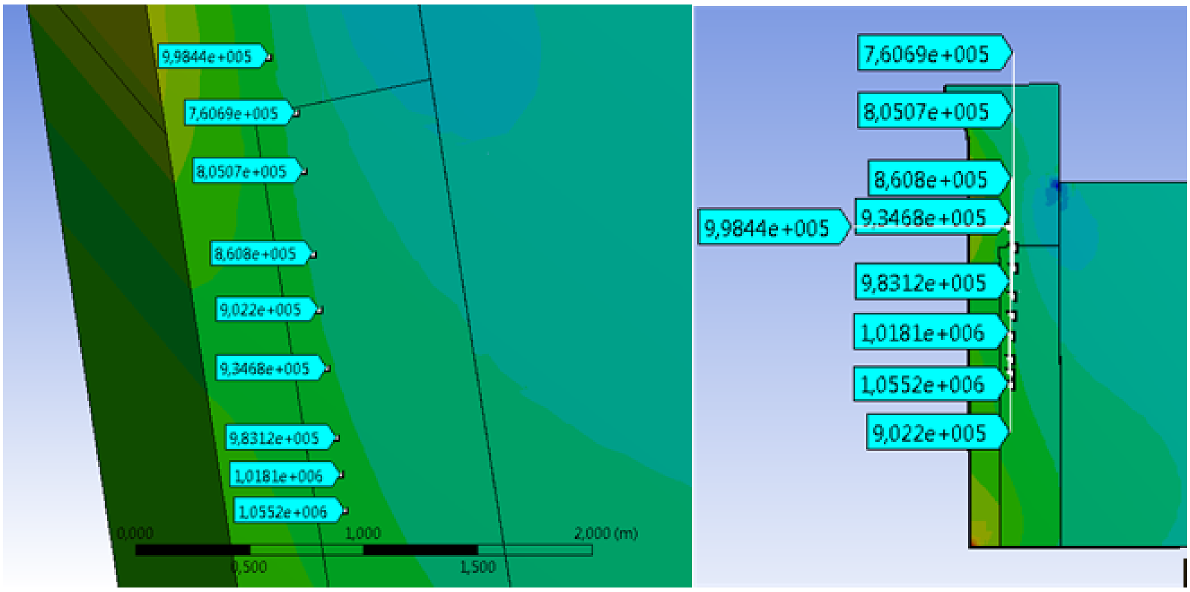


Figure 58: Verification of tensile stresses for cut at 2.3m

This assumption is shown to be valid hence the old wall will be modelled as rectangular section of 4m tall (MSL at 1.7m + 2.3m for cut) and 0.8m wide with a modulus of elasticity of 30 GPa, given its age and significant exposure to weathering. The dimensions of the new wall are governed by the necessary cover at the bottom of the new wall, on the seaward facing side of the old wall, and the dimensions of the recurve as designed by (L.F. Córdova et al, 2016). These can be seen in Figure 59. The recommended concrete to be used according to (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) is C35/45 for its strength and environmental resistance, this gives an E-modulus of 34 GPa. The final component in the model, the dowels, will be indicated without dimensions as these have not been designed in previous investigations and the dimensions depend on the loads in the structure which will be discussed previously. They will be modelled using standard structural steel with an E-modulus of 210 GPa. A complete overview of the geometry is shown below.

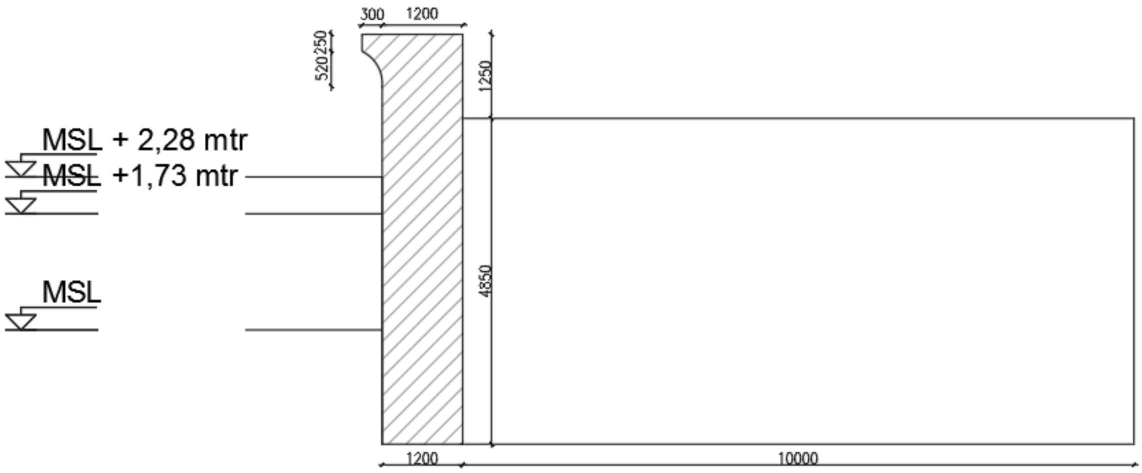


Figure 59: Dimensions of the structure to be modelled

Boundary conditions. The next aspect of the model to be defined are the boundary conditions, these determine how the model will represent the environment in which the structure is placed. While a two dimensional analysis is most favourable for the desired modelling procedure, allowing for simpler and faster modelling runs, ANSYS generates a 3 dimensional model by default. In order to approximate a two dimensional analysis the sides of the structure will be constrained by frictionless supports. The concrete body will be constrained by a fixed support at the back and by a very rigid elastic support at the bottom due to its large weight; the bottom of the old wall will be modelled in the same way. While (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) modelled both bases (bottom supports) of these elements as fixed supports this led to very high tensile stresses in the foot of both. This would mean the connection with the floor can resist very high tensile stresses according to the model which is unlikely to be the case as this connection can only be realised with cementitious materials which are generally weak in tension. Modelling the bases of these elements as fixed supports leads to the stress concentrations as shown in Figure 60.

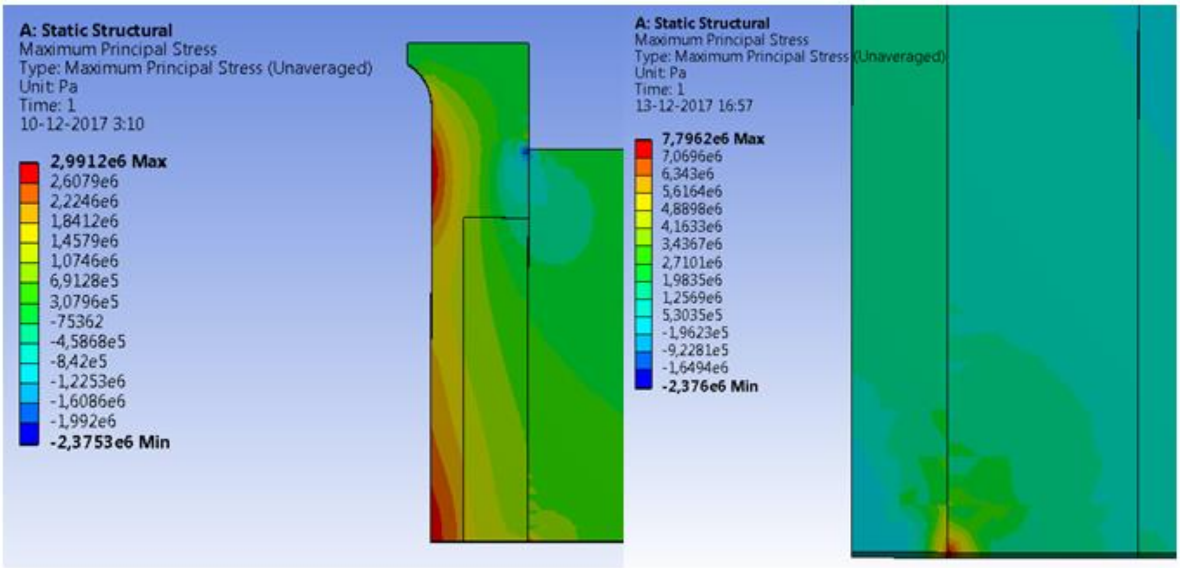


Figure 60: Stress concentrations at the foot due to modelling of the base as a fixed support (three part model)

In order to address this issue the model was expanded to include the dowels, this allowed the bases of the elements to be modelled as weak springs rather than fixed supports as the dowels now provide stability of the new wall. Moreover a more accurate analysis of the dowels and their influence on the structure can be obtained using this four part model (concrete body, old wall, new wall, and dowels). As an initial for modelling the dowels were taken with a diameter of 30mm and a length of 600mm, extending 300mm into both the old and new walls in order to join them. Thus the bases of the old wall and concrete body are modelled as a very stiff elastic foundation and the new wall with a weak elastic foundation to model the weak bond with the natural berm. This eliminated the unrealistic tensile stresses in the foot in each of the elements which can be seen in the results of the ANSYS model in the chapter 1.2.

The mesh is also a key subject in performing a structural analysis using a finite element program such as ANSYS. As discussed by (Hoogenboom, 2012) quadrilateral elements provide more accuracy than triangular elements however given the irregular geometry of the structure this is not always possible. The mesh used consists primarily of quadrilateral elements and was refined until successive runs produced results within 5% of one another as discussed by (la Gasse, van Rooij, Smits, Ton, & Velhorst, Coastal protection Malecón seawall, 2015) and in the book Finite Element Modelling for Stress Analysis (Cook, 1995). The mesh was set to 0.1m and can be found in

Loads on the structure

The loads on the structure can be summarized as follows: gravity acting downward on all elements, hydrostatic water pressure, pressure on the recurve, and the pressure profile on the vertical section of the wall both as discussed in the chapter 'Forcing on the Malecón'. Hydrostatic water pressure will be calculated for the mean sea level and storm surge during Hurricane Wilma, 1.7m + 2.28m, as these are the governing conditions for ultimate limit state (ULS). An overview of the loads is given in Figure 61. It should be noted that the pressures are not to scale and that the magnitude of the pressure on the curved segment is the same as that at the top of the vertical segment. The letter 'g' denotes the weight of the various parts of the structure.

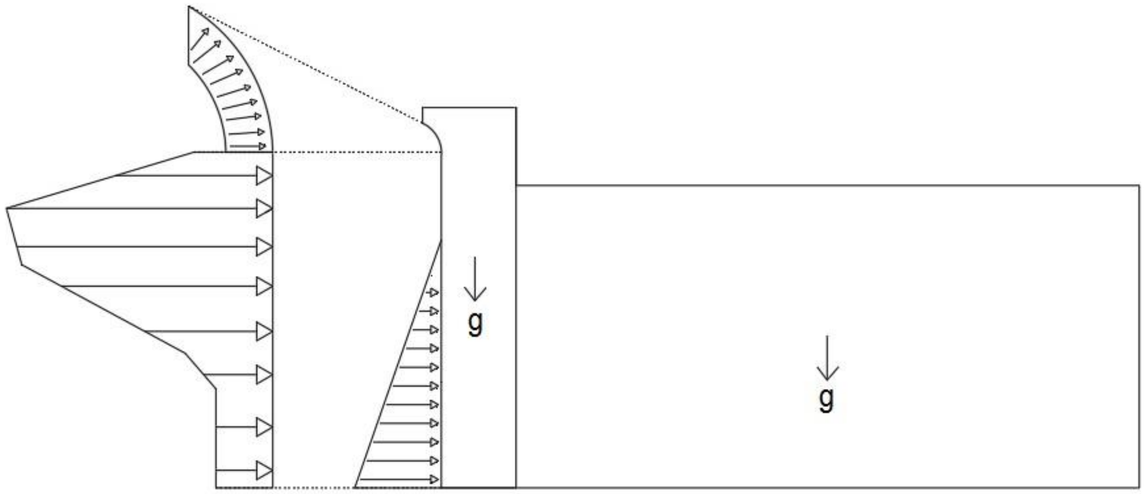


Figure 61: Overview of the loads on the structure

Appendix S. Multi Criteria Analysis measures

					Curved wall		
				Weight	Prefab	In situ construction	Combination
Criteria	Safety	25%	Wave Overtopping	18%	3	3	3
			Structural Integrity	8%	3	4	5
	Cost	25%	Direct cost	20%	5	3	4
			Maintenance costs	5%	4	3	3
	Society and environmental	25%	Social attractiveness	10%	2	2	2
			Characteristic view	10%	3	3	3
			Effect on environment	5%	5	5	5
	Implementation	15%	On site limitations	8%	2	3	4
			Availability of materials and equipment in Cuba	8%	4	4	4
	Construction	10%	Construction Time	5%	2	4	3
			Reduced protection during construction	5%	2	1	2
	Total	100%		100%	3,35	3,1	3,45

					Berm	
				Weight	Permeable berm	Impermeable berm
Criteria	Safety	25%	Wave Overtopping	18%	3	2
			Structural Integrity	8%	3	3
	Cost	25%	Direct cost	20%	2	2
			Maintenance costs	5%	2	4
	Society and environmental	25%	Social attractiveness	10%	3	4
			Characteristic view	10%	3	1
			Effect on environment	5%	3	2
	Implementation	15%	On site limitations	8%	2	3
			Availability of materials and equipment in Cuba	8%	4	4
	Construction	10%	Construction Time	5%	4	3
			Reduced protection during construction	5%	5	5
	Total	100%		100%	2,9	2,7

					Breakwater	
				Weight	Emerged Breakwater	Low crested Breakwater
Criteria	Safety	25%	Wave Overtopping	18%	3	2
			Structural Integrity	8%	3	2
	Cost	25%	Direct cost	20%	3	1
			Maintenance costs	5%	3	3
	Society and environmental	25%	Social attractiveness	10%	3	3
			Characteristic view	10%	1	4
			Effect on environment	5%	2	3
	Implementation	15%	On site limitations	8%	3	2
			Availability of materials and equipment in Cuba	8%	3	3
	Construction	10%	Construction Time	5%	2	1
			Reduced protection during construction	5%	5	5
	Total	100%		100%	2,8	2,375

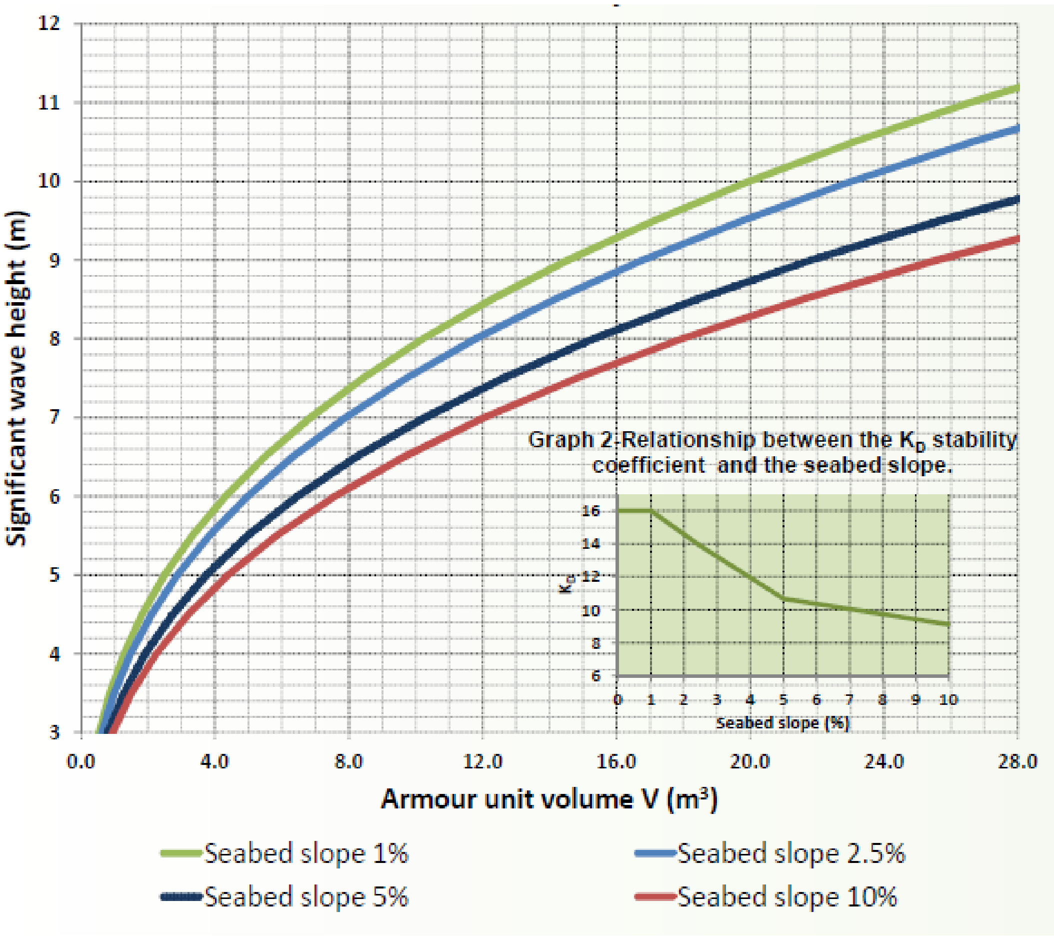
Appendix W. Design data berm and breakwater elements

Design data for ACCROPODE AND ECOPODE (Concrete Layer Innovations, 2012)

The ECOPODE™ unit size is limited to 10m³

Unit Volume (m³)	V = 0.2926H³		1.0	2.0	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	28.0
Unit Height (m)	$H = (V/0.2926)^{(1/3)}$		1.51	1.90	2.17	2.39	2.58	2.74	3.01	3.25	3.45	3.63	3.80	3.95	4.09	4.22	4.34	4.57
Equivalent Cube Size (m)	$Dn = V^{1/3}$		1.00	1.26	1.44	1.59	1.71	1.82	2.00	2.15	2.29	2.41	2.52	2.62	2.71	2.80	2.88	3.04
Armour Thickness (m)	T = 1.36 Dn		1.36	1.71	1.96	2.16	2.33	2.47	2.72	2.93	3.11	3.28	3.43	3.56	3.69	3.81	3.92	4.13
Armour concrete consumption and coverage	Packing density Φ (-)		0.635	0.635	0.635	0.633	0.631	0.629	0.625	0.622	0.618	0.614	0.610	0.610	0.610	0.610	0.610	0.610
	Consumption (m³/m²)		0.635	0.800	0.916	1.005	1.079	1.143	1.251	1.339	1.414	1.479	1.537	1.599	1.656	1.709	1.760	1.852
	Number of units (u/m²)		0.635	0.400	0.305	0.251	0.216	0.191	0.156	0.134	0.118	0.106	0.096	0.089	0.083	0.078	0.073	0.066
	Porosity (%)		53.31	53.31	53.31	53.45	53.59	53.73	54.02	54.30	54.58	54.86	55.15	55.15	55.15	55.15	55.15	55.15
Filter stone underlayer - to meet the following requirement NUL/NLL < 3.0	**NLL (tons)**	**Standard**	0.17	0.34	0.50	0.67	0.84	1.01	1.34	1.68	2.02	2.35	2.69	3.02	3.36	3.70	4.03	4.70
		Min/Max*	0.1 0.2	0.2 0.4	0.4 0.7	0.5 0.9	0.6 1.1	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.6 3.1	1.9 3.5	2.1 3.9	2.4 4.4	2.6 4.8	2.8 5.2	3.3 6.1
	NUL (tons)	**Standard**	0.34	0.67	1.01	1.34	1.68	2.02	2.69	3.36	4.03	4.70	5.38	6.05	6.72	7.39	8.06	9.41
		Min/Max*	0.2 0.4	0.5 0.9	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.9 3.5	2.4 4.4	2.8 5.2	3.3 6.1	3.8 7.0	4.2 7.9	4.7 8.7	5.2 9.6	5.6 10.5	6.6 12.2
	Thickness (m) for standard NLL&NUL Specific density 2,6 t/m3	**Kt=1,15**	1.06	1.33	1.52	1.68	1.81	1.92	2.11	2.28	2.42	2.55	2.66	2.77	2.87	2.96	3.05	3.21
		Kt=0.9*	0.83	1.04	1.19	1.31	1.41	1.50	1.65	1.78	1.89	1.99	2.08	2.17	2.24	2.32	2.38	2.51

(Concrete Layer Innovations, 2012)

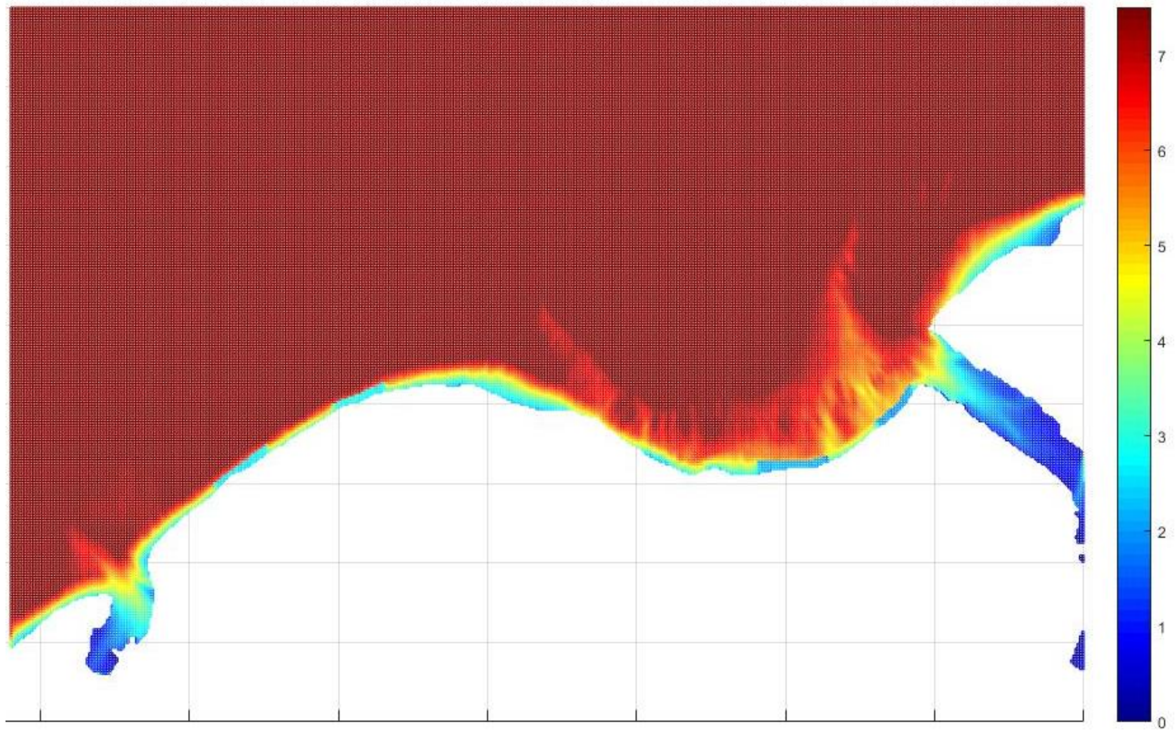


Mean Sigh and bottom data per section, stability calculations and resulting unit sizes.

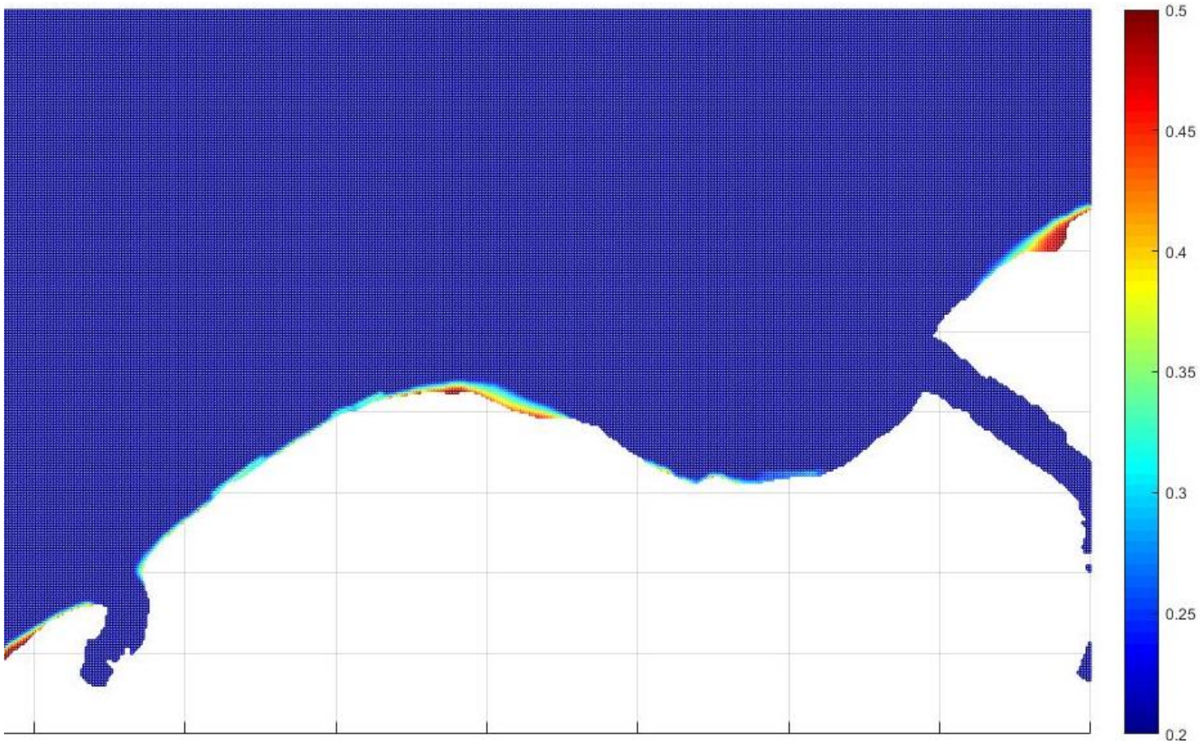
SubSection	Mean SigH at breakwater	MeanSigH at wall	Option	Bottom slope	Unit type	V from graph	V minimum	Dn	Stability	V for design	Stability
	m	m		(%)		m^3	m^3	m	-		-
1		2,8100	3	8,30%	Eco	1,2	2	1,259921	2,63	3	2,30
2		**3,3190**	3	8,30%	Eco	1,2	2	1,259921	2,63	3	2,30
3	7,2		4	8,30%	Accro	12,4	14	2,410142	2,99	24	2,50
4	7,2		4	8,30%	Accro	12,4	14	2,410142	2,99	24	2,50
5		3,5000	3	8,30%	Accro	1,5	2	1,259921	2,78	3	2,43
6		**3,1240**	3	8,30%	Eco	1,1	2	1,259921	2,48	-	-
7	7		4	11,20%	Accro	12,8	14	2,410142	2,90	22	2,50
8	7		4	11,20%	Accro	12,8	14	2,410142	2,90	22	2,50
1		**3,2130**	3	11,20%	Eco	1,6	2	1,259921	2,55	3	2,23
2		2,3630	2	-	-					-	-
3		2,3170	2	-	-					-	-
4		2,5280	2	-	-					-	-
5		2,5200	3	13,00%	Eco	2,5	3	1,44225	3,07	6	2,44
6		**4,4280**	3	13,00%	Eco	2,6	3	1,44225	3,07	6	2,44
7		3,9880	3	13,00%	Eco	2,6	3	1,44225	3,07	6	2,44
8		2,3190	3	5,00%	Eco	0,8	1	1	2,86	3	1,98
1		2,8560	3	5,00%	Eco	0,8	1	1	3,29	3	2,28
2		**3,2870**	3	5,00%	Eco	0,8	1	1	3,29	3	2,28
3		2,9840	3	5,00%	Eco	0,8	1	1	3,29	3	2,28
4	5,7		4	9,20%	Accro	6,2	8	2	2,85	12	2,49
5	5,7		4	9,20%	Accro	6,2	8	2	2,85	12	2,49
1		3,1400	2	-	-					-	-
2		3,3280	2	-	-					-	-
3	5,7		4	9%	Accro	6,2	8	2	2,85	12	2,49
4	5,7		4	9%	Accro	6,2	8	2	2,85	12	2,49

Appendix X. SWAN modelling breakwaters

Significant Wave Height Havana Bay with breakwaters



Wave setup Havana Bay with breakwaters

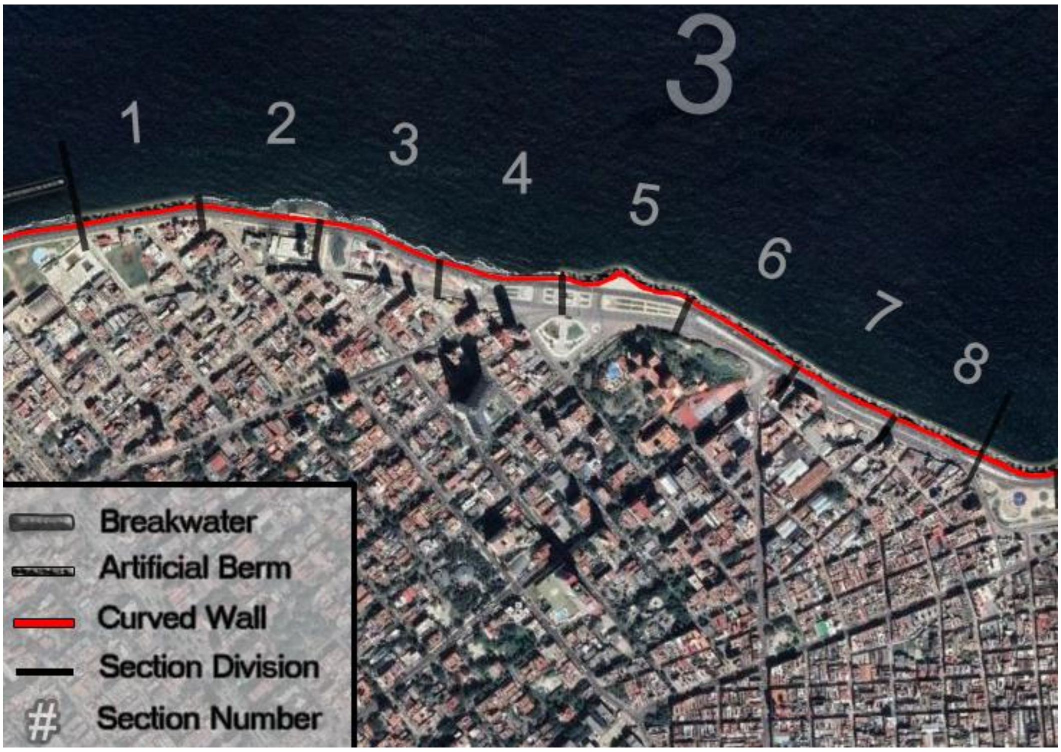


Appendix Y. Wave overtopping calculation final design per section

Section	SubSection	Height wall MSL +m	MeanSigH m	Zeta Mean m	Elevation m	Tp s	BotLev m	hs m	h* -	Rc m	q Overtop m3/s/m	q Overtop l/s/m	Option
2	1	3,10	2,8100	0,33510	1,95	12,0435	1,2340	3,5191	0,026	2,175	0,116	116	3
	2	3,35	3,3190	0,33510	1,95	12,0435	1,2340	3,5191	0,022	2,175	0,210	210	3
	3	3,60	4,2300	0,33510	1,95	12,0435	1,2340	3,1840	0,014	2,510	0,094	83	4
	4	3,60	4,1090	0,33510	1,95	12,0435	1,2340	3,1840	0,015	2,510	0,083	73	4
	5	3,75	3,5940	0,33510	1,95	12,0435	1,2340	3,5191	0,021	2,175	0,279	279	3
	6	3,80	3,1240	0,33510	1,95	12,0435	1,2340	3,5191	0,024	2,175	0,169	169	3
	7	4,10	3,8710	0,33510	1,95	12,0435	1,2340	3,1840	0,016	2,510	0,072	72	4
	8	4,43	3,5027	0,33510	1,95	12,0435	1,2340	3,1840	0,017	2,510	0,041	36	4
3	1	4,30	3,2130	0,34150	1,95	12,0435	1,2544	3,5459	0,023	2,169	0,188	188	3
	2	4,30	2,3630	0,34150	1,95	12,0435	1,2544	3,5459	0,032	2,169	0,154	135	2
	3	4,30	2,3170	0,34150	1,95	12,0435	1,2544	3,5459	0,032	2,169	0,147	129	2
	4	4,30	2,5280	0,34150	1,95	12,0435	1,2544	3,5459	0,030	2,169	0,180	158	2
	5	4,00	4,4280	0,34150	1,95	12,0435	1,2544	3,5459	0,017	2,169	0,590	590	3
	6	4,00	3,9880	0,34150	1,95	12,0435	1,2544	3,5459	0,019	2,169	0,407	407	3
	7	4,00	2,5200	0,34150	1,95	12,0435	1,2544	3,5459	0,030	2,169	0,079	79	3
	8	4,00	2,3190	0,34150	1,95	12,0435	1,2544	3,5459	0,032	2,169	0,059	59	3
4	1	4,16	2,8560	0,28360	1,95	12,0435	1,0379	3,2715	0,022	2,226	0,116	116	3
	2	4,33	3,2870	0,28360	1,95	12,0435	1,0379	3,2715	0,019	2,226	0,191	191	3
	3	3,97	2,9840	0,28360	1,95	12,0435	1,0379	3,2715	0,021	2,226	0,135	135	3
	4	3,97	3,4990	0,28360	1,95	12,0435	1,0379	2,9879	0,015	2,510	0,043	38	4
	5	3,97	3,9090	0,28360	1,95	12,0435	1,0379	2,9879	0,014	2,510	0,070	62	4
5	1	3,97	3,1400	0,16520	1,95	12,0435	1,1765	3,2917	0,021	2,345	0,230	203	2
	2	3,97	3,3280	0,16520	1,95	12,0435	1,1765	3,2917	0,019	2,345	0,264	232	2
	3	3,94	3,2230	0,16520	1,95	12,0435	1,1765	3,1265	0,018	2,510	0,029	25	4
	4	3,94	3,3790	0,16520	1,95	12,0435	1,1765	3,1265	0,017	2,510	0,035	31	4

Appendix Z Overview of measures per section.

Section 2



Section 3

Section 4

Section 5

Appendix AA. Cross sections detailed design

Cross section for sections 2.1 and 2.2 at the start Calle X until Calle X. A permeable berm is designed from concrete ECOPODE 3 m^3 units with a crest width of 20 meters and a crest height of 2.28 above MSL. Due to the relative low crest height of the wall compared to MSL berm type 1 is not applicable in these sections, therefore berm type 2. The seabed is comparable with an average slope of 8.3 %.

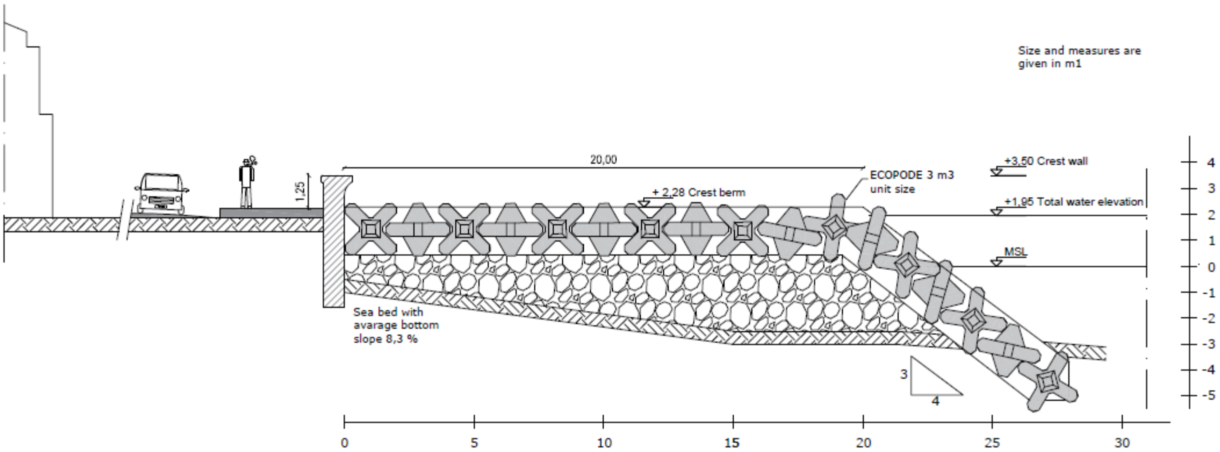


Figure 62 Detailed design for section 2.1 and 2.2

Cross section for sections 2.3, 2.4, 2.7 and 2.8 both have a breakwater of approximately 500 meters. The section is from Calle X – Calle X and Calle X – Calle X. The breakwaters are designed with concrete ACCROPODE 22 and 24 m^3 units at a crest width of 12 meters with a crest height of 3.28 above MSL. Seabed of the first breakwater has an average slope of 8.3 % and the second 11.3 %. The large elements are needed due to combination of wave height and slope. The tow of the breakwater is visual at approximately 35 meters from the wall with normal sea conditions.

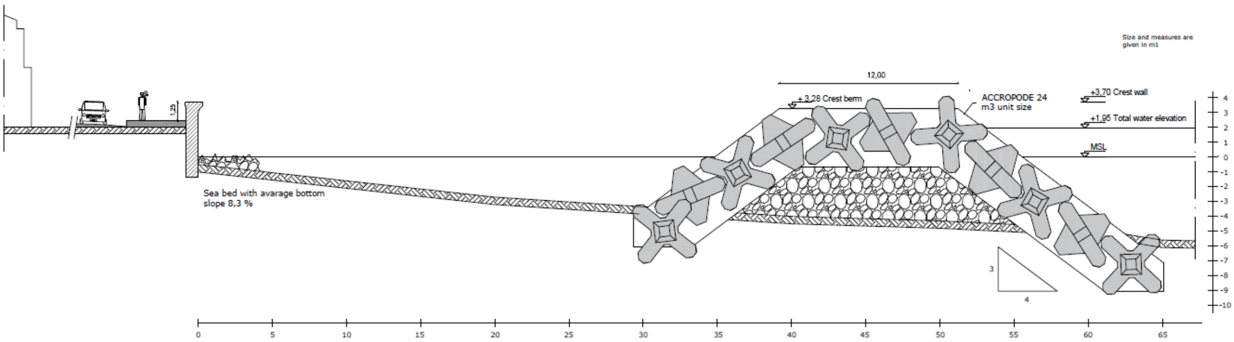


Figure 63 Detailed design for section 2.3, 2.4, 2.7 and 2.8

Cross section for sections 2.5 and 2.6 and at the start Calle X until Calle X. A permeable berm is designed from concrete ECOPODE 2 and 3 m^3 units with a crest width of 5 meters and a crest height of 3.28 above MSL. Section 2.6 requires 3 m^3 units due to higher wave conditions to reach the required stability factor. The seabed profile in both subsections are comparable with an average slope of 8.3 %.

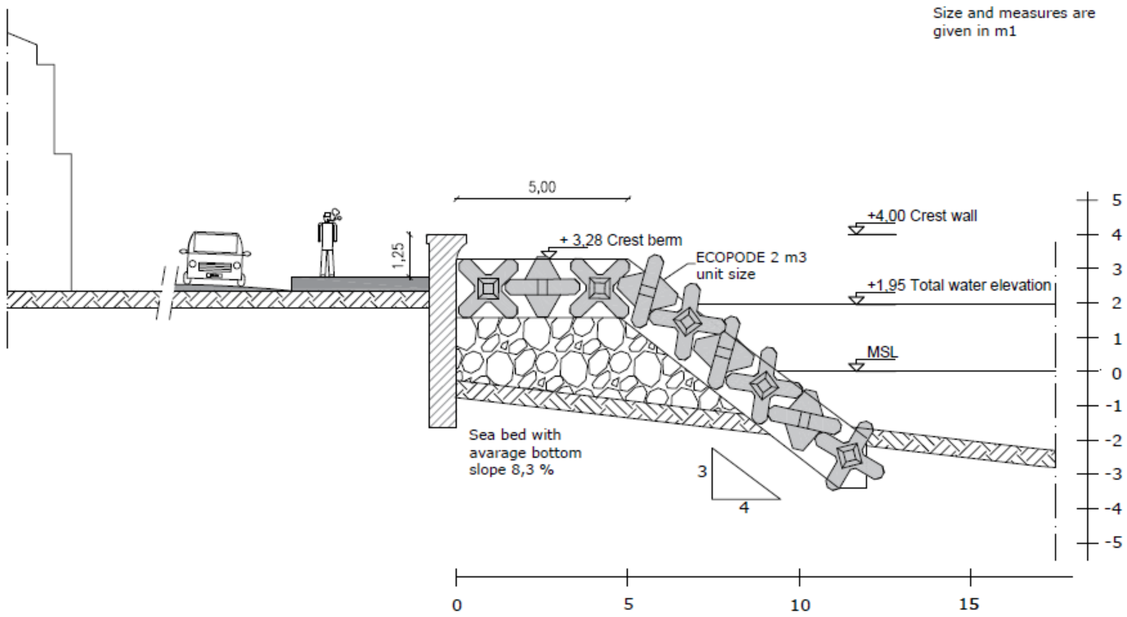


Figure 64 Detailed design for section 2.5 and 2.6

In sections 3.2, 3.3, 3.4 5.1 and 5.2 no additional measures are applied besides the curved sea wall. The relative favourable wave conditions in combination with the existing natural berm provide sufficient reduction.

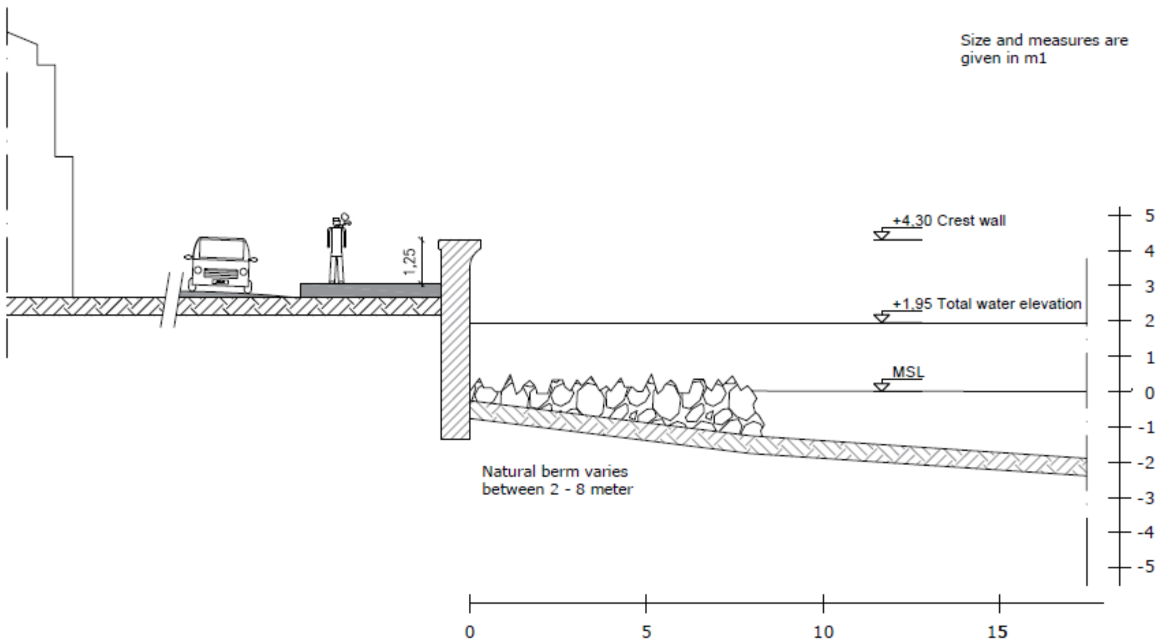


Figure 65 Detailed design for section 3.2, 3.3, 3.4, 5.1 and 5.2

In the sections 3.5, 3.6 and 3.7 from Calle X till Calle X a permeable berm is designed from concrete ECOPODE 6 m^3 units with a crest width of 5 meters and a crest height of 2.38 above MSL. Due to relative strong wave conditions and a steep sea bed the armour units for this berm are relative large. The wall in combination with the berm realises an average reduction of 64 % but the wave overtopping remains high, an average of 0,359 m^3/sec/m. A breakwater would be favourable but the steep sea bed makes that very expensive. The seabed slopes are comparable with an average slope of 13 %.

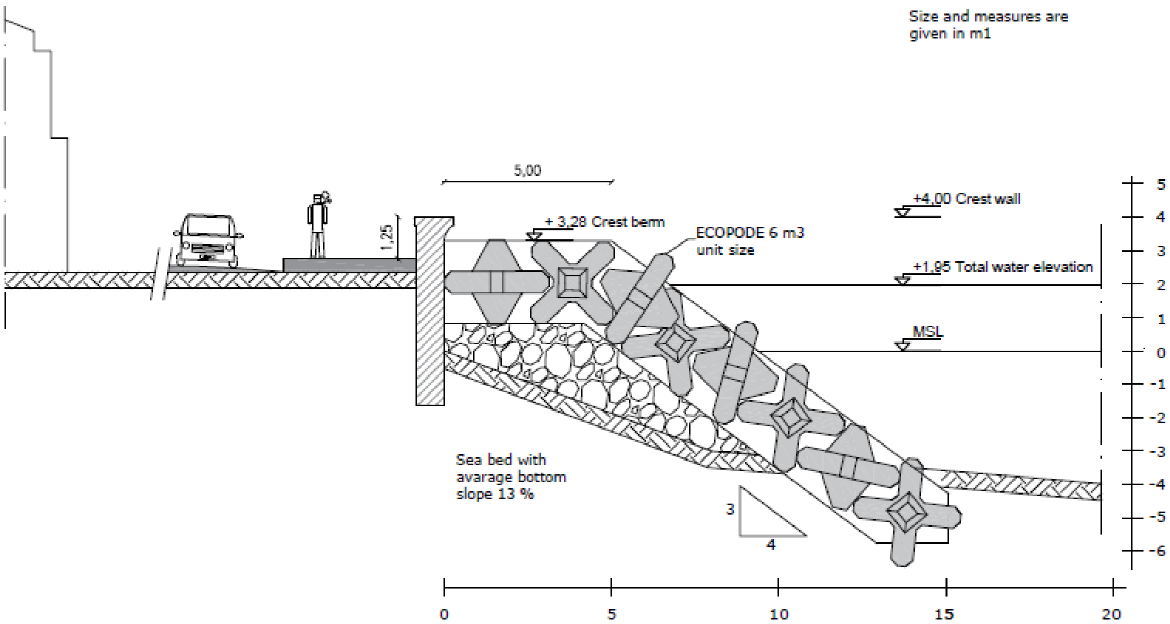


Figure 66 Detailed design for sections 3.5, 3.6 and 3.7

The detailed designs for sections 3.1, 3.8, 4.1, 4.2 and 4.3 are comparable with a permeable berm in addition to the curved sea wall. Designed from concrete ECOPODE 3 m^3 units with a crest width of 5 meters and a crest height of 3.28 above MSL. The seabed profile in both subsections are comparable with an average slope of 5% except for section 3.1 which is 11.2 %.

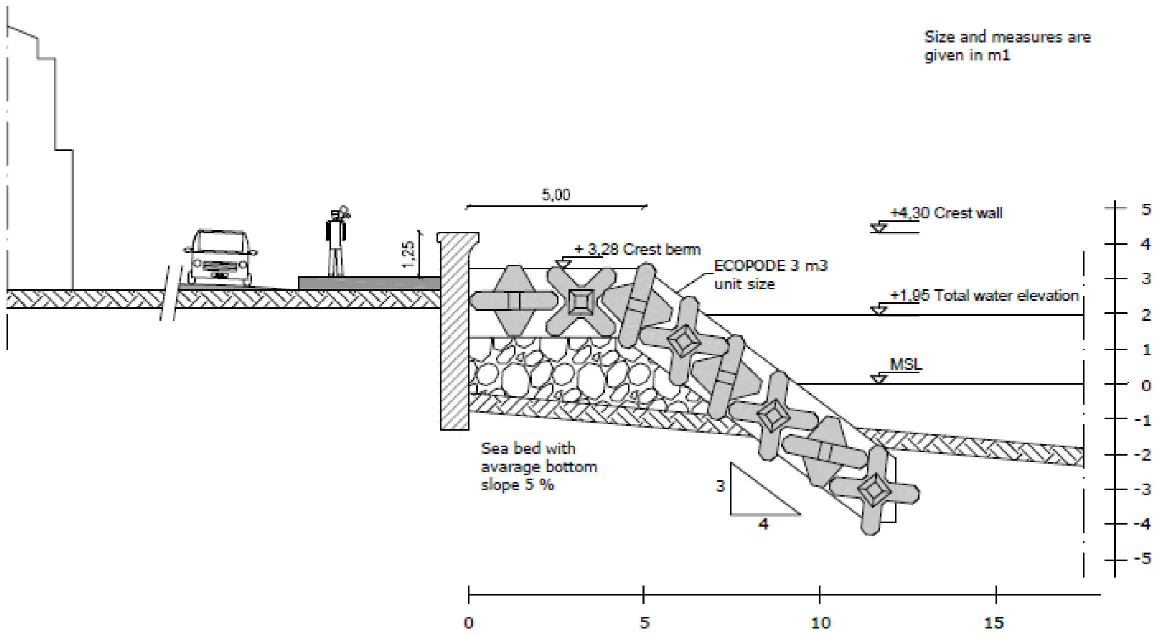


Figure 67 Detailed design for section 3.1, 3.8, 4.1, 4.2 and 4.3

Cross section for sections 4.4, 4.5, 5.3 and 5.4 both have a breakwater of approximately 500 meters in front of the Malecón Traditional. The breakwaters are designed with concrete ACCROPODE 12 m^3 units at a crest width of 12 meters with a crest height of 3.28 above MSL. Seabed of the first breakwater has an average slope of 9.2 % and the second 9.0 %. The tow of the breakwater is visual at approximately 30 meters from the wall with normal sea conditions.

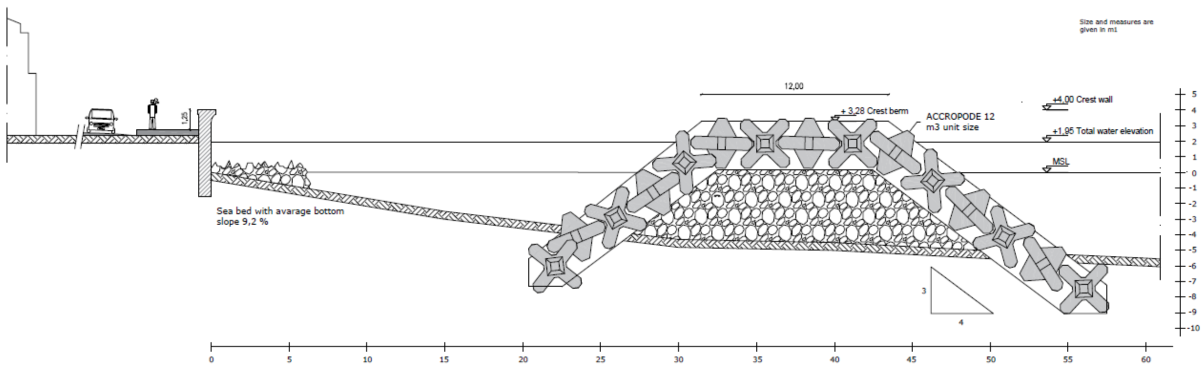


Figure 68 Detailed design for section 4.4, 4.5, 5.3 and 5.4

Appendix AB. Cost estimations

Cost estimate curved sea wall

Curved wall 100 meter				
Primary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
C1 Direct Costs of Material				
Concrete curved sea wall 4,3 m3/m1 (m3)	430,5	$ 450,00	$ 193.725,00	
				$ 193.725,00
C2 Direct costs of work by hand				
Design phase (hrs) *included in total design costs		$ 40,00	$ -	
Technical preparation (hrs)	480	$ 23,00	$ 11.040,00	
Execution phase 20 pers; 15 workingdays (hrs)	2400,0	$ 23,00	$ 55.200,00	
			$ -	
				€ 66.240,00
C3 Direct costs of equipment				
Movable crane 15 workingdays (hrs)	120	$ 60,00	$ 7.200,00	
Fuel, Lubricant, Oil (gallons)	1000	$ 0,64	$ 640,00	
			$ -	
				$ 7.840,00
C4 Direct costs of means of support and small material				
				$ 8.034,15
C5 Total direct costs(C1 + C2 + C3 + C4)				$ 275.839,15
C6 Indirect Costs (11 % of C5)				$ 30.342,31
C7 Total Costs (C5 + C6)				$ 306.181,46
C8 Profit (20% of total costs)				$ 61.236,29
C9 Total primary costs				$ 673.599,20

Secondary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
P1 Temporary facilities				
Storage, toilets ect.	1	$ 3.000,00	$ 3.000,00	
				$ 3.000,00
P2 Transport				
Concrete trucks 5 trucks (hrs)	600	$ 24,28	$ 14.568,00	
Other materials 2 trucks (hrs)	240	$ 24,28	$ 5.827,20	
				$ 20.395,20
P3 Additional costs				
Proof of good quality	1	$ 2.000,00	$ 2.000,00	
Transport unused materials (5 % of P2)			$ 1.019,76	
Closing of the Malecón			$ 15.000,00	
				$ 18.019,76
P4 Banking				
P5 Security (1% of C7)			$ 3.061,81	
				$ 3.061,81
P6 Unpredictable costs(10 % of C9)			$ 67.359,92	
				$ 67.359,92
P7 Total Secondary Costs				$ 111.836,69
T Total Costs of Construction				$ 785.435,90

Cost estimate berm

Berm 100 meter 5 mtr crest + 3,28 MSL				
Primary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
C1 Direct Costs of Material				
Prefab concrete, Ecopode (m3)	2369,85	$ 581,90	$ 1.379.008,61	
Core material (1000 kg)	4524	$ 43,64	$ 197.437,65	
Reinforcement dowels and formwork		$ 220,00	$ -	
				$ 1.576.446,26
C2 Direct costs of work by hand				
Design phase (hrs) *included in total design costs		$ 40,00	$ -	
Technical preparation (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
Execution phase (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
			$ -	
				€ -
C3 Direct costs of equipment				
Movable crane (hrs)		$ 24,28	$ -	
Fuel, Lubricant, Oil (gallons)	1000	$ 0,64	$ 640,00	
Placement core material, dry construction (tons)	1740	$ 20,37	$ 35.437,53	
Divers for placement tow *included in price Ecopode			$ -	
Excavation tow	550	$ 34,33	$ 18.881,50	
			$ -	
				$ 54.959,03
C4 Direct costs of means of support and small material				
				$ 48.942,16
C5 Total direct costs(C1 + C2 + C3 + C4)				$ 1.680.347,44
C6 Indirect Costs (11 % of C5)				$ 184.838,22
C7 Total Costs (C5 + C6)				$ 1.865.185,66
C8 Profit (20% of total costs)				$ 373.037,13
C9 Total primary costs				$ 4.103.408,46

Secondary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
P1 Temporary facilities	1	$ 3.000,00	$ 3.000,00	
				$ 3.000,00
P2 Transport				
Transportation by sea, 20 km (ton/km)	113752,8	$ 0,58	$ 66.192,41	
Transportation by land, 20 km (ton/km)	90480	$ 0,73	$ 65.812,55	
				$ 132.004,96
P3 Additional costs				
Proof of good quality	1	$ 2.000,00	$ 2.000,00	
Transport unused materials (5 % of P2)			$ 6.600,25	
Closing of the Malecón			$ 10.000,00	
				$ 18.600,25
*P4 Banking *included in unpredictable costs*				
P5 Security (1% of C7)			$ 18.651,86	
				$ 18.651,86
P6 Unpredictable costs(10 % of C9)			$ 410.340,85	
				$ 410.340,85
P7 Total Secondary Costs				$ 582.597,91
T Total Costs of Construction				$ 4.686.006,37

Berm 100 meter 20 mtr crest + 2,28 MSL				
Primary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
C1 Direct Costs of Material				
Prefab concrete, Ecopode (m3)	5416,8	$ 581,90	$ 3.152.019,67	
Core material (1000 kg)	41912	$ 43,64	$ 1.829.135,03	
Reinforcement dowels and formwork		$ 220,00	$ -	
				$ 4.981.154,70
C2 Direct costs of work by hand				
Design phase (hrs) *included in total design costs		$ 40,00	$ -	
Technical preparation (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
Execution phase (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
			$ -	
				€ -
C3 Direct costs of equipment				
Movable crane (hrs)		$ 24,28	$ -	
Fuel, Lubricant, Oil (gallons)	1000	$ 0,64	$ 640,00	
Placement core material, dry construction (tons)	41912	$ 20,37	$ 853.596,35	
Divers for placement tow *included in price Ecopode			$ -	
Excavation tow	550	$ 34,33	$ 18.881,50	
			$ -	
				$ 873.117,85
C4 Direct costs of means of support and small material				
				$ 175.628,18
C5 Total direct costs(C1 + C2 + C3 + C4)				$ 6.029.900,73
C6 Indirect Costs (11 % of C5)				$ 663.289,08
C7 Total Costs (C5 + C6)				$ 6.693.189,81
C8 Profit (20% of total costs)				$ 1.338.637,96
C9 Total primary costs				$ 14.725.017,57

Secondary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
P1 Temporary facilities	1	$ 3.000,00	$ 3.000,00	
				$ 3.000,00
P2 Transport				
Transportation by sea >300 kg, 20 km (ton/km)	260006,4	$ 0,58	$ 151.296,94	
Transportation by sea <300 kg, 20 km (ton/km)	838240	$ 0,36	$ 304.855,84	
Transportation by land, 20 km (ton/km)	0	$ 0,73	$ -	
				$ 456.152,78
P3 Additional costs				
Proof of good quality	1	$ 2.000,00	$ 2.000,00	
Transport unused materials (5 % of P2)			$ 22.807,64	
Closing of the Malecón			$ 10.000,00	
				$ 34.807,64
*P4 Banking *included in unpredictable costs*				
P5 Security (1% of C7)			$ 66.931,90	
				$ 66.931,90
P6 Unpredictable costs(10 % of C9)			$ 1.472.501,76	
				$ 1.472.501,76
P7 Total Secondary Costs				$ 2.033.394,08
T Total Costs of Construction				$ 16.758.411,65

Cost estimate breakwater

Breakwater 100 meter 12 mtr crest + 3,28 MSL				
Primary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
C1 Direct Costs of Material				
Prefab concrete, Ecopode (m3)	25516,92	$ 581,90	$ 14.848.219,20	
Core material (1000 kg)	40222	$ 43,64	$ 1.755.379,59	
Reinforcement dowels and formwork		$ 220,00	$ -	
				$ 16.603.598,79
C2 Direct costs of work by hand				
Design phase (hrs) *included in total design costs		$ 40,00	$ -	
Technical preparation (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
Execution phase (hrs) *included in unit prices Ecopode/core		$ 23,00	$ -	
			$ -	
				€ -
C3 Direct costs of equipment				
Movable crane (hrs)		$ 24,28	$ -	
Fuel, Lubricant, Oil (gallons)	3000	$ 0,64	$ 1.920,00	
Placement core material, dry construction (tons)	40222	$ 20,37	$ 819.177,14	
Divers for placement tow *included in price Ecopode			$ -	
Excavation tow	4150	$ 34,33	$ 142.469,50	
			$ -	
				$ 963.566,64
C4 Direct costs of means of support and small material				
				$ 527.014,96
C5 Total direct costs(C1 + C2 + C3 + C4)				$ 18.094.180,39
C6 Indirect Costs (11 % of C5)				$ 1.990.359,84
C7 Total Costs (C5 + C6)				$ 20.084.540,24
C8 Profit (20% of total costs)				$ 4.016.908,05
C9 Total primary costs				$ 44.185.988,52

Secondary Costs				
	Quantity	Prite Per Unit	Subtotal	Total
P1 Temporary facilities	1	$ 3.000,00	$ 3.000,00	
				$ 3.000,00
P2 Transport				
Transportation by sea >300 kg, 20 km (ton/km)	1224812	$ 0,58	$ 712.714,52	
Transportation by sea <300 kg, 20 km (ton/km)	804440	$ 0,36	$ 292.563,26	
Transportation by land, 20 km (ton/km)	0	$ 0,73	$ -	
				$ 1.005.277,79
P3 Additional costs				
Proof of good quality	1	$ 2.000,00	$ 2.000,00	
Transport unused materials (5 % of P2)			$ 50.263,89	
Closing of the Malecón			$ 10.000,00	
				$ 62.263,89
*P4 Banking *included in unpredictable costs*				
P5 Security (1% of C7)			$ 200.845,40	
				$ 200.845,40
P6 Unpredictable costs(10 % of C9)			$ 4.418.598,85	
				$ 4.418.598,85
P7 Total Secondary Costs				$ 5.689.985,93
T Total Costs of Construction				$ 49.875.974,45