Coastal protection Malecón seawall

A study to develop a sea defence solution that prevents unacceptable flooding and damage of the *Malecón Tradicional* in Havana, Cuba

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Abstract

This report presents a study of the Malecón Tradicional in front of 'Centro de Havana' and 'Havana Vieja' in the city Havana, Cuba. The objective is to develop a sea defence solution that prevents unacceptable flooding of the hinterland and damage of the Malecón seawall.

Implementing the recommended solutions reduces the overtopping sufficiently. Along the entire study area, it is recommended to raise the seawall and to construct a revetment. It was concluded that a curved wall is only useful in combination with a revetment for a small part of the seawall.

Both the top part and the seaside face of the Malecón seawall need a structural fortification in order to withstand the design conditions. It is recommended to partly remove the top of the current wall and replace it with a new wall, which is extended in seaward direction. Both for the design storm and for the ultimate limit state, the proposed solution satisfies the strength requirements.

Foreword

This is the final report of a study project as part of the Master program Civil Engineering at Delft University of Technology. The work was carried out by five students during eight weeks in Havana at the Hydraulic Research Centre (CIH) of Instituto Superior Politécnico José Antonio Echeverría (CUJAE).

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Summary

The objective of the project is to develop a sea defence solution in Havana, Cuba, to prevent unacceptable flooding of the hinterland and damage of the Malecón seawall. This solution can be offshore, nearshore and/or onshore. The study area entails the Malecón Tradicional in front of 'Centro de Havana' and 'Havana Vieja' between La Punta and Calle Marina. The provided solution is designed for a lifetime of 50 years with a maximum allowable wave overtopping of 50 L/s/m for a storm event with a return period of 50 years.

From a statistical analysis of the marine climate the hydraulic boundary conditions with a combined return period of 50 years have been established. The governing offshore significant wave height equals 7.80 m. The governing storm surge level equals 1.88 m, including astronomical tides and sea level rise. Using a SWAN model the wave height is transformed, resulting in a maximum significant wave height of 4.27 m nearshore.

In order to present an accurate solution, the project area was divided into five sections and two subsections, based on the state of the current wall and hydraulic boundary conditions. Then, a representative significant wave height was selected per section. With these results the wave overtopping and loads on the Malecón seawall have been determined.

With the finite element program ANSYS the current state of the seawall was modelled using both the load of the design storm with a return period of 50 years and an extreme load condition, for which hurricane Wilma was taken. From this analysis it was concluded that the wall cannot withstand the design storm in its current condition. The wall fails locally due to high tensile and shear stresses at sidewalk level, which may cause the top of the wall to break off.

Based on social criteria, technical criteria and costs, a multi criteria analysis selected four feasible solutions: a curved wall, a revetment, a submerged breakwater and an emerged breakwater. The option of raising the wall is restricted by the boundary conditions and was only applied as partial solution in combination with any of the four selected alternatives. A cost-benefit analysis of preliminary designs showed that it is impossible to reduce the overtopping enough by merely raising or curving the seawall. The revetment and breakwater were designed in such a way that the overtopping is reduced to 50 L/s/m.

The partial solution of a raised wall is most economic when it is combined with the necessary structural fortification, which differs per section. For all sections of the Malecón seawall, the design of the revetment turned out to be more economic than the design of a

breakwater. When the revetment is combined with a raised and/or curved wall, a smaller revetment can be used, since part of the overtopping is resolved by raising or curving the wall. The best solution per section is displayed in table 1.

Section	1	2	2*	3	4	4*	5
Revetment	x	x	x	x	x	x	x
Breakwater							
Major structural fortification		x	x		x	x	x
Minor structural fortification	x			x			
Significant raise of the wall	x			x	x	x	x
Curved wall				x			

Table 1: Overview of the combinations of solutions per section

Both the top part and the seaside face of the Malecón seawall need a structural fortification. For this fortification a partial replacement alternative is recommended for all sections. This entails partly removing the top of the current wall and replacing it with a new wall, which is extended in seaward direction. The solution ensures limited crack width to prevent corrosion, since the structure is placed in salt water. The total costs for constructing the new seawall along the entire seawall amount to approximately 6-9 million CUC.

A detailed design was made per section, including dimensions, types of materials and a construction method. Concrete cubes are used as armour material of the revetment with a slope of 1:1.5. The total costs for constructing the revetment along the entire seawall amount to approximately 16-20 million CUC. The resulting overtopping and effectiveness of the cummulative solutions are displayed in table 2.

	Current wall	+ Raised wall		+ Curved wall		+ Revetment	
Section	q	q	effectiveness	q	effectiveness	q	effectiveness
1	345	345	0 %			50	100 %
2	427	416	3 %			50	100 %
2*	384	344	12 %			50	100 %
3	754	420	47 %	344	58 %	50	100 %
4	624	558	11 %			50	100 %
4*	741	400	49 %			50	100 %
5	624	558	11 %			50	100 %

Table 2: Resulting overtopping in L/s/m and effectiveness for cummulative solutions

Concluding, the overtopping will be reduced to 50 L/s/m for a storm with a return period of 50 years along the entire seawall, if the recommended solutions are implemented. Both for a storm with a return period of 50 years and for the ultimate limit state, the proposed solution for the reconstruction of the seawall satisfies the strength requirements. Finally, it is recommended to perform a physical model test to confirm the overtopping quantities for the proposed design.

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Chapter 1

Introduction

1.1 Problem description

The Malecón is a seawall with a length of seven kilometres in Havana, Cuba. The Malecón became the fast road between the old and the newer part of Havana, and nowadays it is still one of the most important roads in city. The study area for this project is called the 'Malecón Tradicional' with approximately two km of length. During aggressive weather and hurricanes, waves overtop the wall, damaging the wall itself and the buildings along the seawall. At this moment restoration works are still in progress, not only to improve the living conditions at the Malecón, but also to make it more attractive for tourists. Nowadays the Malecón has a very important social function, being declared the area that protects Patrimony of the Humanity by the UNESCO.

The main objective of the project is to find a sea defence solution for 'Malecón Tradicional' to prevent unacceptable flooding.

1.2 Approach

First the project scope is defined by analysing the study area, making an overview of the important stakeholders and defining the design criteria. Next, a statistical analysis is performed on the marine climate data in order to obtain combinations of loads that have a combined return period corresponding to the established design criterion. The nearshore hydraulic boundary conditions are obtained using a numerical model. The corresponding wave overtopping and hydraulic loads are calculated using empirical relations. The seawall is divided into sections based on a structural assessment of the current seawall, that is performed by analysing the construction of the seawall and using a numerical model.

Next, alternative solutions are developed and the four most feasible solutions were selected using a multi criteria analysis. The selected alternatives are then used to find the best solution per section, using a cost/benefit analysis. If necessary to meet the design criteria, a combination of solutions is applied. The final solution is designed in detail, including dimensions, materials and a construction method. Moreover, the final design is evaluated using numerical models both for the hydraulic and the structural design criteria. The costs of the final design are approximated. Finally, some conclusions and recommendations for further research are given.

1.3 Outline of the report

This report presents a study of the Malecón seawall. Chapter 1 gives general information about the project and the approach used to cope with the project. Moreover, it describes the structure of the report. Chapter 2 describes the project scope: the problem is defined more concisely, the background information is given, the stakeholders in the project are analysed and the design criteria are established. In chapter 3 the marine climate of Havana is discussed. It contains information on morphology and hydrodynamics. Currents, waves and water levels are described, as well as detailed historical information on cold fronts and hurricanes. In chapter 4 the hydraulic boundary conditions are established by transforming offshore wave heigths into nearshore values that are used to calculate overtopping and loads on the seawall. Chapter 5 contains an initial structural assessment of the seawall. In this chapter the structural properties of the Malecón are discussed and the Malecón is divided into different characteristic sections. Furthermore the current situation is assessed using the ANSYS model.

Alternative solution are developed in chapter 6, including a multi criteria analysis to select feasible solutions. Chapter 7 describes the method used to come to a final design. For all feasible solutions some first calculations are made to research the overtopping reduction. In chapter 8 the final design is presented for the structural as well as the hydraulic solution per section, including dimensions, materials, construction method and costs. Finally the conclusion and recommendations are presented in chapter 9.

Chapter 2

Project scope

This chapter describes the analyses. In section 2.1 a more concise problem definition is given. Section 2.2 gives more background information and elaborates on the study area. The stakeholders are analysed in section 2.3. Finally, in section 2.4 the design criteria and boundary conditions are established and the assumptions are listed.

2.1 Problem definition

2.1.1 Problem analysis

After several years of lack of maintenance and care, the Malecón seawall in Havana fell into disrepair. The Historian Office initiated a total improvement of the Malecón area including buildings, infrastructure, water and power lines and the coastal protection. The coastal protection will be considered in this project.

The Havana City Seawall is built as a part of the coastal protection in Havana, but it is no longer satisfactory. The seawall should protect Havana from hurricanes, cold fronts and other hydraulic threats. Unfortunately the Malecón suffers from wave overtopping, which sometimes results in flooding.

2.1.2 Problem definition

Havana City suffers from flooding due to wave overtopping of the Malecón, caused by storms and a lack of maintenance of the sea wall.

2.1.3 Objective

The objective of this project is the development of a solution, offshore, nearshore and/or onshore, that reduces future flooding and wave overtopping of the Malecón in Havana, while maintaining the environmental, historical and cultural value of the Malecón area.

2.2 Study area

In order to define and describe the study area, first some background information will be given, going from large to small scale, about Cuba, Havana and the Malecón in general. Then the study area will be defined and the directly effected zone by the project will be described.

The information in this paragraph is based on the report Coastal defence for Centro Habana [Baart et al., 2006], the book Cuba [Frey, 2002] and the powerpoint presentation [de las Cuevas Toraya, 2013].

2.2.1 Background information

Geography

Cuba lies in the Caribbean. The north coast faces the Atlantic Ocean and the Caribbean Sea lies on its south coast. Cuba lies on the relatively small Caribbean tectonic plate. Between Cuba and Jamaica lies the Cayman Trench, which is 7,200 m deep. This separates the North American and Caribbean tectonic plates. Due to tectonic activity Cuba has been gradually tilted, causing the formation of limestone cliffs in the north and mangrove swamps on the south coast.

Climate

Cuba has a subtropical climate. It is predominantly warm with an average temperature of 25.5 °C and average relative humidity of 78%. The average rainfall is 1,386 mm per year. Two seasons can be defined for Cuba, viz. a dry season (November to April) and a rainy season (May to October). Moreover, hurricanes can hit Cuba between July and November. However comparable storms can affect Cuba in other months as well. Between October and April cold fronts can cause strong winds (that may reach hurricane-type forces), heavy rainfall and high waves. Dangerous wave conditions can develop, particularly for winds that are generated by cold fronts in the Gulf of Mexico.

City of Havana

With 2,500,000 inhabitants Havana is the capital of Cuba. The center of the city (Centro de Havana) and Old Havana (Havana Vieja) are protected from the sea by the Malecón seawall. Especially these two parts of Havana have a characteristic Spanish colonial architecture, because for 400 years the city was occupied by Spanish colonists. Due to this interesting mix of architecture the UNESCO designated Old Havana as a World Heritage Site in 1982.

The Malecón sea wall

The Malecón runs from Castillo de la Punta in Havana Vieja, also the entrance of the harbour, to La Chorrera, another castle at the mouth of Río Almedares. The American authorities started the construction of the Malecón in 1901. During the erection of the Malecón high waves were observed, so the plans were changed in order to protect Havana. The total construction of the Malecón lasted for about 60 years and was finished on 3 June 1959. The Malecón was built in several parts as can be seen in figure 2.1. Due to the long construction period and the evolving knowledge on concrete, there is large variance in the quality of construction of the seawall. The Malecón in Havana Vieja and Centro Havana was constructed first, section 1 and 2 in the figure, this part was finished around 1925. Therefore this part is the most vulnerable to extreme weather conditions.

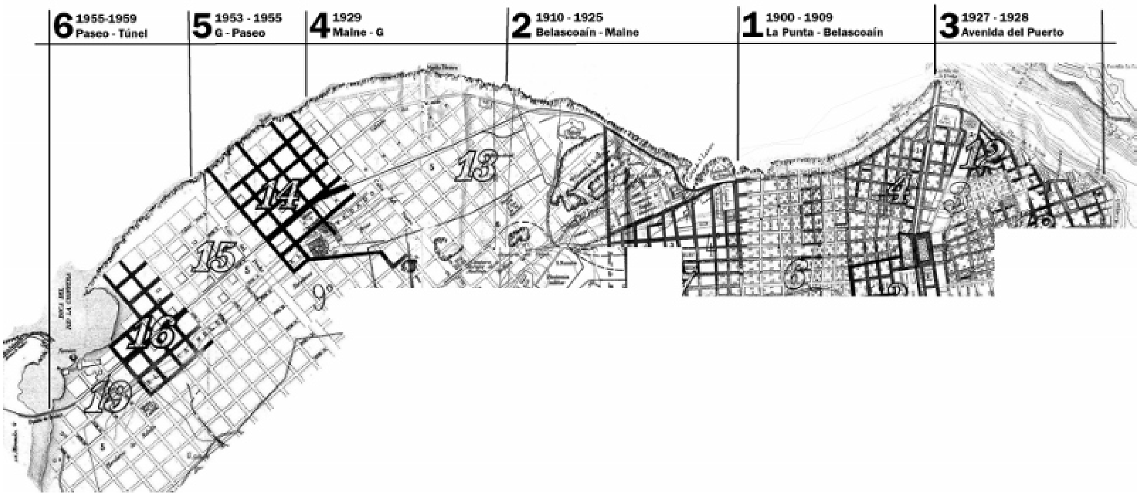

Figure 2.1: The order in which the Malecón sea wall was built

Apart from acting as a coastal defence for Havana, the Avenida Antonio Maceo, the road along the Malecón, connects the older and newer part of Havana. Also the Malecón has a significant social amenity value. The inhabitants of Havana use the Malecón as a place to socialize. Especially at night, when a lot of inhabitants come together to meet informally or to fish at the Malecón wall. The seawall is not used for port activities and there are no possibilities nor facilities to berth ships.

2.2.2 Definition of study area

The section of the seawall that is considered in this project is the Malecón Tradicional. This is the oldest part of the wall between La Punta and Calle Marina, just west of Hospital Nacional Hermanos Almejeiras. It consists of section 1 and a small part of section 2 of figure 2.1. This part of the Malecón and the area behind the wall is part of a rehabilitation and development project of the Office of the Historian of Cuba. The coastal protection that will be considered in this project has an influence on the entire city of Havana and its residents, but is especially important for the area directly behind the Malecón when considering the flood risk reduction.

Figure 2.2 shows the area that is most affected by the flooding. The purple line is the Malecón Tradicional with a crest height of about four meters above mean sea level. The red line is the line in the landscape behind the wall which has a height of four meters above MSL as well. When a severe storm hits the seawall, some of the water will overtop the wall. The area between the purple and red line will be flooded, because here the ground is lower than four meters above MSL. In between lies the San Lazaro, which is a higher street with a height of four meters above MSL as well. Therefore this street will act as an obstacle for the water. The arrows show the direction of the flooding. As can be seen, initially most of the water will come from Parque Maceo and La Punta as these are lower areas. From Havana Vieja the height data is not complete, but probably a considerable part will be flooded as well.

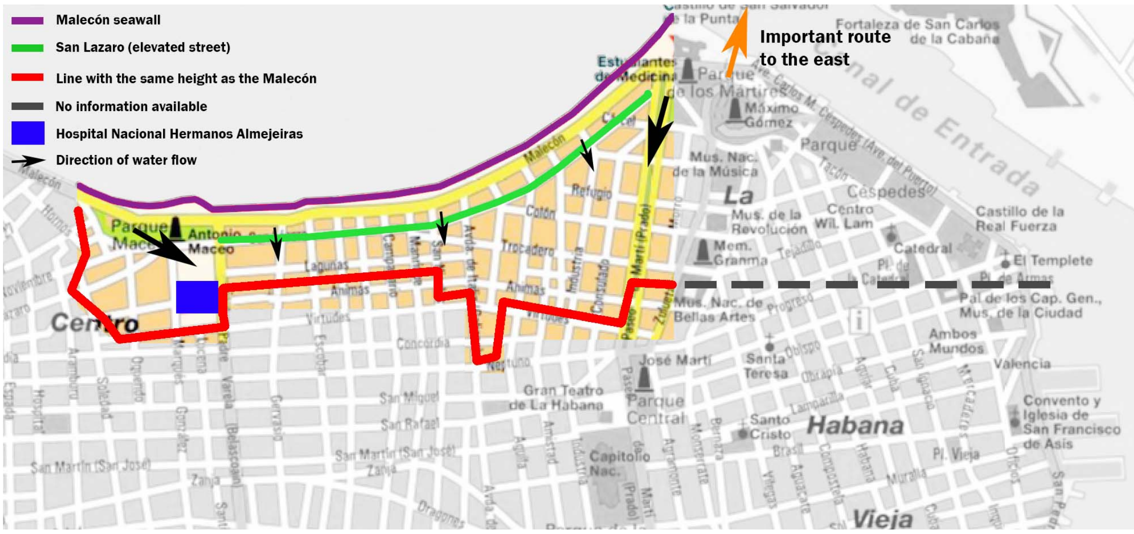

Figure 2.2: Definition of the study area

2.3 Summary of key stakeholders

In this paragraph the stakeholders of the project will be discussed. All the groups which have in some way concern with the project or are affected by the project are stakeholders in the project. They will be listed here, including a description of their role in the project. For example some of these groups are significantly affected, but have little influence on the development of the project. A lot of the information was based on the document Terea Technica [Oficina del Historiador de la ciudad de la Habana, 2012].

- **Investors**
  - *Oficina del Historiador de la Ciudad de la Havana*
    The Oficina del Historiado de la Ciudad de la Havana is the main investor of the project. This Office of the Historian is responsible for historical monuments. Their aim is to preserve and restore these monuments as much as possible while maintaining the characteristic view of Havana. The Office of the Historian is the main investor in this project, as it aims to invest in the area behind the Malecón by restoring buildings and by creating new hotels and restaurants. By law they are obligated to evaluate the flood risk before starting such projects. Therefore the Office of the Historian has requested an investigation of the study area. In order for the Malecón to maintain its function and preserve the beauty and aesthetics of this part of Havana, reduction of the flood risk is necessary.
    The group Grupo Inversionista Plan Malecón Unidad Presupuestada Inversionista (UPI) is part of the Historian Office and is responsible for all investments in Old Havana. It is related to all studies, surveys and other activities related to the project on the subject of flood protection of the traditional Malecón.
  - *Grupo Coordinador Provincial de Redes Técnicas. CAP. (Pending confirmation)*
    This is a group from the capital government, which has the task to direct the entire investment process with regard to the technical networks of the capital, such as the water networks, the roads and the electrical grids. It is an entity subordinate to the city government. This party has not yet confirmed its participation in the project, but has a good potential in doing so.
  - *Delegación Provincial de Recursos Hidráulicos. INRH. (Pending confirmation)*
    This is the representation of the National Institute of Hydraulic Resources, which can be seen as the ”Ministry of Water” in Cuba. This party also has not yet confirmed its participation in the project, but has a good potential in doing so.
- **Research organizations**
  - *Centro de Investigaciones Hidráulicas (CIH), CUJAE. MES.*
    The hydraulic research center of CUJAE, which studies the problem and provides possible solutions.
  - *Instituto de Meteorología (INSMET). CITMA.*
    The meteorological institute, which provided the bathymetric information and historic climate data.
  - *Geo Cuba and Empresa Geominera Oriente (GEOM). MINBAS.*
    Geological institutes, which research soil conditions.
  - *Facultad de Arquitectura, CUJAE. MES. (Pending action)*
    The Faculty of Architecture of CUJAE. This faculty will be part of the new

aesthetical design team of the study area.

- **Inhabitants and enterprises situated directly behind the Malecón**
  - *Hospital Nacional Hermanos Almejeiras*
    One of the largest and most important hospitals in Cuba is the Hospital Nacional Hermanos Almejeiras. Furthermore the hospital is one of the buildings which characterizes the skyline of Havana, because of its remarkable height. The Hospital Nacional Hermanos Almejeiras is located in Havana Centro and is constructed roughly 150 meters behind the Malecón. Therefore the hospital also has a great interest in the project. In addition to the damage caused by floods, its services are affected during these conditions.
  - *Inhabitants of the study area*
    The inhabitants of Havana Vieja and Centro de Havana have a great interest in the project. They suffer considerably if the area behind the Malecón is flooded. Therefore they will benefit greatly from the project. Even though they will benefit, they will be reluctant to accept any change of view on the Malecón and the sea.
  - *Businesses in the study area*
    Businesses like companies and restaurants in Havana Vieja and Centro de Havana are damaged when flooding occurs, which will lead to economic losses. Therefore these businesses will benefit greatly from the project.
- **EPOT (Enterprise of Projects of Transport Works)**
  A company which will make the final design for the project, it is important to note that during this study, the final design of the project is not yet assigned to any company.
- **EOM (Enterprise of Maritime Works)**
  The contractor of the project, again it is important to note that during this study, the construction of the project is not yet assigned to any company.
- **National government of Cuba**
  The project is of interest for the national government as well because it will improve the potential of the economical development of the study area.
- **Recreational users of the Malecón**
  The Malecón is used for a lot of social activities. One of the aims of this project will be to preserve the social activities of the seawall as much as possible.
- **Road users of the Avenida Antonio Maceo (Malecón)**
  Over the centuries, the road on the Malecón (Avenida Antonio Maceo) has become of significant importance for Havana. Hence it is favourable to protect the road against closure and damages during extreme weather conditions. During construction works the usage of the road will possibly be hindered, this negative effect should be minimized.

- **Port of Havana**
  The port of Havana has interest in the project as well. They are especially concerned with the kind of solution proposed. Whatever the solution of the project will be the harbour needs to stay accessible for all ships, which have Havana as destination.

- **UNESCO**
  As a result of the long Spanish colonial period, Havana Vieja has an interesting mix of architecture. Because of this characteristic architecture the UNESCO enlisted Havana Vieja, including the Malecón, as a World Heritage Site in 1982.

In order to determine which of these stakeholders are important for this project the influence and the stake is determined per stakeholder. The results are presented in figure 2.3. From this figure the most important stakeholders that may alter the course of this project along the way can be deduced.

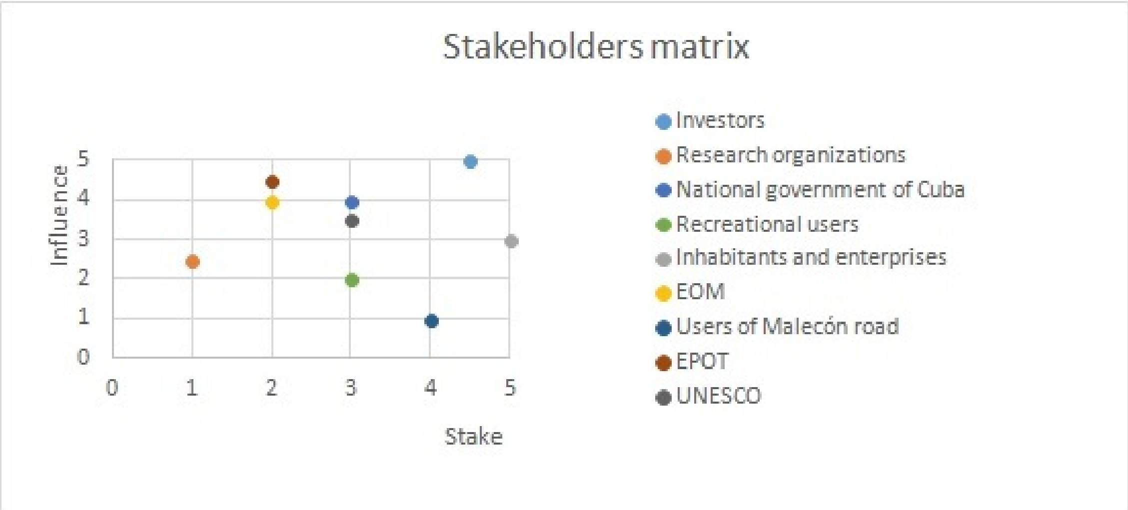

Figure 2.3: Stakeholders matrix

In conclusion, the different investors, which are named above, have the largest stake and influence in the project. Therefore it is important to follow their demands and wishes. Nevertheless the demands and wishes of other parties have to be considered as well.

2.4 List of conditions and criteria

The different stakes of the stakeholders discussed in section 2.3 result in a list of design criteria. The boundary conditions are defined by environmental and human requirements which are outside the scope of this study, that form the absolute boundaries of the project.

Assumptions are made for parameters, which together with the criteria and conditions, will be used to develop different design alternatives.

2.4.1 Design criteria

1. The area behind the Malecón should be protected from storms with a return period of 50 years [Oficina del Historiador de la ciudad de la Habana, 2012].
2. The provided solution should be designed for a design life of 50 years [Oficina del Historiador de la ciudad de la Habana, 2012].
3. The maximum allowable mean wave overtopping is 50 L/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 [Oficina del Historiador de la ciudad de la Habana, 2012], [Verhagen et al., 2009].
4. The maximum allowed difference in height between the crest level of the seawall and the adjacent sidewalk is 1.25 m [Oficina del Historiador de la ciudad de la Habana, 2012].
5. The view of the city of Havana should be preserved as much as possible, because the Malecón is an icon for Havana and Cuba [Oficina del Historiador de la ciudad de la Habana, 2012].
6. Habana Vieja is stated as a National Monument by the National Monument Commission Resolution 1978, so buildings in this part of the city have to be preserved [UNESCO CLT WHC, 2015].
7. The Malecón has a strong social function for the inhabitants and visitors of Havana who use the sea wall as a place to do several social activities. This social function should be respected [Oficina del Historiador de la ciudad de la Habana, 2012].
8. Low maintenance costs are favourable.
9. The provided solution should be a standard practice and built with local materials.
10. Hospital Nacional Hermanos Almejeiras should be able to be used in a normal manner during the design storm.
11. The harbour has to remain accessible for ships. As long as no adaptations are made to the entrance channel, the harbour does not need to be considered in further detail.
12. Value and costs should be evaluated.

2.4.2 Boundary conditions

1. The area investigated during this project is the Malecón adjacent to Havana Centro and Havana Vieja, with Calle Marina as the west and Calle Prado as the east boundary.
2. The bathymetry as provided by the Office of the Historian will be used.
3. The area around the coastline consists mainly of rock [Baart et al., 2006].
4. The sea level rise will be 0.27 m by 2050 [Centella et al., 2009].
5. Plans for reducing flood risk should not intervene with the plans of the Office of the Historian for the total project area.
6. If there is no Cuban law available or suitable, European standards will apply.
7. For a return period of 50 years the offshore significant wave height is 9.2 m, see section 3.3.
8. For a return period of 50 years the total surge amounts 1.67 m, see section 3.4.
9. For a return period of 50 years the total elevation is therefore MSL +1.94 m, considering the total surge and sea level rise.

2.4.3 Assumptions

1. The solution provided for the project area will fit with adjacent sections of the seawall.
2. During storms the drainage system cannot discharge the surplus water behind the seawall.
3. The amount of sediment near the Havana coast is negligible, as is the longshore sediment transport, confirmed by prof. Córdova. This is due to the fact that the sea bed mainly consists of rock.

Chapter 3

Marine climate data analysis

The Malecón seawall is exposed to several different hydraulic loads, of which the most important ones are considered here. Combinations of waves and high water levels cause loads on the seawall and wave overtopping. First section 3.1 describes currents in the Havana Bay area. In section 3.2 the morphology of Havana Bay is described. The information on waves and water levels is analysed in section 3.3 and 3.4 respectively. Finally the governing load combination is determined in section 3.5.

3.1 Currents

According to the Cuban Department of Oceanography [Frag. et al., 1995] two important types of currents exist near the Cuban coast. The first type is tidal currents, that have a periodic character. The second type is residual currents, that have a non-periodic character. Coastal currents are influenced by the tide. During the winter season currents influenced by atmospheric phenomena can lead to high velocities (> 1 m/s) near the coast and can block the tidal current.

Tidal currents have an average velocity of about 0.1 m/s with maximum velocity of 0.37 m/s. The tidal character at Cuba's north coast is diurnal with small tidal range (see section 3.4.1). Residual currents are mainly caused by differences in temperature, density, salinity or pressure or due to wind driven currents. The tidal and residual currents have low velocities. Thus it is assumed these will have little effect and currents are not taken into account.

3.2 Morphology

The majority of the sea floor in Havana Bay consists of rock. Further offshore, in the Straight of Florida, very little sediment is found at large depths. Nonetheless, for this report it is assumed that this does not affect the morphology of the Havana Bay area. Therefore, cross-shore and longshore sediment transport will not be considered.

3.3 Waves

As introduced in section 2.2.1 the most important phenomena that cause large significant wave heights in Havana Bay are hurricanes and cold fronts. The meteorological institute of Cuba [Meteorological Institute, 2013] has measured historical wave data for thirty six years of events caused by each of these phenomena.

3.3.1 Tropical cyclones

Tropical cyclones are circular systems that form the center of low pressure zones. In the Atlantic Ocean near Cuba these tropical cyclones originate at latitudes between 5 ° and 35 ° north in the summer season. For the formation some necessary conditions apply. First an initial disturbance (such as a cold front) should be present, the temperature of the sea surface water exceeds 26.5 °C and the Coriolis parameter should be of significant magnitude (only for latitudes larger than 5 °). These storms exclusively form above warm water, because here the required energy can be obtained to maintain the convective movement of air and humidity upto high atmospheric levels. If the conditions are favourable in the surrounding atmosphere, the wind starts to rotate in anticlockwise direction (in the northern hemisphere) around the center of minimal pressure that generates it.

According to global conventions [Simpson, 1974] tropical cyclones are classified according to their maximum wind speed. Wind speeds between 63 and 117 km/h are classified as tropical storms. If the wind speed exceeds 118 km/h, the tropical cyclone classifies as a hurricane. Table A.1 in appendix A contains wave data for tropical storms and hurricanes.

3.3.2 Cold fronts

Cold fronts separate masses of cold and dry air at high latitudes from masses of warm and humid air at lower latitudes. They generally occur between October and April, although they have occasionally been registered in September and May as well. Cuba is affected by cold fronts that are characterised by a mass of dry continental air, that can cause very strong winds from the north (occasionally reaching hurricane force) as well as rainfall and high waves.

The Cuban Meteorological Institute uses the following classification of cold fronts [Meteorological Institute, 2015]:

- Classical fronts are associated with (extra) tropical lows that move in the Gulf of Mexico or territories adjacent to the United States. This type of cold front at first produces southerly winds. When the low center moves moves towards Cuba the direction of the wind changes from south to west to northwest. Along with the change of direction the wind velocity increases a lot. Behind the cold front a high pressure zone with low temperatures exists.

- Revised fronts produce a backward motion of wind in eastern to north-eastern to northern direction when affecting Cuba. These are accompanied by increasing cloudiness and rainfall and a larger temperature drop.

- Secondary fronts strike Cuba one or two days after a classical front. These maintain the previous barometrical system but with a certain discontinuity in the meteorological elements.

Moreover, cold fronts can be classified according to their intensity of the winds, based on the maximum wind speed at an elevation of ten meters:

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

Cold fronts of moderate and heavy intensities have historically caused coastal flooding. Classical fronts with moderate intensity are the most dominant type of cold fronts (84% of all cold fronts). Table A.2 in appendix A contains wave data for cold fronts.

3.3.3 Return period of significant wave height

Using a peak-over-threshold method, the wave data is statistically analysed for different probability distributions, viz. exponential, Gumbel and Weibull, in order to find the significant wave height as a function of the return period. This method is explained in detail in appendix B. Additional to the measured wave data, the meteorological institute also provided significant wave heights for several return periods, see table 3.1. However the data on which these return periods are based is unknown. Using Microsoft Excel trendline function a logarithmic curve is fitted to this data. Both the fitted distributions based on the wave data and the given distribution are compared in figure 3.1.

It can be seen that the Gumbel and Weibull distributions lead to significantly lower significant wave heights for a given return period. Evaluating the wave data in appendix A, it can be concluded that a 5.8 m wave occurs three times in the total measured period of 36 years. Therefore the return period for a 5.8 m wave can be assumed to be equal to twelve years. In the figure it can be seen that the Gumbel and Weibull distributions give results that are closer to this assumption than the exponential extrapolation and the fitted trendline in Excel. This means that for the given data set a Gumbel or Weibull distribution is more suitable than an Exponential distribution or the Excel extrapolation.

Return period [yrs]	H_s [m]
5	6.0
10	6.9
20	7.8
50	9.2
100	10.1

Table 3.1: Significant wave height for several return periods provided by the meteorological institute

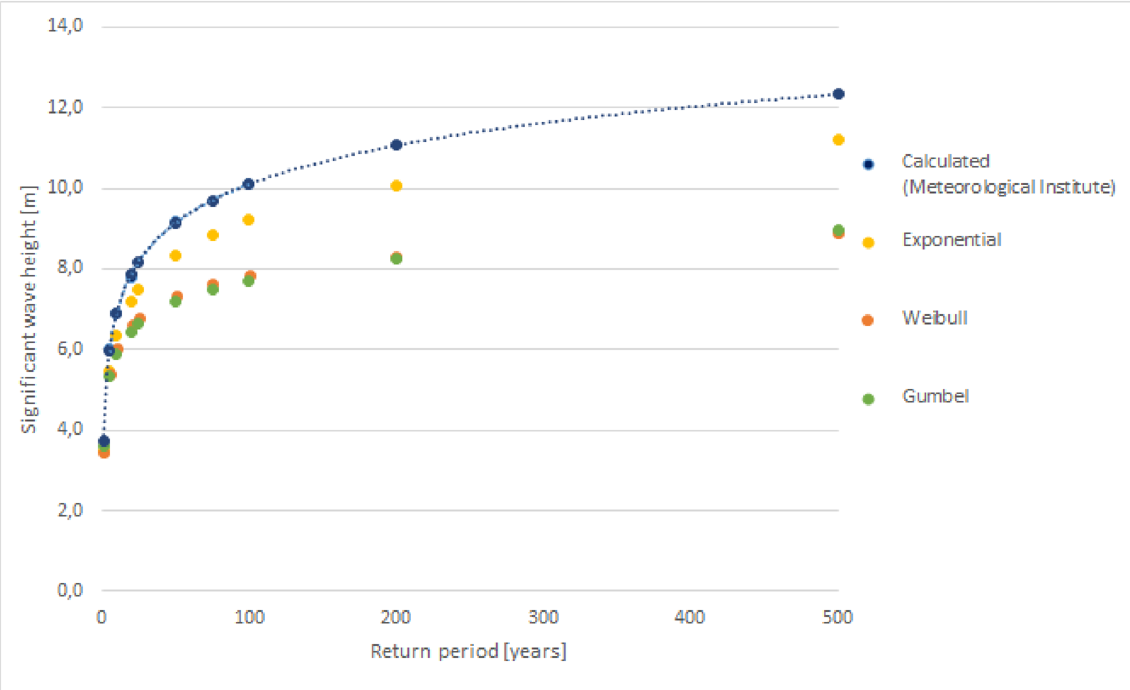

Figure 3.1: Significant wave height as a function of return period

From this, however, it can not be concluded that the Gumbel and Weibull distributions are more suitable for the given wave data set, since it is not known on which data the results from the Meteorological Institute are based. This might have been a more extensive data set, and therefore may still be a better representation of the actual wave climate.

3.4 Water level elevation

Besides waves, the water level elevation is a major contributor to severity of the overtopping of the Malecón. The total elevation consists of several independent phenomena. This section describes these phenomena: the tide, the storm surge and relative sea level rise mainly due to climate change. Likewise as for waves, the total elevation can be expressed as a function of the return period.

3.4.1 Tide

The tidal character at Cuba's north coast is diurnal with small tidal range. Therefore the tide at Havana is characterized by mild conditions. The average tidal amplitude is 0.31 m and the spring tide is 0.61 m [NOAA Tides and Currents, 2015]. This value is included within the data of surge levels given by the meteorological institute [Meteorological Institute, 2015].

3.4.2 Storm surge

Storms elevate the water level causing a so called storm surge. The main mechanisms causing the storm surge are wind setup, wave setup and regional low atmospheric pressure. Especially hurricanes are accompanied by a larger storm surge [Meteorological Institute, 2015]. There is a difference in the offshore storm surge and the onshore storm surge, as additional setup occurs near the coast. This is wave setup due to shallow water wave phenomena (i.e. shoaling, refraction, etc.) and wave breaking. The combined tidal water level and surge surge component are used to calculate overtopping. The extreme sea level in figure 3.2 are located offshore and include the effects of the tide and are obtained from the meteorological institute [Meteorological Institute, 2015]. From the figure it can be deduced that a 50 year return period corresponds to a water level elevation of 1.67 m and a 250 year return period corresponds to a water level elevation of 2.19 m.

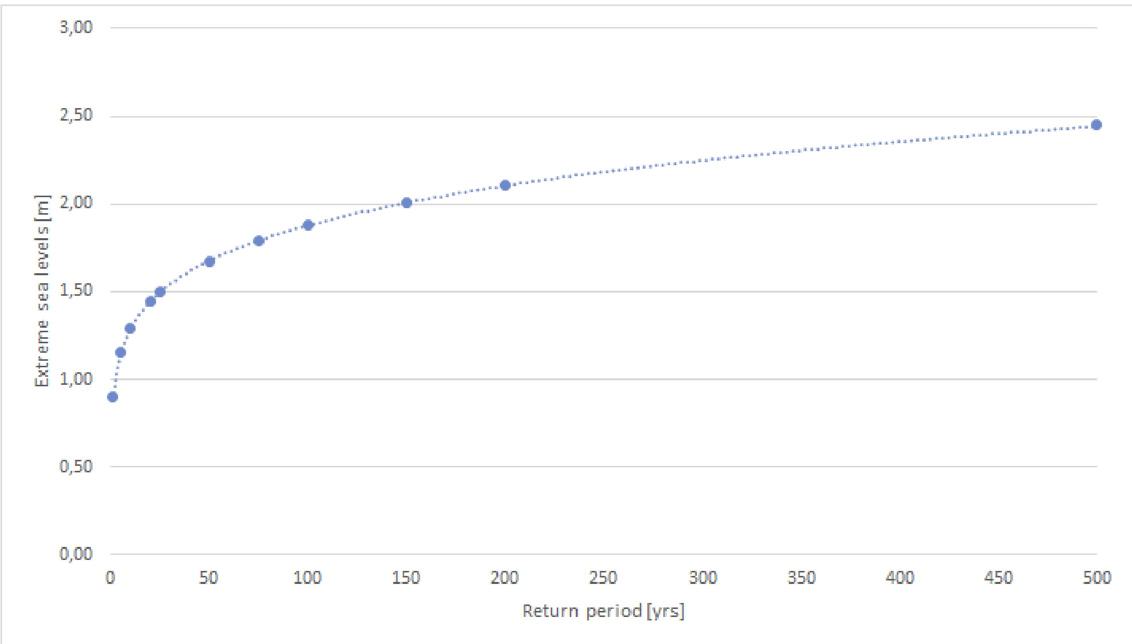

Figure 3.2: Extreme sea levels as a function of return period

3.4.3 Relative sea level rise

As a result of climate change the sea level is rising. According to the IPCC [Church et al., 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."

Therefore, regional influences have been investigated by the environmental agency in Cuba in a project called macro proyecto. This is a collaboration of more than 50 institutes and 500 specialists. According to their findings, the rising of the sea level is slightly compensated by tectonic uplift of the coast in front of Havana. As stated in section 2.2.1 tectonic movement causes the Cuban island to tilt around its west-east axis, causing the northern part to rise. The combined effect is expressed in the relative sea level rise.

Combining all scenarios for climate predictions by the IPCC [Church et al., 2013] , the environmental agency foresees a relative sea level rise between 0.08 m and 0.27 m by the year 2050 upto 0.59 m by the year 2100, based on a mean temperature rise between 1.6 and 2.5 $^{\circ}C$ in 2100 [Centella et al., 2009]. For the design life of 50 years, the value of 0.27 m relative sea level rise is used.

3.4.4 Total water level elevation

The total water level elevation (ζ) during design events equals MSL+1.95 m. Here the relative sea level rise for 2050 is added to the storm surge, including the tide.

3.5 Statistical analysis

In order to determine the event on which the design should be based, the design event, a combination of loading variables has to be used. This event has to fulfil the design criteria that it has a 50 year return period. A conservative approach would be to use for all individual variables the values corresponding to the individual 50 year return period. In the following, this will be called the 'zero combination'. This, however, implies a fully dependent correlation between the variables. A better, more realistic, combination of these values for the design event leads to a more economic solution. This section describes the method to find this more realistic combination of variables, keeping the safety level on the high standard, by using knowledge of the relation between these variables.

3.5.1 Variables

The most important variables of the hydraulic loading which control the design of the solution are the significant wave height H_s and the water level elevation ζ. For both the variables the offshore values are given as a function of the return period. For example, for the 50 year return period H_s equals 9.2 m (see figure 3.1) and ζ equals 1.95 m.

The wave height is given offshore, measured or computed, but the forcing of the proposed solution is nearshore. This requires a shallow water transformation, for example by the mathematical model SWAN. This requires careful attention between the different values. The used specification in this report is as follows: the relation between the occurrence of the variables is established offshore. These values are then used in the shallow water transformation leading to the design parameters.

3.5.2 Choosing the governing combination of H_s and ζ

Appendix C elaborates on three possible relations between H_s and ζ from a statistical point of view: *I* full dependence, *II* full independence or *III* a partial correlation. Using the latter option, an interval of probabilities can be computed. In 1977 Ditlevsen found a formula to find these probabilities. Ditlevsen [1977] stated that when the correlation ρ is known, an interval of probabilities for a combined event can be found based on the probabilities of the individual events, see appendix C. The correlation for the given data set equals 0.44. As the data set consists of a limited number of events, the correlation might be biased and for a conservative approach the use of a confidence interval is necessary. Therefore a correlation of 0.8365 is used, as this corresponds to the upper bound of a 98% confidence interval. If a more extensive data set would be used, the correlation could be determined more accurately.

Table 3.2 provides an overview of different combinations of H_s and ζ that lead to a combined return period of 50 years, as well as the 'zero combination'. It can be seen that this 'zero combination' of a $\frac{1}{50}H_s$ and a $\frac{1}{50}\zeta$ has a combined return period of 92.6 years according to the statistical analysis.

		Wave height		Extreme water level	
Combination	Combined RP [yrs]	H_s [m]	RP [yrs]	ζ [m]	RP [yrs]
0	92.6	9.14	50.0	1.95	50.0
1	50.0	6.52	7.5	1.95	50.0
2	50.0	7.80	18.9	1.88	40.0
3	50.0	9.14	50.0	1.50	7.5
4	50.0	8.84	40.0	1.70	18.9
5	50.0	8.44	30.0	1.80	28.6

Table 3.2: Governing combinations of a significant wave height and water level

3.6 Conclusion

No morphologic change or currents are taken into account. For a return period of 50 years the significant wave height amounts 9.2 m. The extreme water level (including climate change) with a 50 year return period equals 1.95 m. In order to have a storm event with a 50 year return period a combination of wave height and extreme water level (including climate change) is needed, as explained in section 3.5. The governing combinations are listed in table 3.2. These are used to calculate the overtopping in chapter 4 in order to find the design event. The 'zero combination' of a 1/50 H_{sig} and 1/50 ζ is also included in the table, for an assumed correlation of 0.8365 this combination has a combined return period of 92.6 years according to the statistical analysis.

Chapter 4

Hydraulic boundary conditions

In this chapter the hydraulic boundary conditions are described. All combinations of wave height and surge level established in chapter 3 have a return period of 50 years, but these combinations do not have the same effect on nearshore structures. In order to evaluate these differences all combinations are transformed into nearshore values in section 4.1 using a SWAN model. The results are used to calculate the wave overtopping with the method of EurOtop Team [2007] in section 4.2. The overtopping, which is the main cause of flooding of the study area, is finally used to determine which combination is governing. In section 4.3 the governing wave directions are used to select a representative wave height per section. In section 4.4 the governing combinations of wave height and surge level are evaluated to determine the hydraulic loads on the seawall by using empirical relations and physical model test results.

4.1 Wave transformation with a SWAN model

This section describes the wave transformation that is made using a SWAN model. SWAN is a numerical, third-generation wave model that can be used to compute wave transformation from deep water to nearshore values, based on a wave action balance. It does not solve for individual waves, but transforms an offshore wave spectrum to a wave spectrum nearshore. Section 4.1.1 describes the set up of the SWAN model that is used for the wave transformation, which basically corresponds to the information in the SWAN command file. Section 4.1.2 shows the results of the wave transformation. In appendix D background information is given about the SWAN model that is used. It contains a model description, the SWAN command files used and additional results.

4.1.1 Model input

Several runs were performed for the load combinations determined in section 3.5 in order to find the governing combination. This governing combination is imposed from different wave directions, in order to find the governing direction. This section describes the set up of the SWAN model that is used for the wave transformation, which basically corresponds to the information in the SWAN command file (see section D.2 in the appendix). The most important input parameters will be discussed here.

First of all SWAN is run in a stationary two-dimensional mode, since all parameters are time independent. A nesting approach is employed with a large coarse grid offshore, a finer nested grid at Havana Bay and a very fine nest of the study area. All grids are rectilinear, using Cartesian coordinates (translated from UTM coordinates). The largest grid is 72 x 27 km with a cell size of 730 m. The first nested grid is 12x12 km with a cell size of 120 m. Finally, the last nest is 5.6 x 3.2 km with a cell size of 19 m.
Based on bathymetric data from the Office of the Historian a bottom file was generated separately for each grid, ensuring that the bottom grid exactly matches the computational grid. Land points were filtered out in order to end up with a bottom file that merely describes the wet grid cells.
A wind speed of 25.0 m/s from the predominant wind direction (different for each run) is used, as this is the maximum wind speed that occurred in the last 36 years (see appendix A. All the physical phenomena that are included in SWAN, as mentioned in section D.1 in the appendix, are included in the computation.

A JONSWAP spectrum is imposed at all offshore wave boundaries with a peak enhancement parameter of 3.3. For all runs a peak period of 12.0 s is used. The directional spreading is defined by a standard deviation of 30°. 36 meshes are defined in the directional space, which results in 10° per mesh if all directions are included (which is the case here). Frequencies between 0.03 Hz and 1.0 Hz are included, as is advised for hurricane areas [SWAN team, 2015].
It is important to note that the offshore boundary is placed on the north as well as on the east and the west, so waves can enter the domain from all boundaries (the same incident wave angle is used at all boundaries). In doing so, a small inaccuracy occurs, as the offshore wave climate is also imposed on the shallow areas at the east and west boundaries. Closing the east and west boundaries, however, would lead to more severe inaccuracies, due to the shadow zone that would develop, influencing Havana Bay's wave climate. By employing this approach, the offshore wave climate will definitely not be underestimated.

4.1.2 SWAN results

Wave transformations were computed for all combinations of wave height and surge level established in section 3.5. The results can be seen in appendix D. Table 4.1 gives the maximum wave height after the wave transformation for all combinations in the last grid point in front of the seawall. This is the wave height directly in front of the wall, since SWAN calculates the wave height in the grid points. Based on the corresponding overtopping, combination 2 is selected as governing combination in section 4.2. The resulting wave climate for combination 2 in the finest grid is displayed in figure 4.1.

Combination	ζ [m]	Offshore H_s [m]	Nearshore H_s [m]
0	1.95	9.14	4.35
1	1.95	6.52	4.08
2	1.88	7.80	4.27
3	1.50	9.14	4.35
4	1.70	8.84	4.37
5	1.80	8.44	4.32

Table 4.1: Wave transformation for different combinations of H_s and ζ

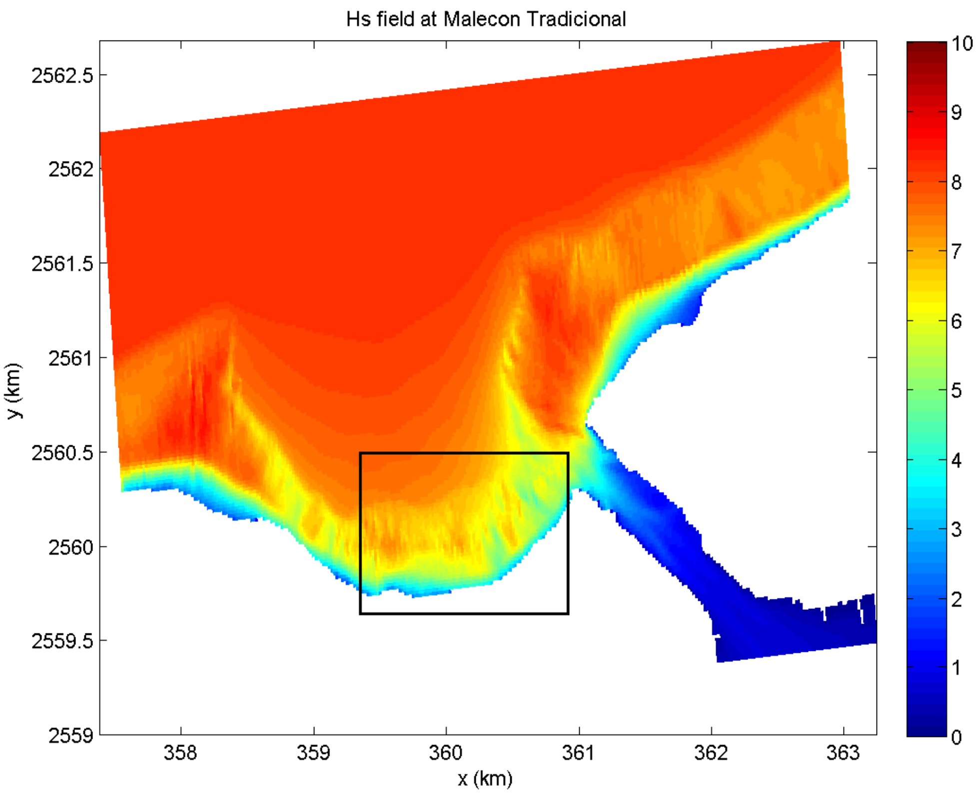

Figure 4.1: Resulting wave field in Havana Bay for combination 2

4.2 Wave overtopping

As noted in the introduction wave overtopping is the most important cause of flooding of the study area. It is caused by a combination of a water level and a significant wave height. In order to be able to choose the governing combination of a significant wave height and a water level, the overtopping corresponding to this combination must be calculated. This can be done in three ways, with formulas from the EuroTop Overtopping Manual [EurOtop Team, 2007], by calculating it with a numerical model or by physical model tests. In this section the EurOtop Overtopping Manual is used.

4.2.1 Overtopping formulas

In the EurOtop Overtopping Manual it is stated that two types of wave conditions can occur at the seawall, non-impulsive and impulsive conditions. Non-impulsive or pulsating conditions occur when waves are relatively small in relation to the local water depth and of lower wave steepness. Impulsive conditions occur on vertical or steep walls when waves are larger in relation to local water depths. Under these conditions, some waves will break violently against the wall.

To determine whether there are non-impulsive or impulsive wave conditions an impulsiveness parameter h_* is defined.

$$h_* = 1.35 * \frac{h_s}{H_{m0}} * \frac{2\pi * h_s}{gT_{m-1,0}^2} \tag{4.1}$$

For $h > 0.3$ non-impulsive conditions dominate the wall. Impulsive conditions occur when $h < 0.2$. Since non-impulsive conditions result in less overtopping, these are preferable.

After this, the overtopping itself can be determined. The general formula for finding the overtopping discharge is as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = a * \exp\left(-b * \frac{R_c}{H_{m0}}\right) \tag{4.2}$$

This is an exponential function with the dimensionless overtopping discharge $\frac{q}{\sqrt{gH_{m0}^3}}$ and the relative crest freeboard $\frac{R_c}{H_{m0}}$. In which q is the mean overtopping discharge in $m^3/m/s$, H_{m0} is the significant wave height in m and R_c is the crest freeboard in m. The constants a and b differ for varying structures.

Assuming a plain vertical wall, eq. 7.4 for deterministic design and non-impulsive conditions from the Overtopping Manual applies with $a = 0.04$ and $b = 1.8$, which is valid for $0.1 < \frac{R_c}{H_{m0}} < 3.5$.

For impulsive conditions eq. 7.7 from the Overtopping Manual should be used. However for values of $h_* \frac{R_c}{H_{m0}} < 0.03$, eq 7.7 is not valid any more. An adjustment downwards of the prediction may be possible assuming that waves break before they hit the wall, described in formula 7.9. For $0.02 < h_* \frac{R_c}{H_{m0}} < 0.03$ there is a transition between equations 7.7 and 7.9, in this report there is assumed that an linear interpolation is possible. Eq. 7.7 from the Overtopping Manual corresponds to equation 4.3. Eq. 7.9 from the Overtopping Manual corresponds to equation 4.4.

$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 2.8 * 10^{-4} * \left(h_* * \frac{R_c}{H_{m0}}\right)^{-3.1} \tag{4.3}$$

This formula is valid for $0.03 < h_* \frac{R_c}{H_{m0}} < 1.0$.

$$\frac{q}{h_*^2\sqrt{gh_s^3}} = 3.8 * 10^{-4} * \left(h_* * \frac{R_c}{H_{m0}}\right)^{-2.7} \tag{4.4}$$

This formula is valid for $h_* \frac{R_c}{H_{m0}} < 0.02$ and broken waves.

4.2.2 Overtopping computations

Table 4.2 shows the parameters that are used in the overtopping computations. The depth in front of the wall (h_s) and the crest freeboard (R_c) add up to the total height of the wall (from toe to crest) of 5.28 m. For the wave height (H_{m0}) the significant wave height at the toe of the wall is used. This value differs per combination. $T_{m-1,0}$ is the wave period calculated from the first negative moment of the spectrum. For all combinations $T_{m-1,0}$ is equal to 10.9 s.

Combination	R_c [m]	h_s [m]	H_{m0} [m]	h_* [-]	formula
0	2.01	3.27	4.35	0.018	7.9
1	2.01	3.27	4.08	0.019	7.9
2	2.08	3.20	4.27	0.017	7.9
3	2.46	2.82	4.35	0.013	7.9
4	2.26	3.02	4.37	0.015	7.9
5	2.16	3.12	4.32	0.016	7.9

Table 4.2: Parameters values used in the overtopping computations

The results of the overtopping computations are summarised in table 4.3. As can be seen from the table, combination **2** is governing in means of overtopping (the 'zero combination' is not taken into account, as its return period is not 50 years, as explained in section 3.5). Therefore it is concluded that combination 2 is the governing combination of H_s and ζ for a fixed return period of 50 years. This report assumes that applying this combination as the governing load on hydraulic structures is justified, as the wave heights in terms of hydraulic forcing do not significantly differ and also multiple directions have to be considered for the governing load.

Combination	ζ [m]	Offshore H_s [m]	Nearshore H_s [m]	Overtopping [L/s/m]
0	1.95	9.14	4.35	946
1	1.95	6.52	4.08	761
2	1.88	7.80	4.27	808
3	1.50	9.14	4.35	540
4	1.70	8.84	4.37	695
5	1.80	8.44	4.32	757

Table 4.3: Overtopping for different combinations of H_s and ζ

4.3 Representative wave height per section

In section 5.1 the Malecón seawall is divided into sections based on the structural assesment. The conclusion of this section is presented in table 4.4.

Section	1	2	3	4	5
Location	Calle Marina - Parque Maceo	Parque Maceo - Calle Lealtad	Calle Lealtad - Galiano	Galiano - Crespo	Crespo - Prado
Length section [m]	270	520	420	250	390
Characteristic crest height [m]	3.98	4.32	3.99	3.96	3.94

Table 4.4: Sections of the Malecón Tradicional as determined in section 5.1 of chapter 5

Now that the governing load combination has been determined, the wave heights can be determined per section. This is important input for designing alternative solutions. Selecting the maximum wave height could lead to an overestimation of the wave height for the majority of the section. Using the mean wave height would lead to too much overtopping at the locations with highest waves. Therefore a representative wave height is selected per section. Figures 4.2, 4.3 and 4.4 show the resulting wave heights at the location of Malecón seawall for wave directions north, north north west and north west respectively.

For section 1 waves coming from the north lead to the largest wave height in front of the wall. The maximum value of 3.61 m is used as representative value. As the mean and maximum values do not deviate much, this does not lead to a large overestimation.

For section 2 waves coming from the north again lead to the highest wave height in front of the wall. However, in this section the wave heights in front of the wall have a large variance. If one representative value would be selected, the wave height would be overestimated for the majority of the section. Therefore it was decided to choose a representative value for section 2 and define a special section 2* that will have extra protection. This is section will be designed for a representative wave height of 3.87 m, that is determined by taking mean value plus the standard deviation of the wave height in this section. For the rest of section 2 the maximum wave height of 3.12 m is selected as representative wave height.

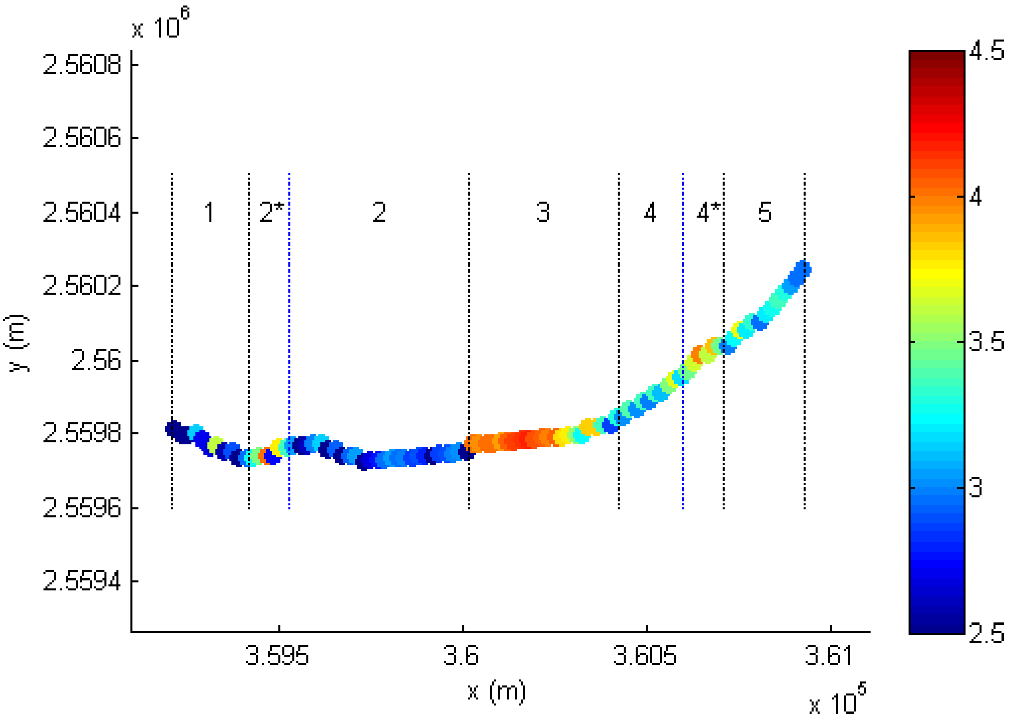

Figure 4.2: Resulting wave heights at the Malecón seawall for wave direction north

For section 3 waves coming from the north north west lead to the largest wave height in front of the wall. The maximum wave height of 4.19 m is selected as representative wave height. As the mean and maximum values do not deviate much, this does not lead to a large overestimation. The wave height at this section is much larger than that of adjacent sections, due to the steep bathymetry in front of the section and the lack of a natural rock berm.

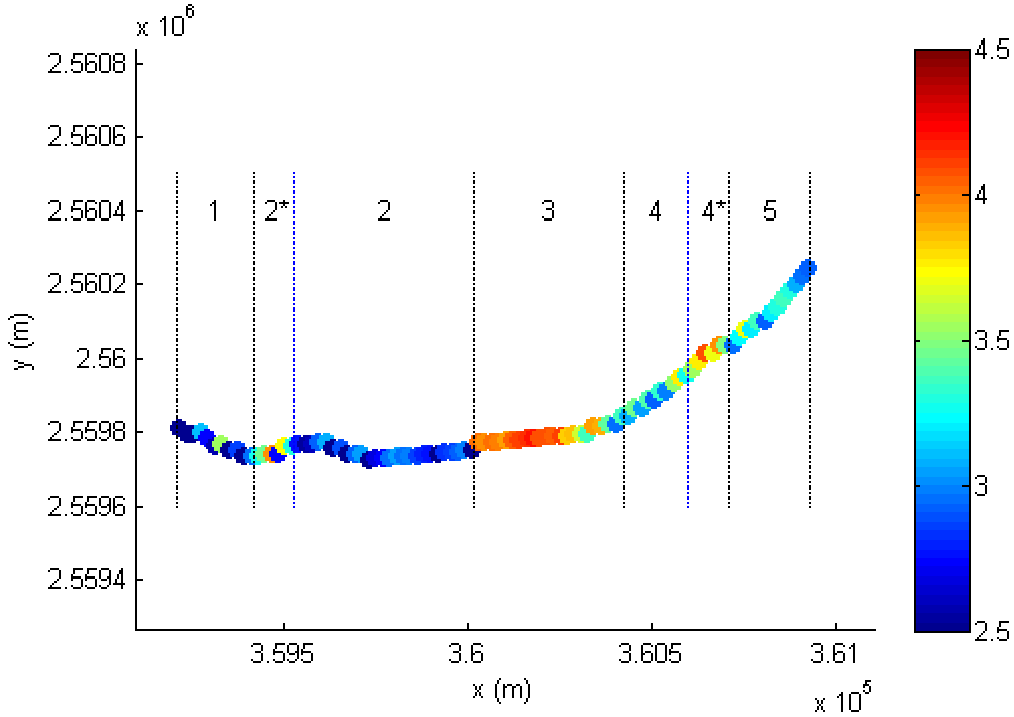

Figure 4.3: Resulting wave heights at the Malecón seawall for wave direction north north west

For section 4 and 5 waves coming from the north west lead to the largest wave height in front of the wall. Again, in these two sections the wave heights in front of the wall have a large variance. One representative value could be selected for both sections, except for a small part that suffers from a large wave load. Therefore it was decided to choose one representative value for sections 4 and 5 and another special section 4* is defined that will have extra protection. This is section will be designed for a representative wave height of 4.11 m, that is determined by taking mean value plus the standard deviation of the wave height in this section. For the rest of sections 4 and 5 the mean wave height of 3.34 m is selected as representative wave height.

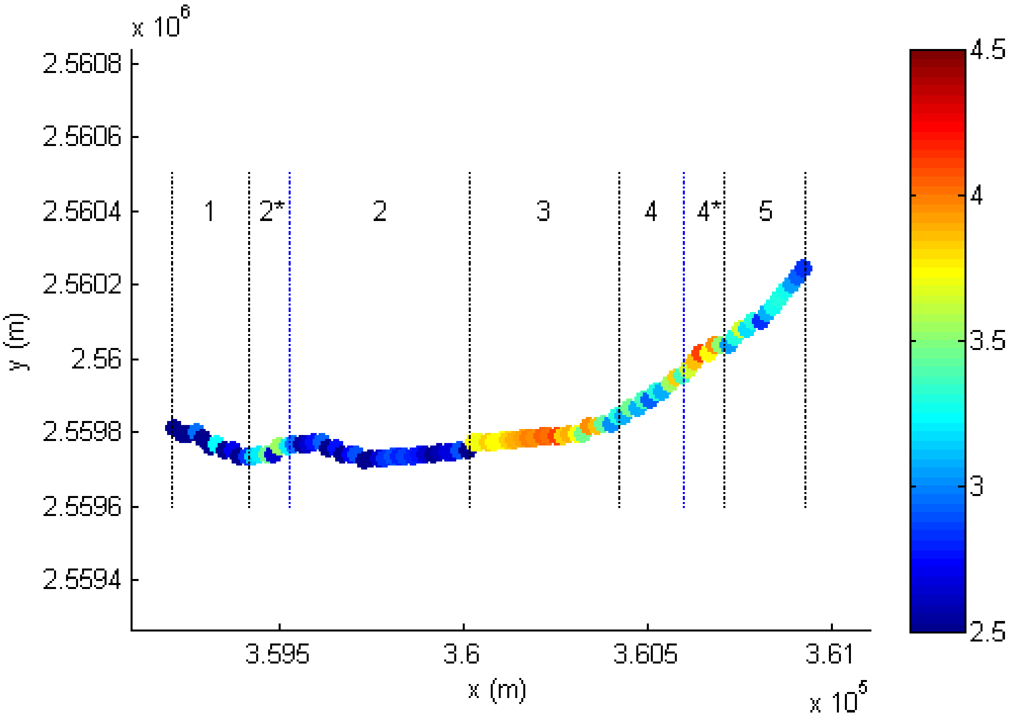

Figure 4.4: Resulting wave heights at the Malecón seawall for wave direction north west

Table 4.5 displays the selected wave heights per section.

Section	1	2	2*	3	4	4*	5
R_c [m]	2.10	2.44	2.44	2.11	2.07	2.07	2.07
Wave direction	N	N	N	NNW	NW	NW	NW
$H_{s,rep}$ [m]	3.15	3.12	3.87	4.19	3.34	4.11	3.34
q [L/s/m]	345	427	384	754	624	741	624

Table 4.5: Representative wave height per section

4.4 Forcing on the Malecón seawall

In order to make a structural analysis of the Malecón it is necessary to convert the results from the hydraulic models to loads on the seawall. First this section will describe a physical model test that has been done to evaluate the pressures on the Malecón. The results of this physical model test will be compared with the results of Goda's method to convert wave conditions to pressures to evaluate the governing loads.

4.4.1 Physical model test

For the Malecón a physical model test has been done to evaluate the wave overtopping and pressures on the wall for several wave conditions. This test was executed by the University Centre for the Forecast and the Prevention of Major Hazards Federico II from the University of Naples and the University of Salerno, and requested by Prof. L. Cordova [Buccino et al., 2013]. For the coupling between the hydraulic and structural models especially the pressures on the seawall are relevant.

For the test a model with a scale of 1:30 has been built of the seawall and bathymetry. The results obtained by the model tests have been converted to the so called prototype scale, which is the scale of the true situation (the prototype is still a simplified version of reality). The dimensions of the slope of the prototype can be seen in figure E.2 in appendix E. The used water levels and locations of the transducers are found in figure E.3 in appendix E.

4.4.2 Method of Goda

There are several methods of calculating the loads on the wall theoretically. The selection of the method that can be applied depends on the type of waves that hit the wall. These types are non-breaking waves, breaking (plunging) waves with a vertical front and and breaking waves with a large air pocket. In the case of the Malecón plunging waves have to be considered. For both types of plunging waves, the method of Goda is used to calculate the pressures on the wall. For the project the methodology as described in the Coastal Engineering Manual Part IV [U.S. Army Corps Engineers, 2003] is used. Goda's equations are empirical and have been calibrated by prototype vertical breakwaters. Therefore there is an uncertainty and bias when the horizontal forces and moments are considered. For the pressures this is not necessary.

There are two Goda methods applicable for impermeable vertical walls: *Goda Formula for Irregular Waves (Goda 1974; Tanimoto et al. 1976)* and *Goda Formula Modified to Include Impulsive Forces from Head-on Breaking Waves (Takahashi, Tanimoto, and Shimosako 1994a)* [U.S. Army Corps Engineers, 2003]. The modified Goda formula will be most suitable for the situation near the Malecón, because of the impulsive nature of the waves. The pressure distribution calculated by the Goda formula's looks like the one in figure E.1 in appendix E. This pressure distribution only includes the hydrodynamic pressure not the hydrostatic pressure. For the entire computation with the Goda equations see appendix E.

4.4.3 Comparing physical model test with Goda's method

The results of the physical model test have been compared with the pressures computed with the method of Goda. The parameters used to compute the pressures, are the ones of the prototype by the physical model test. This to keep the values comparable. The pressures found with Goda are interpolated so they can be compared with the values found at transducer 0, which is approximately at storm surge level. The validity of the Goda formulas has been checked for the parameters of the physical model test as can be seen in appendix E. The pressures found by the physical model test have been corrected for the use on a vertical wall. A factor of two was used as suggested by Kortenhaus [Kortenhaus et al., 2003] for impulsive loads.

Figure 4.5 shows the pressure distribution for the parameters of test 11 as given by the Goda method, the one for a $\frac{1}{250}$ wave in the physical model test and a wave with a probability of exceedance of 10%. The pressure distribution for a $\frac{1}{250}$ wave, will be used, because the 10% values are too low to be representative. As can be seen in figure 4.5, most pressures evaluated by Goda are bigger than for the physical model test except for the peak pressure. For most tests this peak pressure is strongly underestimated. A table with the peak pressures for 4 governing tests including the used parameters can be found in appendix E. According to the article 'Breaking wave loads at vertical seawalls and breakwaters' [Cuomo et al., 2010], Goda's method is an underestimation especially for pressures from impact loads, but a quite good method to evaluate quasi-static loads. The impact load acts on the wall at SWL, so that is probably why Goda is not safe for this point on the wall.

4.4.4 Evaluation of the governing pressure distributions

To make a complete pressure distribution from the top to the toe of the wall, both the values from the physical model test as the values given by Goda's method will be used. The physical model test only gives the pressures at four specific locations and not at the top and toe of the wall. For the bottom part of the wall Goda gives a conservative approximation of the distribution. Therefore Goda will be used for this part. For the peak pressure the values of the physical model test will be used. According to [Cuomo et al., 2010] the impact pressure distribution does have a peak value at the SWL as can be seen in figure E.4 in appendix E. So it is assumed that the pressure decreases above SWL and therefore decreases from the top value of the physical model test to the top of the wall. For the top of the wall there is no measured value available, so there the result of Goda

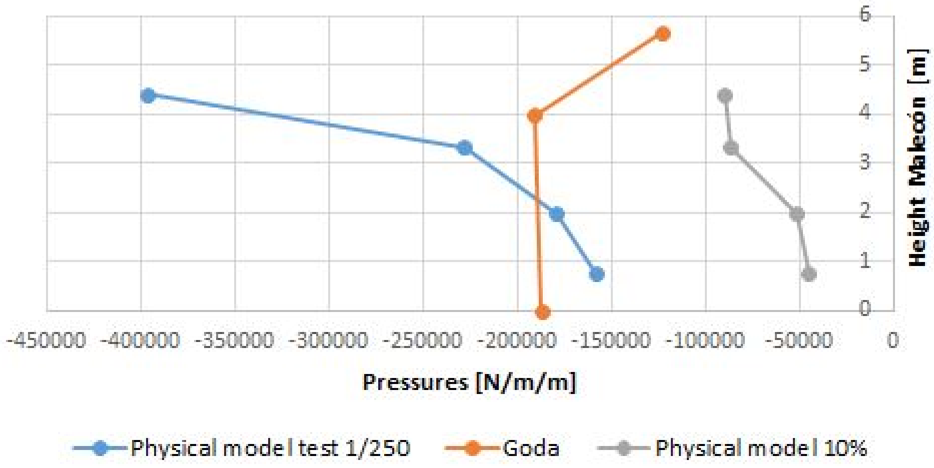

Figure 4.5: Pressures for test 11

defines the pressure. This could be an underestimation, but as more accurate data is not present for this point of the wall, this is the most accurate value that could possibly be obtained.

Return period of 50 years

From the output of SWAN it was concluded that for a return period of 50 years the governing combination is a surge level of 1.88 m and a significant offshore wave height of 7.8 m, which is combination 2. The parameters of test 11 of the physical model test best approach this combination of loads with a surge level of 1.73 m and an offshore wave height of 8 m. The pressure distribution on the current seawall that follows is given in figure 4.6 by the line named 'combination'.

Ultimate Limit State

For the analysis of the seawall not only the pressure distribution of a storm condition with a return period of 50 years are of interest. In the recent past hurricane Wilma has caused severe damage on the Malecón seawall. Therefore this load condition will be used as an Ultimate Limit State (ULS). The conditions of hurricane Wilma have a return period of 340 years. The probability of exceedance for a storm with these conditions is 22% during a design life of 50 years, considering sea level rise. This has been determined with the use of equation 4.5 with f being the storm frequency and a lifetime of the structure of n years.

$$\frac{p}{100} = 1 - (1 - f)^n \tag{4.5}$$

The pressure distribution on the current seawall for the ULS condition is found in figure E.5 in appendix E.

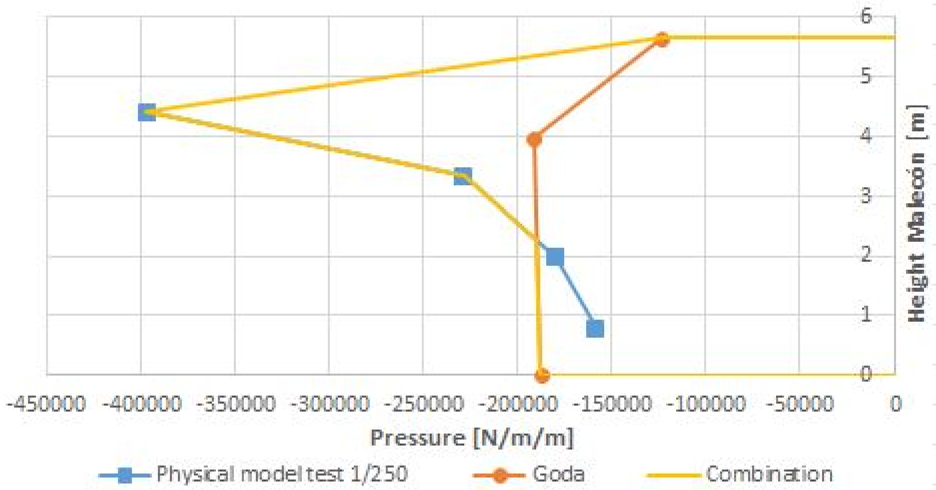

Figure 4.6: Pressures for $T_{return} = 50$ years on the current seawall

Loads for a curved wall and for an elevated wall

For a curved wall the uncorrected pressures from the physical model test can be used, without dividing the pressures with a factor 2 as suggested by Kortenhaus [Kortenhaus et al., 2003]. For an elevated Malecón seawall to the maximum crest height as described in the design criteria, the pressures will be as given in figures E.6 and E.7 in appendix E.

4.5 Conclusion

In order to find the hydraulic boundary conditions for each section, the offshore wave conditions were transformed into nearshore conditions using a SWAN model. Runs were performed for several load combinations in order to determine the governing combination. This governing combination was imposed from different wave directions, in order to find the governing direction. The results were used to calculate the corresponding wave overtopping. Thus, combination 2 was selected as the governing combination of wave height and surge level. The design surge level equals 1.88 m, the offshore wave height equals 7.80 m and the maximum nearshore wave height equals 4.27 m. Moreover, the hydraulic boundary conditions were determined per section, as displayed in table 4.5. The hydraulic loads on the seawall have been determined by using a combination of the Goda method and results of a physical model test. It can be concluded that the empirical models all underestimated the impulsive impact load, compared to the physical model test.

Chapter 5

Initial structural assessment of the Malecón seawall

In order to carry out a realistic structural analysis, the structural aspects of the Malecón have been assessed. In section 5.1 the differences in condition and structure of the Malecón Tradicional have been outlined in order to divide the wall into several sections. In section 5.2.1 the schematisation of the wall will be presented. In the last parts of this section the different material properties which are needed to compute the FEM are discussed.

5.1 Characterization of the Malecón seawall

The Malecón seawall has been constructed over a period of 60 years and differs considerably along the total length. Also the concrete of which the Malecón was made is in different stages of deterioration. In order to model the behaviour of the wall and the solutions to reduce the flood risk, the Malecón will be divided into several sections. The division of these sections is based on structural conditions.

Before the start of this project the Malecón seawall had already been broken down into different sections, see figure 5.1. The track concerning this project was divided into three sections by the Meteorological Institute [Meteorological Institute, 2012]. Section 1 being from Calle Marina upto Parque Maceo, section 2 from Parque Maceo upto Calle Lealtad and section 3 from Calle Lealtad upto Prado. These sections are divided considering the average crest height and the existence of a 'natural' berm in front of the sea wall, as can be seen in 5.1. The new division of the wall keeps the previous division, but two more sections are introduced, see figure 5.2. The location of the sections can be seen in figure 5.3. Next each section will be discussed separately. The dimensions of each section is based on the information given by the Meteorological Institute [Meteorological Institute, 2012]. With this data cross sections have been sketched for each section, which can be found in appendix F.

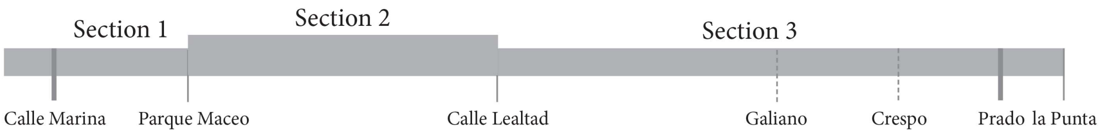

Figure 5.1: Previous division of the Malecón sea wall

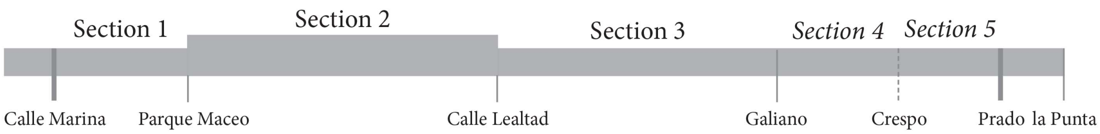

Figure 5.2: New division of the Malecón sea wall

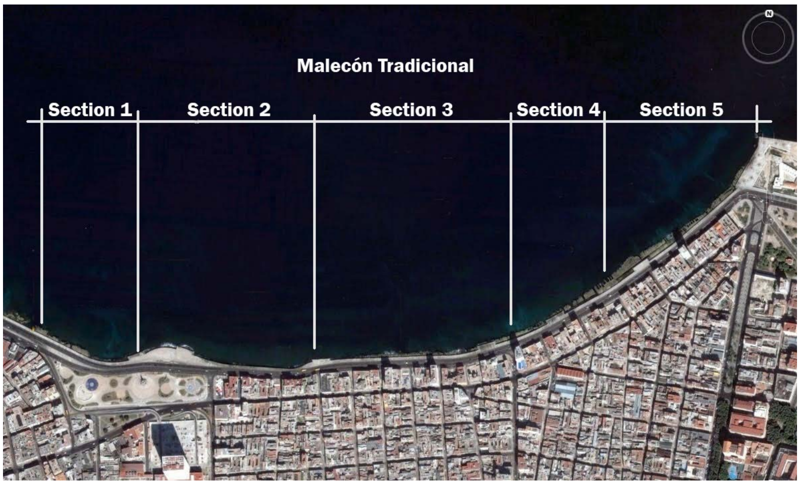

Figure 5.3: Location of the determined sections

5.1.1 Section 1

Section 1 does not change in comparison to the division made by the Meteorological institute. It consists of the wall between Calle Marina and Parque Maceo. This part of the wall has an average height of MSL +3.98 m and has a small natural rock berm in front of the wall. The wall itself is not in the worst condition, but it definitely needs some work done. The concrete has damage on the surface of the wall, as can be seen in figure 5.5.

5.1.2 Section 2

Section 2 will also be similar to the division made by the Meteorological institute. It runs from Parque Maceo to Calle Lealtad. The wall has a higher average crest height of MSL +4.33 m. This part of the wall does not have a natural berm. In this area there used

Figure 5.4: The seawall in section 1

Figure 5.5: Damage on the surface of the wall

to be an old harbour, but now it is closed off and filled with material. Hence, the wall is constructed on reclaimed land.

Due to the absence of the berm there is a lot of overtopping in this part and several parts of the wall have been damaged during hurricane Wilma in 2005. At two locations the wall has been repaired with the help of Bordstein-Ries, a German company. They used a new type of sand-like material that should be more resistant to salt water. However, the recent repairs seem to have been superficial, because the wall has been damaged again.

This section is in a bad condition, due to cracking and scour. This entire section of the wall has scour protection in the form of a concrete beam at the foot of the wall, as scour problems due to the heavy storms were observed at the toe of the wall. It is important to note, however, that the connection of the beam is not executed properly. Scour is still occurring at the interface between the beam and the wall. Eventually this scour will cause instability of the wall, if no mitigation measures are taken.

Figure 5.6: Current condition of the wall in section 2 showing cracking

Figure 5.7: Scour protection in section 2

5.1.3 Section 3

Section 3 is from Calle Lealtad to Galiano. The average crest height of this section is 3.97 m and a natural berm lies in front of the wall and is approximately three meters wide. The wall itself is in a relatively good condition as it has been recently repaired.

Figure 5.8: Change of crest height between sections 2 and 3

Figure 5.9: The sea wall in section 3

5.1.4 Section 4

Section 4 is from Galiano to Crespo. The average crest height of the section is 3.97 m, which is similar to section 3. This section of the wall differs from the previous section as it has not been repaired recently and is therefore in a worse condition. There is also a natural berm in this section, but the berm was also used as a place to bath in the beginning of the 20^{th} century. In order to create these baths, little pools were cut out of the berm and buildingswere placed above these pools. The houses were destroyed long ago, but the pools are still visible in the berm. The berm is about ten meters wide. At the part where there are no artificial pools, the berm can act like a ramp (considering the dimensions and geometry) increasing the amount of overtopping.

5.1.5 Section 5

This part of the wall is very old. It has been repaired, but only superficially. The average crest height is 3.94 m and a berm is present. Additionally there are very long vertical cracks in this old part of the wall and the concrete seems to be made from finer aggregates with a smaller D_{50} than the rest of the wall.
At the end of this section lies Castillo de la Punta. This part suffers a lot from scour, because there is no berm.

Figure 5.10: Pools in front of the Malecón in section 4

Figure 5.11: Condition of the sea wall in section 4 showing the scour problem

Figure 5.12: The sea wall in section 5

5.1.6 General observations

Along the entire length of the study area, the seawall is a mass concrete structure without reinforcement. Due to severe weather conditions the wall has been destroyed locally. In order to restore the wall new concrete was poured onto the existing damaged areas. At these locations the wall will be relatively weak because of the cold formed connections, between the old concrete and the new concrete. When the wall fails during storms, it is likely to happen at these positions.

5.2 Structural analysis with an ANSYS model

For the structural analysis of the Malecón seawall the program ANSYS will be used. ANSYS is a professional program that allows an engineer to use the finite element method when evaluating a structure in search of a field quantity, such as the displacements. A detailed description of ANSYS is given in appendix H.

5.2.1 Schematization of model

Some assumptions have to be made, before modelling the Malecón seawall in ANSYS. It was decided to make a 3D model, but measures have been taken to make it a 2D analysis as much as possible. With ANSYS a linear finite element analysis will be dome in order to compute the deformations and stresses in the wall. The found deformations and stresses will be interpreted in order to determine if the structure can resist these deformations and stresses or not.

Boundary conditions

Due to the method of construction of the wall, discussed in appendix G, the wall itself will be modelled as one block in ANSYS. In figure 5.13, a general free body diagram of the wall is given. The concrete body is used to model the support of the wall. Behind the seawall a combination of rock and concrete is placed as support for the wall as is discussed in appendix G. This support also ensures the stability of the seawall. The precise dimensions of each section are given in table 5.1. The different crest levels and height of the concrete body of each section are based on[Meteorological Institute, 2012]. Because there was no accurate data for the pressures on the wall the pressures from the physical model test have been used, as is discussed in 4.4. In these physical model tests a water depth of 1.7 is always taken, so therefore a water depth of 1.7 m is assumed for modelling the wall, even if this is not the actual case. For section 2 the actual water depth in front of the wall is 1.9 m. The difference in maximum water level can lead to slightly higher pressures, but this will not be substantial as the difference is only small. For the thickness of the concrete wall also no accurate data was available, so measurements were taken on site and a thickness of 0.8 m was assumed for all the sections. Given the construction method it is assumed the wall thickness is the same over the entire height.

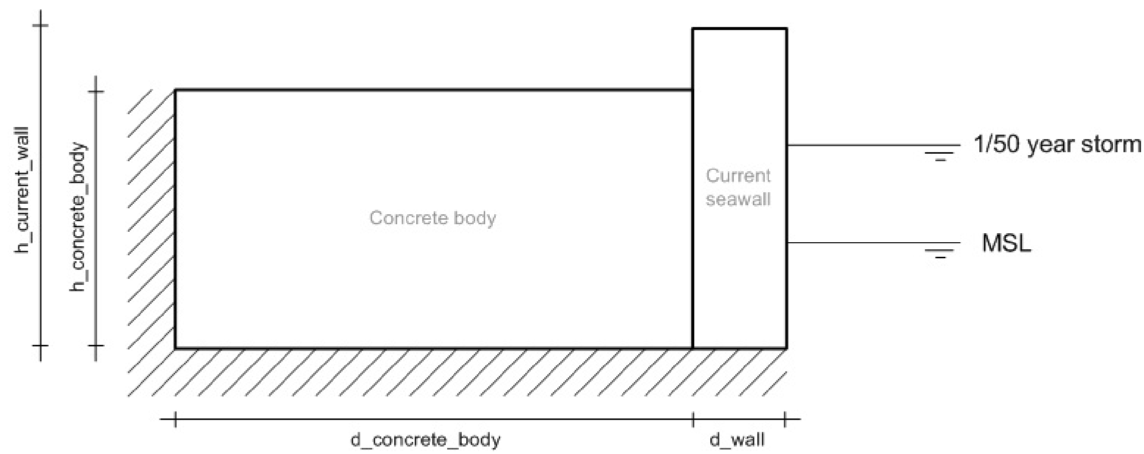

Figure 5.13: Free body diagram of the situation

	Section 1	Section 2	Section 3	Section 4	Section 5
h_current_wall [m]	5.68	6.02	5.69	5.66	5.64
h_concrete_body [m]	4.78	4.85	4.93	4.96	4.90
d_concrete_body [m]	10	10	10	10	10
d_wall [m]	0.8	0.8	0.8	0.8	0.8

Table 5.1: Dimensions of the sections

During extremely heavy weather circumstances the top part of the wall will be critical for failure. During the Wilma hurricane in 2005 several pieces of this part were sheared off. If the wall is damaged, it can be said that damages will generally occur in this top part itself or in the connection to it. When it was necessary, parts were reconstructed on the existing wall and a cold connection was formed, which is a weaker horizontal connection than before it was before. It can be concluded that this top part is the most critical and has suffered the most damage in the last decade.

Initially the wall will be modelled in 3D, but in longitudinal direction the wall will be constrained by a frictionless support. With this additional frictionless support the 3D analysis will approach a 2D analysis. Therefore the horizontal cracks will not be taken into account. In order to make the model as accurate as possible the concrete body of the road will also be put in the model. This will help to prevent the wall from moving for the most part, however it is important to note that this body is not incompressible and therefore needs to be modelled as well. An alternative to this solution would be to have a Winkler-bedding instead of modelling a concrete body, however it was decided that modelling the concrete body would be a better approximation of the reality.

In order to get results from ANSYS, supports need to be defined. It was decided to model the wall with a fixed support on the bottom, but it is important to note that this is an overestimation of the reality. The concrete of the wall was directly poured on to the existing ground. The ground consists of limestone rocks and is therefore an uneven foundation of the wall that will help prevent displacements. Moreover, the addition of the large concrete body behind the wall will help prevent any displacements and rotations at the bottom of the wall. Thus, it was decided to model this as a fixed support. The same argumentation holds for the horizontal support of the concrete body.

The concrete body will also be fixed, so horizontal displacement will be prevented. Before the Malecón seawall was built, the coast consisted of terraces, see figure 5.14 for a schematisation. The terraces consisted of the same limestone rock of which the ground is made. Due to the uneven raise of the foundation, the concrete body has a vertical fixed support.

It is not exactly known how the area behind the Malecón seawall was filled. Most likely it was a combination of large rocks and concrete. If this is modelled as a concrete body, it will probably overestimate the real stiffness. In order to compensate for this overestimation

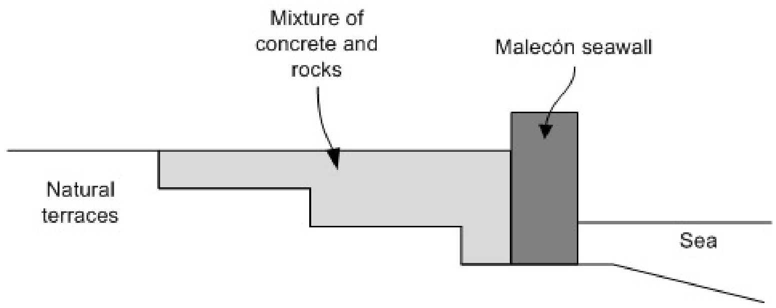

Figure 5.14: Schematisation of the actual situation

the concrete body will be modelled with a very low strength concrete.

Loads on the model

The model will be subjected to three basic load conditions. First the concrete body and the wall will have a gravity load. Second the wall will be subjected to the loads of the water, these can be split into two different load cases: the normal hydrostatic pressure and the additional wave pressure, which is calculated in 4.4. The combination of the loads can be seen in figure 5.15.

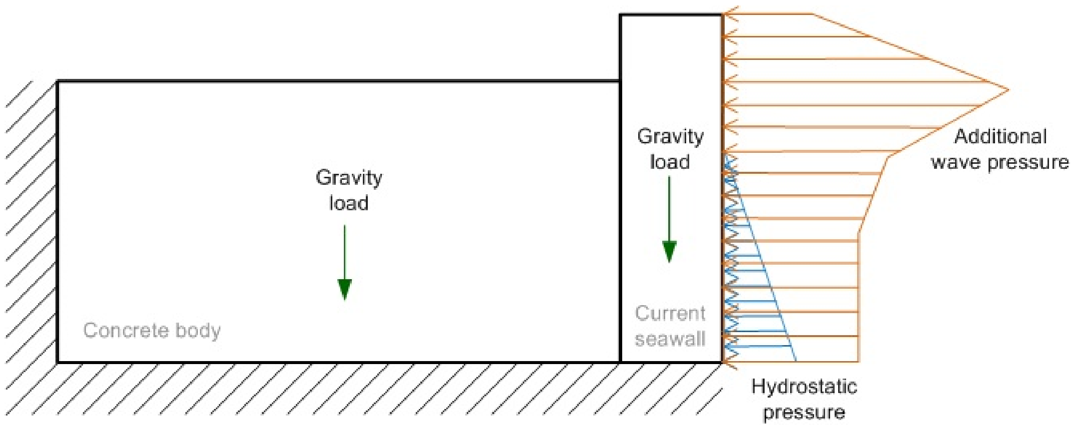

Figure 5.15: Schematisation of the loads in the ANSYS model

Verification of the element size

With the use of the output data ANSYS gives, as will be discussed in great length in section 5.2.2 and 5.2.4, an appropriate element size needs to be determined. The book Finite Element Modelling for Stress Analysis [Cook, 1995] states the element size of the mesh is accurate enough, if the output given by different element sizes is smaller than 5%. By changing the element size it was concluded that the difference in output was less than 5% for an element size of 0.1 m. Therefore all the ANSYS models will be run with an element size of 0.1 m. As is discussed in [Hoogenboom, 2012], quadrilateral elements are more accurate than triangle elements of the same size. Therefore it was decided to compute the models with quadrilateral elements. In figure H.4 in appendix H the element mesh of the model can be found.

5.2.2 Failure analysis

In order to determine the critical spots of the Malecón seawall it is necessary to evaluate all possible failure mechanisms. According to the fault tree in [Theunissen, 2004], there are quite a few failure mechanisms possible. The fault tree is presented in the appendix J in fig J.1. It can be seen that there are four categories in which the quay wall could fail. Each category consists of different failure mechanisms.

Total instability of the structure

For the total instability of the structure a few failure mechanisms can be thought of. According to the fault tree described in[Theunissen, 2004], the most important failure mechanisms are: moment equilibrium is not satisfied, failure due to shear, heave and Bishop. Due to the construction of the seawall, which is discussed in appendix G, the seawall is more or less fixed to the ground and constrained at the land side because of the large body of rock and concrete. Therefore it will be unlikely that any of these failure mechanisms will occur.

Too much groundwater flow

There are two main causes for too much ground water flow, these are: too much upward pressure and piping. The upward water pressure will never be a problem, as the only water present will be the sea water. Since the wall is constructed on limestone, which is not permeable, no groundwater will be present. For this reason and because the ground gradually increases behind the seawall, piping cannot occur.

Local failure of the structure

Another category of failure mechanisms for this kind of seawall is local failure of the structure. Again the moment equilibrium can be a failure mechanism and the failure due to shear also need to be considered. Because of the geometry of the Malecón seawall and the presence of the concrete body behind the wall, the moment equilibrium can be a problem for the top part of the wall. This kind of failure will be seen when the wall is analysed in more detail later in this paragraph. Failure due to shear will definitely be a problem. This failure mechanism had been observed quite a few times in the last 100 years. Especially the combinations of no moment equilibrium and too much shear stress will result into failure of the wall. From now on the failure mechanisms ’too much tensile stresses due to great moments', 'too much shear' and the combinations of these two will be taken into account.

Other causes

Other causes are also mentioned in the fault tree in figure J.1. For these other causes failure mechanism like scouring, collisions, etc. can be thought of. These failure mechanisms consist of accidental loads and actions and are therefore not taken into account in the fault tree.

Governing failure mechanisms of Malecón seawall

In order to determine the correct material properties of the Malecón seawall, the local failure mechanisms were investigated. It is known these failure mechanisms occurred in the past. Therefore it was chosen to model section 2 with the loads of hurricane Wilma as it is known that this section was destroyed. During this hurricane the top of the wall sliced off, as can be seen in figure 5.16. The pressure distribution has been composed as described in section 4.4. The dimensions of this section of the wall have been determined and can be found in table 5.1 in section 5.2.1.

Figure 5.16: Damage after hurricane Wilma

After applying the loads on the structure the stresses and displacements have been determined using the finite element calculations of ANSYS. Especially the stresses are of interest in order to determine if the structure cracks, therefore these will be discussed in detail. The displacements due to these loads can be found in figure I.1 in appendix I. The maximum displacements are very small, in the order of magnitude of less than 1 mm. The figure I.2 in appendix I shows the vector notation of the principal stresses, which are based on Mohr's circle, and clearly shows how the structure behaves. These figures help to understand how the structure reacts to the loads and therefore explain the distribution of the stresses, which will be discussed next.

Figure 5.17 shows the maximum principle stresses, bases on Mohr's circle, in the cross-section. These maximum stresses are of interest, because concrete is weak when a tensile stresses are concerned. The tensile stresses are referred to as the positive stresses and are shown as red stresses in the figure 5.17. In the front of the wall high tensile stresses occur at the height of the sidewalk with a maximum value of 2.2 MPa. The strength of lower classes of concrete is in the order of magnitude of 1 MPa. Also high tensile stresses can be found at the toe of the wall. These two points will be the critical points of the wall.

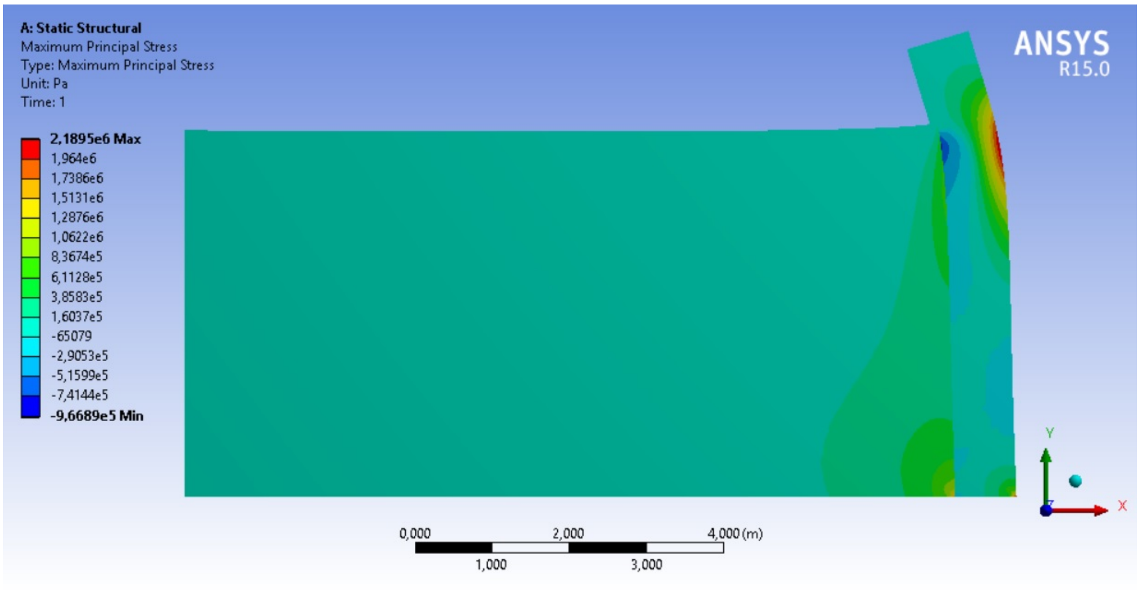

Figure 5.17: Maximum principal stresses due to Wilma load

The largest compressive stresses can be found in figure 5.18. This figure shows the minimum principle stresses and because the compressive stresses are given a negative value, these are the largest compressive stresses. The maximum compressive stress of 3.5 MPa is found at the top connection of the concrete foundation block and the wall. But since concrete can take high compressive stresses, the lowest concrete class has a strength of 8 MPa, these stresses will not be governing.

Figure 5.19 shows the shear stresses in the structure. It is marked that the highest stresses can be found around the transition between the supported part and the free end. The highest absolute value of 0.87 MPa is found at the part of the wall right above the sidewalk. Figure 5.17 showed that, at this location, a tensile stress occurred in the wall, so there will not be much friction at the location of the maximum shear stresses. Therefore the shear strength can not be increased and will be relatively low. This combination of

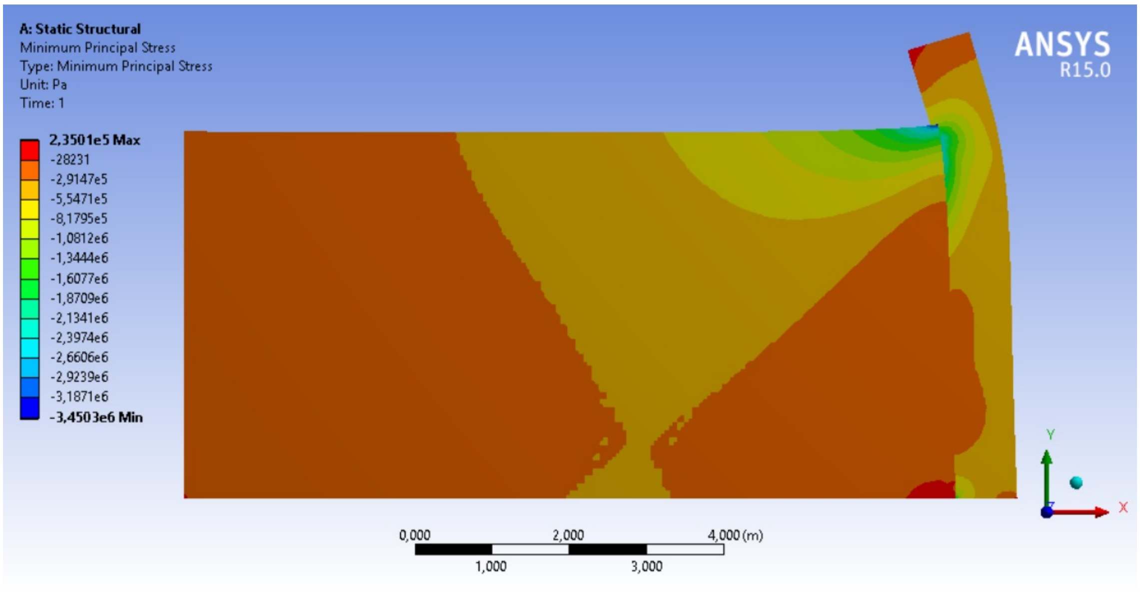

Figure 5.18: Minimum principal stresses due to Wilma load

stresses is probably the reason why this part sliced off during the Wilma hurricane.

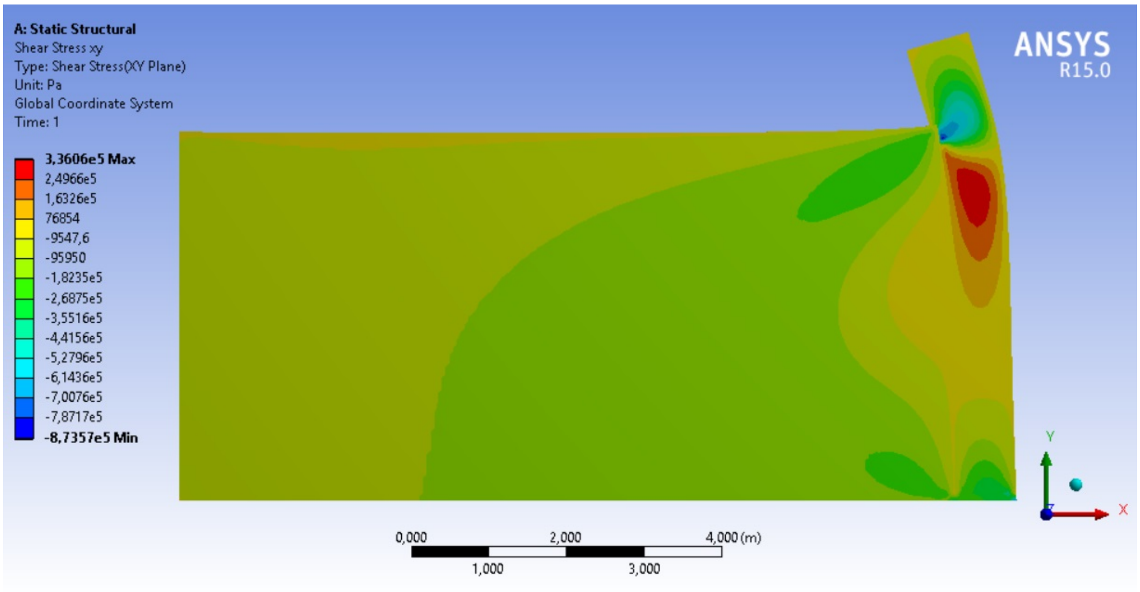

Figure 5.19: Shear stresses due to Wilma load

As it was known that the wall failed during the hurricane Wilma the concrete properties were iteratively changed until the maximum stresses exceeded the maximum concrete stresses. Since the characteristic properties of concrete mainly depend on the modulus of elasticity, this was changed until the tensile and shear stresses exceeded the strength properties. The maximum tensile stress is linked to the modulus of elasticity. The maximum shear stresses can be computed using formula 5.1.

$$v_{min} = 0.035 * k^{3/2} * f_{ck}^{1/2} \tag{5.1}$$

In which:

v_{min} = the maximum shear stress in N/mm^2
$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$
f_{ck} = the characteristic cylinder compressive strength
d = thickness of concrete part in shear in mm

It was found that for concrete class C12/15 both the tensile strength $f_{ctd} = 0.73$ MPa and the shear strength $v_{min} = 0.22$ MPa were exceeded. C12/15 is quite a low concrete class, but as the wall was constructed more than 100 years ago and quite a lot of cracks have formed since then, it is possible that this concrete class best describes the material of the Malecón seawall. This is a conservative approach but it is better to underestimate the material properties than to overestimate them.

The behaviour at the toe of the Maleón seawall can also be seen in reality. Because of the high tensile stresses cracks will be present in this part of the wall. As the concrete is already cracked it will be extra vulnerable to scour and the waves can take pieces of the concrete from the wall. The scour at the bottom of the wall is a known problem of the seawall and some sort of protection for scouring has already been placed.

5.2.3 Determining material properties

In the section 5.2.2 the current situation was analysed and the material properties were determined for the current seawall, which are presented in table 5.2.

	Current wall
Concrete strength class	C12/15
Modulus of elasticity E_{cm} (GPa)	27
Characteristic compressive strength f_{ck} (MPa)	12
Design compressive strength f_{cd} (MPa)	8
Characteristic tensile strength $f_{ctk,0.05}$ (MPa)	1.1
Design Tensile strength f_{ctd} (MPa)	0.73
Shear strength v_{min} (MPa)	0.22

Table 5.2: Material properties seawall [CEB, 2010]

As was described in section 5.2.1 a concrete body will be modelled behind the seawall. As it is not known how the area behind the Malecón was filled, it is assumed a mixture of rocks and concrete is used. Therefore the concrete body will be modelled as a very low strength concrete. It was decided to model this concrete body with a modulus of elasticity E_{cm} of 25 GPa. With this assumption some uncertainties arise whether the stiffness of this concrete body is assumed correctly. It is recommended to investigate the actual material properties of the fill behind the seawall.

5.2.4 Evaluation of the current wall

In order to evaluate the current condition of the wall, each section was analysed in ANSYS with a storm condition which has a return period of $\frac{1}{50}$ years. The storm conditions are described in section 3.5.2. The coupling from wave conditions to pressures is described in section 4.4. In this section the pressures on the seawall are determined. It was decided to evaluate each section separately. The geometry that is used for each section can be found in table 5.2 in section 5.2.1.

Due to the nature of the governing failure mechanisms both the shear and the principal stresses are of interest. For each section the general distribution of the stresses is the same, so only section 2 will be analysed in detail. Section 2 was selected, because this is the most unfavourable section. Naturally the exact values differ for each section so the extreme values of each section will be compared to the set material properties, which are discussed in section 5.2.3.

Maximum principal stresses

In figure 5.20 the maximum principal stresses for section 2 are displayed for a load of a $\frac{1}{50}$ year storm. As was discussed in section 5.2.2, two critical points exist for the maximum principal stresses. The highest tensile stresses occur at the toe of the wall. For this situation the tensile stresses do exceed the design tensile strength, discussed in 5.2.2. Therefore it can be concluded that something has to be done about the wall in order to resist a $\frac{1}{50}$ year storm. Moreover, the toe of the wall is another critical point, see figure 5.21. Some of these high stresses can be explained by the error the program makes due to the edge disturbance. Nevertheless some tensile stresses will exist at the toe of the wall and some will exceed the design tensile stress. Therefore some cracks will occur at the toe of the wall and the wall will experience scour, if this happens too often.

Another critical spot for the tensile stresses is in front of the wall at the top of the concrete body behind the wall. These tensile stresses are very interesting for the failure of the wall, because (as was said in section 5.2.2) these stresses are exceeded when the top part of the wall shears off. With the $\frac{1}{50}$ year storm the stresses are around 1.3 MPa. This is larger than the design tensile stress of 0.7 MPa.

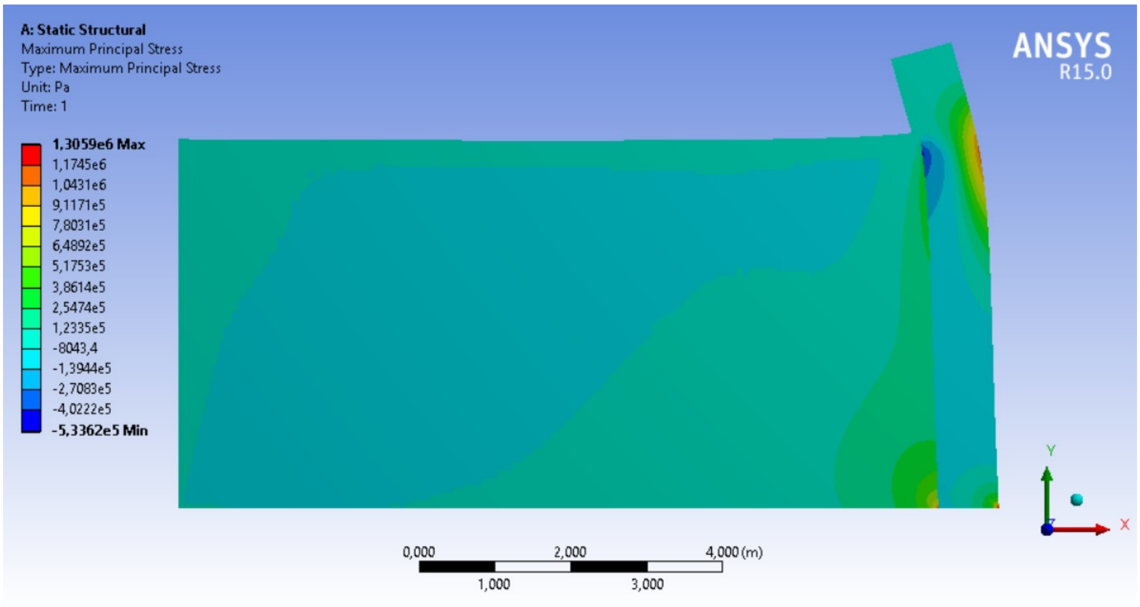

Figure 5.20: Maximum principal stresses in section 2 for a 1/50 year storm

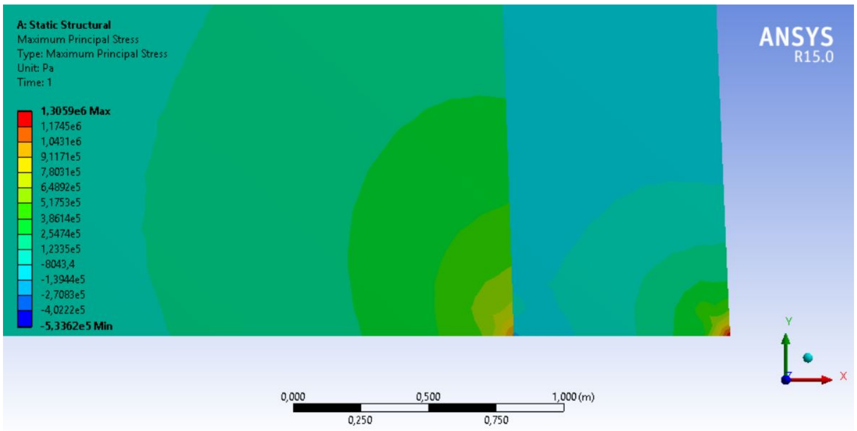

Figure 5.21: Zoom of the toe of the maximum principal stresses in section 2 for a $\frac{1}{50}$ year storm

Minimum principal stresses

Figure 5.22 shows the minimum principal stresses. The critical spot for the compressive stresses is at the right top of the concrete body behind the Malecón seawall. This corresponds to expectations, since the cantilever part of the wall rotates around this point. Even though this will be the critical spot the compressive stresses are not close to the maximum compressive stress, which is 8 MPa. It can be concluded that the compressive stresses will not be critical.

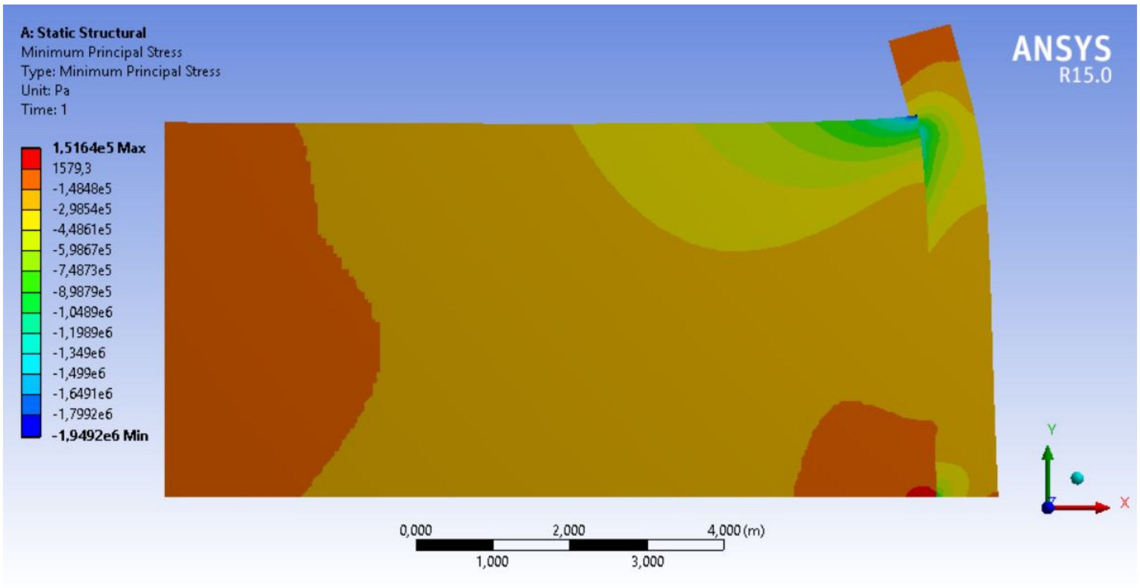

Figure 5.22: Minimum principal stresses in section 2 for a $\frac{1}{50}$ year storm

Shear stresses

Another failure mechanism for the wall is the failure due to too high shear stresses. The shear stresses can be seen in figure 5.23. It can be seen that there are some places where the shear stresses exceed the maximum allowable shear stress of 0.22 MPa. It has to be noted that for the shear stresses it does not matter whether the value is positive or negative, as it is the magnitude that is important. Therefore it can be observed from figure 5.23 that almost the total area, which is in line with the top of the concrete body, exceeds the maximum shear stress. Especially in combination with the observations on the maximum principal stresses, this can potentially be a huge problem. If the maximum principal stresses become too large, a tensile crack will form on the outside of the wall. Directly behind the area where this tensile crack can form, the shear stresses are too large so this crack will propagate backwards. Eventually the crack will reach the back of the wall and the top part will be sliced off.

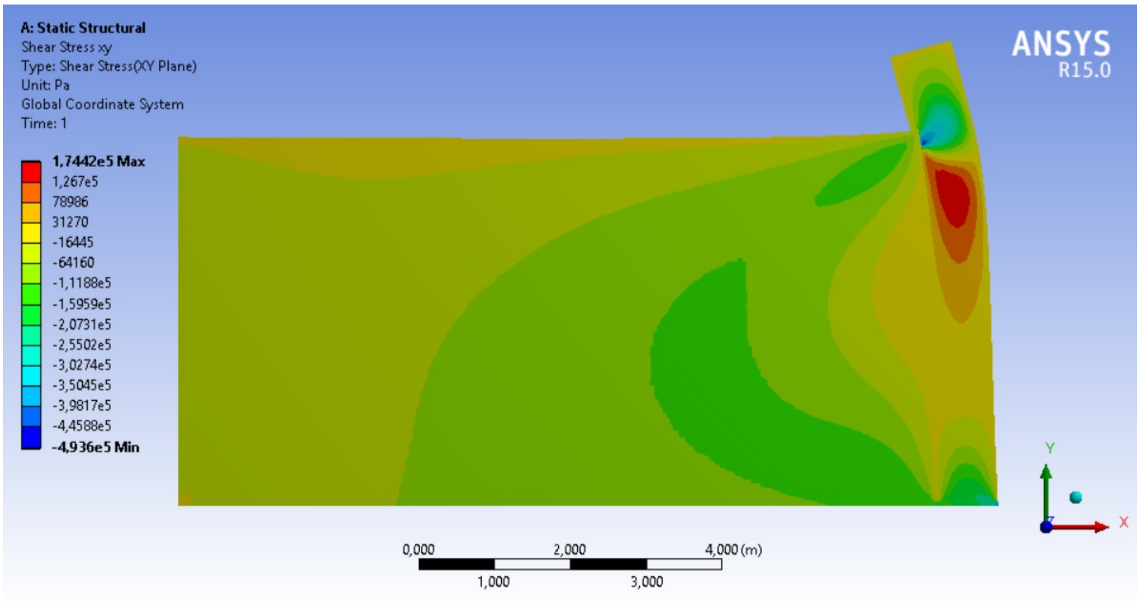

Figure 5.23: Shear stresses in section 2 for a $\frac{1}{50}$ year storm

Maximum values per section

Even though the general distribution of the stresses is the same for each section, the maximum and minimum principal stresses and the maximum shear stresses will differ for each section. The maximum and minimum principal stresses are presented in figure 5.24. In this figure also the ultimate tensile and compressive stresses are displayed. It can be observed that even though the tensile stresses do not reach the ultimate tensile stress in any section, there is not much room for errors. Especially the determination of the pressures on the wall is a source of insecurities. The compressive stresses are very unlikely to cause any problem because they are relatively low.

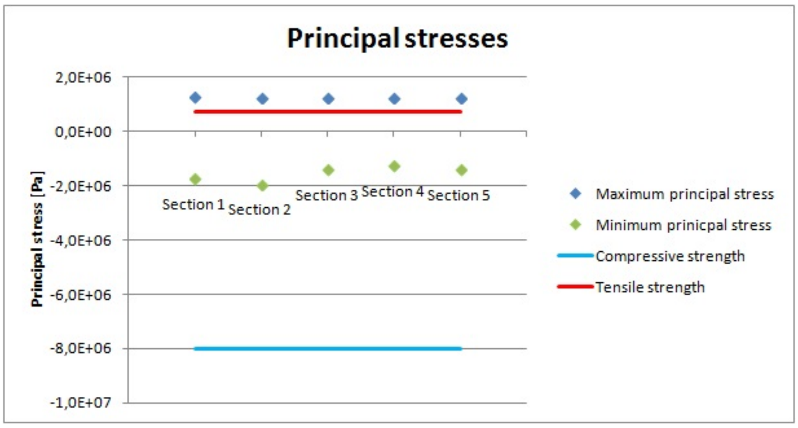

Figure 5.24: Maximum and minimum principal stresses of all sections

The minimum shear stresses in each section exceed the resistance shear stress of the concrete, which is a problem. If figure 5.23 is looked at closely, it can be seen that only a very small area of shear stresses exceeds the maximum shear stress of 0.22 MPa. But this is an indication mitigation measures should be taken to ensure the wall will withstand the load of a $\frac{1}{50}$ year storm.

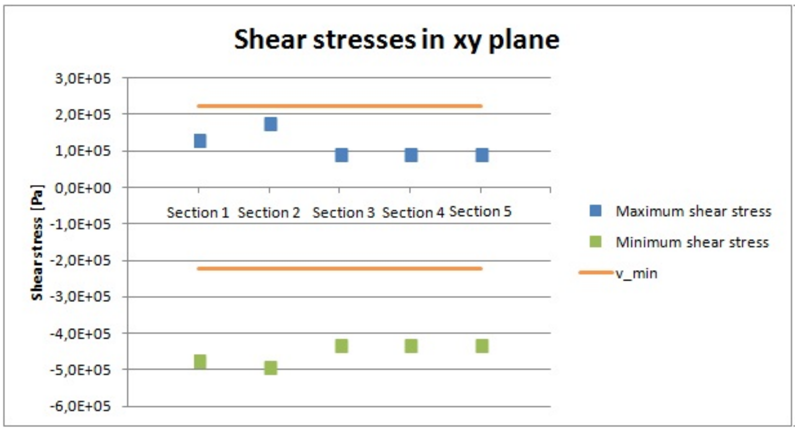

Figure 5.25: Maximum shear stresses of all sections

5.3 Conclusion

In conclusion, the Malecón seawall was divided into five different sections, so that a solution can be presented per section. The division of these sections was based on the state of the current wall. With the FE program ANSYS the current situation was modelled. In order to determine the most accurate material properties the wall was modelled with the loads that occurred during hurricane Wilma. This was done, because the wall is known to have failed during this particular hurricane. The material properties of the current wall have been set to C12/15. After determining all the material properties of the wall, the current wall was modelled with the loads given by a $\frac{1}{50}$ year storm. From this analysis it was concluded that the wall cannot withstand a $\frac{1}{50}$ year storm in its current condition. Both the tensile strength f_{ctd} of 0.73 MPa and the shear strength v_{min} of 0.22 MPa are exceeded during this storm condition. The wall fails locally due to these high tensile and shear stresses at sidewalk level. This may lead to the top of the wall breaking off.

Chapter 6

Alternative solutions

In the following section alternative solutions for reducing the floods of the study area are discussed. After an extensive brainstorm session several alternatives were thought of. The most feasible alternatives will be discussed and are categorized as onshore, nearshore and offshore. In 6.4 a small Multi Criteria Analysis is used in order to select the practical alternatives, these will be investigated further in chapter 7.

6.1 Onshore alternatives

6.1.1 Raising the Malecón seawall

For this alternative the Malecón seawall will be raised entirely. Naturally with a higher seawall the amount of water overtopping the wall will decrease.

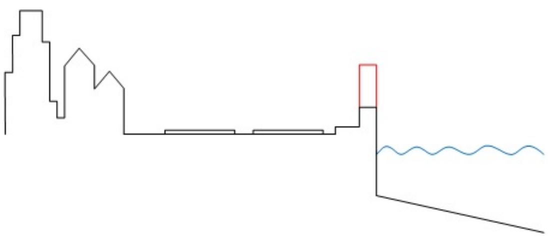

Figure 6.1: Raised wall alternative

6.1.2 Curved seawall

Another onshore alternative is altering the geometry of the seawall. Instead of keeping a straight wall, the wall will be curved. In doing so the waves will break on the wall, but the splashes will be directed back towards the sea instead of up into the air.

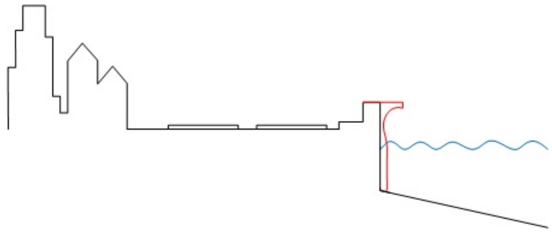

Figure 6.2: Curved wall alternative

6.1.3 Additional movable crown wall

The third onshore alternative involves a movable crown wall in addition to the existing wall. The crown wall will be some kind of storm surge barrier. In normal circumstances the appearance of the Malecón will not be changed. When a storm or hurricane approaches the crown wall will be raised and therefore it will increase the total height. Hence reducing the amount of overtopping.

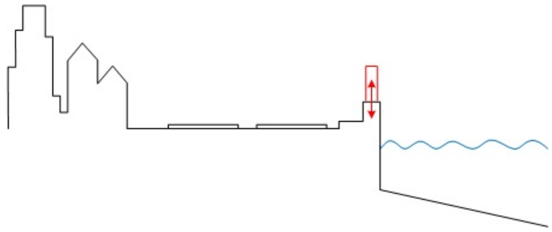

Figure 6.3: Additional movable crown wall alternative

6.1.4 Improve drainage system

In this alternative the lack of a good and efficient drainage system is considered. If the water in the area behind the Malecón can be drained as fast as it comes over the Malecón, the buildings on the other side will be spared. This drainage system needs to be placed close to the Malecón in order to be considered a good solution.

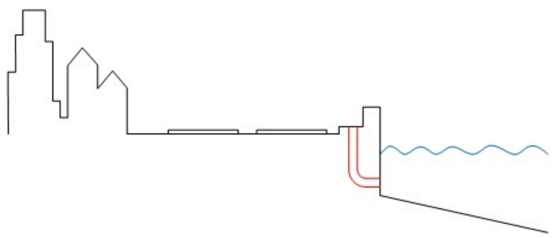

Figure 6.4: Improve drainage system alternative

6.1.5 Absorbing material

The last onshore alternative also reduces the large upward splashes of the waves. By placing an extra vertical layer of absorbing material on the wall the forces generated by the waves will be reduced. Therefore less water will be overtopping the wall.

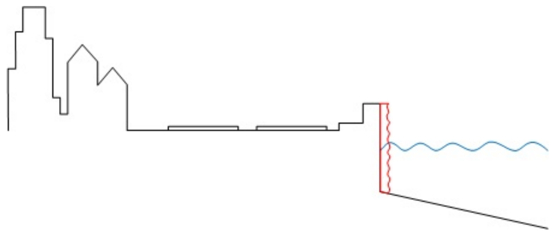

Figure 6.5: Absorbing material alternative

6.2 Nearshore alternatives

6.2.1 Slight slope of seabed

One way to create a dissipating boundary for the waves is slightly tilting the seabed. This will be the first nearshore alternative. The waves will break slightly further offshore due to the slight slope of the seabed. Therefore the waves will lose more energy before the Malecón.

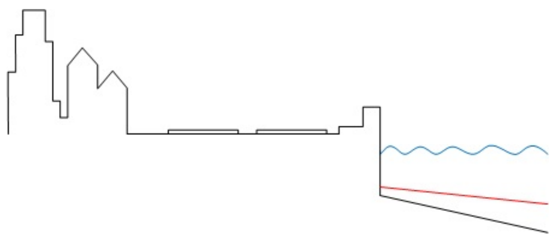

Figure 6.6: Slight slope of seabed alternative

6.2.2 Revetment

The second near shore alternative will be another dissipating boundary in the form of a revetment. This will also cause the waves to break before hitting the seawall.

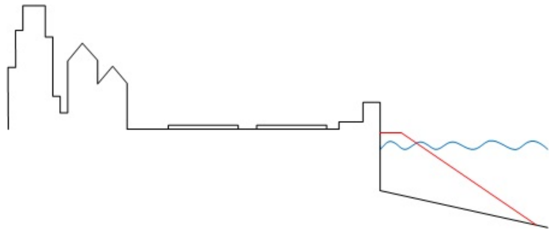

Figure 6.7: Revetment alternative

6.3 Offshore alternatives

6.3.1 Submerged breakwater

This alternative is the offshore breakwater. In order to make an offshore breakwater three choices have to be made. The first choice is whether the breakwater will be submerged or emerged. For this alternative the submerged breakwater will be taken, in section 6.3.2 the emerged alternative will be discussed.

The second choice to be made is about the material of the breakwater. Three main categories can be recognized in material of the breakwater. The first is rubble mound, the second concrete elements and the last is caissons.

The last thing to consider in designing an offshore breakwater is whether to make the breakwater attached or detached.

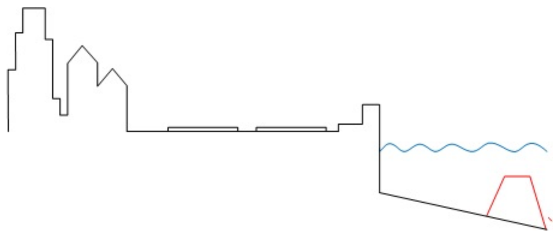

Figure 6.8: Offshore submerged breakwater alternative

6.3.2 Emerged breakwater

Here the emerged breakwater will be discussed. In detailing this alternative again two decisions have to be made, about the material and whether or not the breakwater will be attached or not.

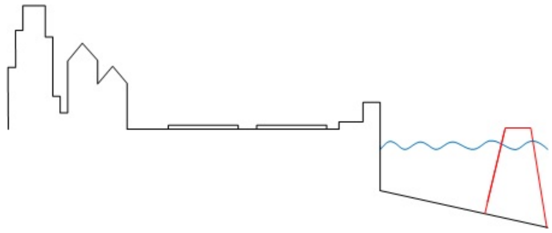

Figure 6.9: Offshore emerged breakwater alternative

6.4 Selection feasible alternatives

The most feasible ones from previous described alternatives will be designed in more detail. To determine these alternatives a qualitative evaluation will be executed. To make this evaluation tangible a Multi Criteria Analysis (MCA) is used. The most important design criteria will be used in basic form to determine the value of the alternatives. These criteria will be weighed in order to make a well-founded decision.

6.4.1 Criteria and weight factors

The criteria used for the first MCA are divided in three types: social criteria, technical criteria and costs. These three types are compared with each other to determine the weight factor as can be seen in table 6.1. When the criteria in the row is more important it is awarded a one and when the criteria in the column is more important it is rewarded a zero. The technical criteria have been given a weight factor two and the social criteria and costs both a factor one.

The criteria used are:

	Technical criteria	**Social criteria**	**Costs**	**Total**
Technical criteria	x	1	1	2
Social criteria	0	x	1	1
Costs	0	1	x	1

Table 6.1: Determining weight factors criteria

Social criteria

- *View of the city of Havana is preserved as much as possible*
  One of the design criteria is to preserve the view of Havana as the Malecón is an icon for the city. The more the current view is preserved the higher the score of the alternative.

- *The social function of the Malecón for the residents and tourists in Havana is respected*

The Malecón has several social functions in its current form. The more these activities are respected in the new design, the higher the score.

Technical criteria

- *Amount of flood risk reduction*
  The more efficient an alternative is in the realizing the set amount of flood risk reduction the higher the score.

- *Increasing the resistance of the Malecón sea wall or decreasing the loads on the wall*
  The current wall fails during design conditions so the design should be effective in keeping the wall from failure.

Costs

- *Minimizing the costs*
  The lower the costs for the alternative the higher the score, because low costs are favorable.

Environmental criteria are of importance as well, but as the client did not specify this as a design criterion only the heritage and social function of the solution are taken into account. It is recommended that the environmental impact of the solution will be evaluated before construction.

6.4.2 Multi Criteria Analysis

The results of the MCA are presented in figure 6.10. The yellow columns are the social criteria, the green columns are the technical criteria and the blue column is the costs. The criteria are awarded with a mark one to three, one being least good and three very good. As can be seen the revetment, sub- and emerged breakwater and the curved wall are the four alternatives that score best.

		Effect on the image of the city of Havana	Effect on the social function of the Malecón	Effect on the flood risk reduction	Effect on the resistance of or the loads on the Malecón sea wall	Costs	**Total**	**Weighed Total**
Onshore	Raising the Malecón sea wall	1	1	3	1	3	9	8
	Curved wall	3	3	3	1	2	12	**9**
	Additional movable crownwall	2	2	3	1	1	9	7
	Improving drainage system	3	3	1	1	3	11	8
	Absorbing material	3	3	1	2	2	11	8
Nearshore	Slight slope of sea bed	3	3	2	1	1	10	7
	Revetment	2	2	3	3	2	12	**10**
Offshore	Submerged breakwater	3	3	2	2	2	12	**9**
	Emerged breakwater	1	3	3	3	1	11	**9**

Figure 6.10: Results first Multi Criteria Analysis

6.4.3 Conclusion

It has been decided to continue the design with the revetment, sub- and emerged breakwater and the curved wall, as well as the raised wall. The raised wall did not score best in the MCA, but it can easily be combined with another alternative to increase the effectiveness. Therefore during further design, it will still be considered as partial solution that can be applied in combination with any of the four selected alternatives.

Chapter 7

Initial Design

To be able to choose the solutions that will be designed in detail, first an initial design is carried out, so the solution alternatives can be compared. Section 7.1 elaborates on the methodology that is used to find the best solution per section. Section 7.2 presents the different structural design alternatives for the Malecón seawall. In section 7.3 the curved wall solution is investigated. Initial designs for the revetment and the breakwater are made in sections 7.4 and 7.5 respectively. In section 7.7 the best solution is selected per section, based on the costs and benefits of the alternative solutions.

7.1 Methodology

As said, the initial design has the purpose of being able to determine the alternative solutions for the detailed design per section of the Malecón seawall. The alternative solutions that are discussed in the initial design are the curved wall, the revetment, the breakwater and feasible combinations of these three. To be able to see which combinations of alternatives will meet the design criteria, some goals are set per alternative. The design of the curved wall aims to find out to what amount the wave overtopping can be reduced and whether or not this exceeds the limit of 50 L/s/m. The revetment aims to do the same, but here the corresponding amount of material needed to establish this amount of overtopping is determined. The initial design of the breakwater is based on the maximal transmitted wave through the breakwater, which leads to an overtopping of 50 L/s/m. Also for the breakwater the necessary material is calculated. Based on these findings, the alternatives that meet the design criteria can be established per section, including the corresponding amount of used material.

In this chapter, sections 3 and 4* are combined, because the hydraulic conditions are similar at these sections. Therefore the same solution will be developed for section 3 and 4*. The amount of material is interpreted as an indication of the costs, which can then be used to compare the alternatives that meet the design criteria. In order to make the initial design, some simplifications and schematisations have to be made in the designing process, which are described in the following sections. More detailed designs are described in chapter 8.

7.2 Structural design alternatives Malecón

There are several alternatives for the structural redesign of the Malecón seawall. First, the wall could be raised or not in order to suffice the overtopping condition. Second, the wall can be curved or not. Last, repairing of the current bottom part of the wall and reconstructing the top part is sufficient for the wall to resist the imposed loads (repairing alternative) or the wall is in such a poor shape that repairing activities will not be enough and a new wall has to be built also for the bottom part. Either a new wall in front of the old one is sufficient (addition alternative) or an entire reconstruction of the top part is necessary as well (partial replacement alternative). All last three named alternatives are possible considering a curved wall and an elevation of the wall. A total replacement has been considered as well, but the construction method is very costly and the type of support at the land side of the seawall is based on a lot of insecurities. Therefore this alternative will not be considered.

7.2.1 General design choices

Material properties

The concrete used should be wear resistant because of the frequent impact wave loads and should be resistant against chloride penetration. Chloride penetration requires great attention, especially in tropical waters. As the new concrete elements and structures will be reinforced, chloride penetration could cause a corrosion problem of the reinforcement steel. According to Quay Walls, second edition [SBRCURnet, 2014], concrete classes with a minimum cubic compressive strength of 35 MPa can be assumed to be wear resistant. It has been determined that the Malecón has an environmental class XS3 as this is a class fit for tidal, splash and spray zones near sea water, given in EN-1992-1-1 table 4.1 [CEN, 2004]. For this environmental class a concrete class of C35/45 is recommended. The strength properties of this concrete class are as given in table K.1 in appendix K.

To resist the chloride penetration concrete with a closed pore structure is desired. Portland cement combined with very fine fillers can be used to obtain this closed pore structure [SBRCURnet, 2014].

Reinforcement and cover

To prevent corrosion a thicker cover should be used in tropical waters than for usual conditions. The usual cover thickness is around 50 mm, but in tropical waters a concrete cover of 100 mm is advised.

The usual strength class of steel used is B500. This reinforcement steel type has a characteristic yield strength of 500 MPa.

7.2.2 Repairing alternative

The repairing alternative entails restoring of cracks in the concrete, restoring surface damage, improving of the scour protection and reconstructing the top part of the wall. Figure 7.1 shows the design in more detail.

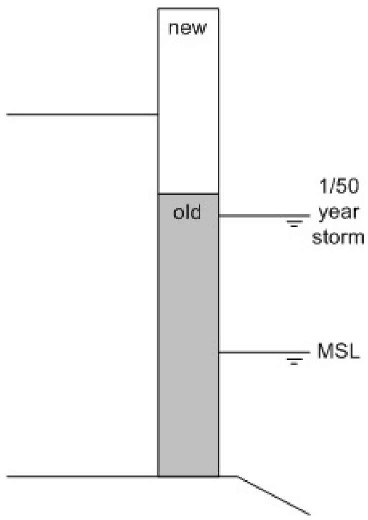

Figure 7.1: Repairing alternative

The connection of the new, and possibly elevated part, and the current seawall is of great interest. First of all the connection should be bonded. This means that it should be a rigid type of connection. This can be achieved with the use of dowels. Secondly, to get a true bonded connection there should be enough friction between the two layers. This can be achieved by a treatment like sand blasting or water blasting of the interface surface. At the moment the most critical point of the Malecón is at the sidewalk level of the wall. This part of the wall is not strong enough to withstand the ULS conditions as was showed in chapter 5, so it has to be redesigned including longitudinal and shear reinforcement. This can only be done by demolishing the top part of the current wall from just below the critical zone onward. The connection of the current seawall to the new elevated part will therefore not be at the current crest height level, but just below sidewalk level. The exact location of the interface is evaluated in section K.2.5 in appendix K. This location has been determined such that no high tensile and shear stresses occur in the current wall. The optimal location is around 0.7 meters above MSL.

The thickness of the new top part of the wall will be the same as that of the current Malecón seawall, as it has to be connected directly on top of the old wall. The thickness is therefore 800 mm. The detailed reinforcement design can be found in section K.2.

Evaluation of the design with ANSYS

The detailed evaluation of the design can be found in section K.2.6 in appendix K. It was concluded that concrete class C35/45 does not have an adequate tensile strength to resist the loads of a $\frac{1}{50}$ year storm, so concrete class C40/50 is advised. To meet the maximum crack width criterion a large amount of longitudinal reinforcement is necessary. The design criterion given in the EN-1992-1-1 [CEN, 2004] is used, but it is recommended to investigate the effect of tropical waters on the corrosion of the reinforcement steel and the maximum allowable crack width. When the height of the Malecón seawall is not raised the structure will not be cracked and the design is already considered to be safe.

7.2.3 Addition alternative

The addition alternative entails constructing a new wall on the seaside of the current Malecón seawall. Figure 7.2 shows the design in more detail.

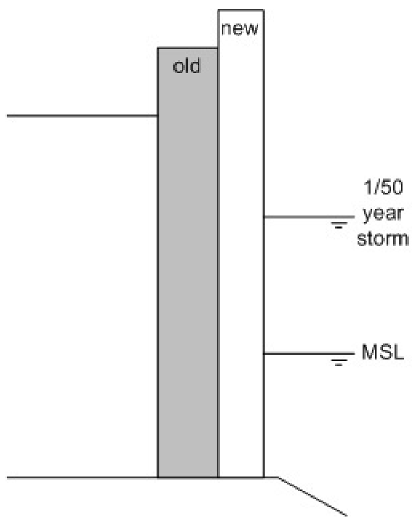

Figure 7.2: Addition alternative

This wall has to be designed according to the present design codes including longitudinal and shear reinforcement. The new wall has to be founded correctly on the limestone foundation and should be attached to the old wall to prevent gaps. This can be achieved with the use of some dowels and a treatment like sand blasting or water blasting to get a frictional bond between the two walls. At the connection penetration of the water should be avoided, so the interface should be properly sealed.

According to formula K.1 in appendix K, the total thickness will be 700 mm as determined in section K.3. A detailed reinforcement design can also be found in this section.

Evaluation of the design with ANSYS

The detailed evaluation of the design can be found in section K.3.5 in appendix K. It was concluded that for a $\frac{1}{50}$ year storm the design is safe and the total structure can resist the imposed loads. For the ULS conditions the stresses in the old wall exceed the strength of this part of the wall. For this design the rule of thumb for the thickness of the wall does not suffice. Therefore a thicker added seaside wall with a thickness of 800 mm is advised. For the toe of the wall it is recommended to investigate the phenomenon of scour.

7.2.4 Partial replacement alternative

As the partial replacement alternative is a combination of the repairing and addition alternative, the design procedure will be a combination as well. The top part of the old wall will be replaced and an additional wall will be placed in front of the seaside face of the current seawall. Figure 7.3 shows the design in more detail.

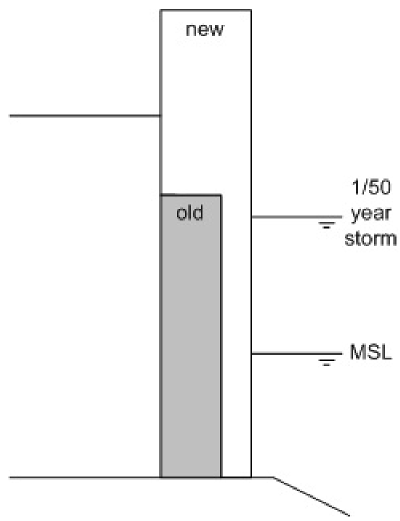

Figure 7.3: Partial replacement alternative

Again the connection between the current wall and the elevated part is of great interest and needs the same treatment as described in section 7.2.2. For the the seaward part of the new wall, the design procedure as described in section 7.2.3 will be used.

For this design as well, the location of the interface between the current wall and the new top part of the wall has been optimized using ANSYS as described in section K.4.5 in appendix K. The optimal location of the intersection is around 2.3 meters above MSL.

The thickness of the bottom part of the new concrete wall is determined to be 400 mm as explained in section K.4 in appendix K. This leads to a thickness of the top part of the new wall of 1200 mm. A detailed reinforcement design can also be found in last-named appendix.

Evaluation of the design with ANSYS

The detailed evaluation of the design can be found in section K.4.6 in appendix K. For a $\frac{1}{50}$ year storm the structures strength is sufficient to withstand the imposed loads. For the ULS condition the tensile stresses slightly exceed the strength, but from the crack width calculation in K.4.4 the crack width is only small. It is advised to check the effect of tropical waters on the maximum allowable crack width. For the toe of the wall it is recommended to investigate the phenomenon of scour.

7.2.5 Comparison of the alternatives

The three structural solution alternatives are compared to make a well-founded choice. The cross section for an elevated wall for section 2 is used to compare the costs of the different alternative. The results are presented in table 7.1.

Alternative	Repairing	Addition	Partial replacement
Thickness of the structure [m]	0.8	1.6	1.2
Cross section [m^2]	3	4.9	4.2
Concrete class	C40/50	C35/45	C35/45
Longitudinal reinforcement	$\oslash$20 - 55 mm	$\oslash$20 - 150 mm	$\oslash$20 - 90 mm
Crack width [mm]	0.27	-	0.19

Table 7.1: Initial design results of the three structural alternatives

The repairing alternative has the smallest cross section which leads to lower costs, but the alternative is not recommended for a raised wall as cracking is a concern. Besides a large amount of longitudinal reinforcement is needed due to the high tensile stresses in the concrete wall during the ULS condition. When the wall is not raised, this alternative is sufficient. The addition alternative is easy to built as the old wall can stay entirely. This also reduces the risk of flooding during the construction period as the current flood protection is maintained during this period. However the cross section needed for a safe design is quite high. The partial replacement principal has a new concrete cross section of 4.2 m^2 and the tensile stresses during the ULS condition are low enough to prevent severe cracking, also when a raised wall is considered. After cracking only a small crack width will be present which is preferable to prevent corrosion of the reinforcement steel.

7.3 Curved and Raised Wall

One of the solution alternatives is constructing a curved wall. Another solution alternative is raising the wall, which is restricted in the boundary conditions to a raise of maximal 0.5 m. As these alternatives work in a similar manner to reduce overtopping, it makes sense to address the curved wall and the raised wall together. They both do not change the wave field in front of the wall, but they (partially) prevent the water that splashes up against the wall to overtop over the crest of the wall. Both of the options and the combination of these two will be discussed in this section.

Adding curvature to the wall will reduce the overtopping because some of the water will be reflected backwards to the sea instead of going upwards. In the Eurotop Overtopping manual [EurOtop Team, 2007] the effectiveness of the recurved wall is quantified by a reduction factor k. This reduction factor is defined by equation 7.1.

$$k = \frac{q_{with\ recure}}{q_{without\ recurve}} \tag{7.1}$$

In section 7.3.5 'Effect of bullnose and recurve walls' of the Eurotop Overtopping manual [EurOtop Team, 2007] it is discussed how to determine the value of k.

The results of this method were compared to the results of the physical model test done in Italy [Buccino et al., 2014]. For this physical model test a curved wall was built according to a study done by prof. L. Córdova, who provided the dimensions of the curve, see figure 7.4. The Eurotop Overtopping manual [EurOtop Team, 2007] does not make an explicit difference between a curved wall and a parapet. It states however that the effectiveness factor k is the same for a curved wall and a parapet. The reduction factor k was determined using the parameters of the curved wall used in the physical model test. However the results of the physical model test [Buccino et al., 2014] and the reduction factor k given by the Eurotop Overtopping Manual [EurOtop Team, 2007] are not comparable with each other. After discussing this with prof. L. Córdova it was decided to continue with the results of the physical model test.

Due to the fact that only a limited number of wave conditions were tested in the physical model test, an estimation was made for the reduction of the overtopping. The wave conditions of each section were determined in section 4.3. In order to make the best possible estimation, these wave conditions are compared to the wave conditions of the different tests of the physical model. The most similar physical model test was determined for each section, it turns out all sections can be described best with two different physical model tests. The wave conditions for these two tests are shown in table 7.2.

The results of the physical model test give an overtopping for six cases: the current wall, raising the current wall with 0.5 m, raising the wall with 1 m, curving the current wall, curving the wall and raising it with 0.5m and curving the wall and raising it with 1 m. For this study it was chosen to compare four cases since raising the wall with 1 m is not

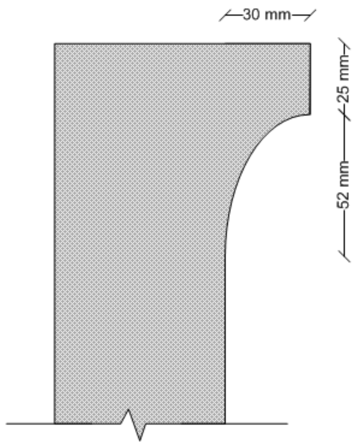

Figure 7.4: Dimensions curved wall

	H_s offshore [m]	H_s at toe [m]	T_p [s]	Set-up [m]
Test 3	8	5.4	12	2.28
Test 11	8	5.4	12	1.73

Table 7.2: Wave conditions comparable physical model tests [Buccino et al., 2014]

realistic given the design boundary conditions. With the results of the physical model test the reduction of overtopping was determined, in comparison to the current situation. The reduction of the physical model test is given in 7.3.

Comparable test	Percentage of reduction		
	Only curved	Only raised	Curved and Raised
Test 3	6	21	34
Test 11	8	24	39

Table 7.3: Reduction of curving and/or raising wall

Given the amount of overtopping of 600-1100 L/s/m [Buccino et al., 2014] for a $\frac{1}{50}$ year storm, it is obvious that raising and curving the wall will not be enough to satisfy the design criterion of 50 L/s/m. Therefore the alternative solution of only creating a raised curved wall is not an option. However, it may be combined with a revetment and/or breakwater. The best combination of alternative solutions will be discussed in 7.7. It should be noted that this conclusion is only based on the hydraulic conditions. For the majority of the study area, the wall needs to be repaired because of the poor condition of the wall.

7.4 Revetment

7.4.1 Design choices

To reduce overtopping one solution could be to a place a revetment, a rock mound, directly in front of the wall. The initial design of the revetment is based on the overtopping and the stability of the armourlayer.

By not allowing the revetment to be higher then the wall, a composite vertical wall is created, see also figure 7.5. To determine the overtopping over such structures, there

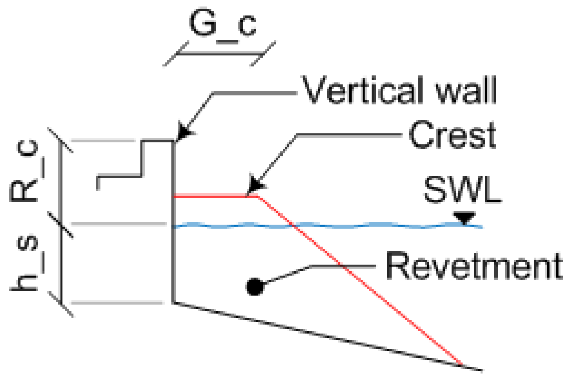

Figure 7.5: Sketch of revetment

are three types of mounds according to the Eurotop manual [EurOtop Team, 2007]: (1) a small toe mound, (2) a moderate submerged mound and (3) an emergent mound. The only type that reduces the overtopping over the Malecón seawall significantly is the emergent mound. This states that the crest of the structure protrudes above still water level during design conditions. Calculations in section 7.4.2 show that a wide crest is needed to reduce overtopping to the desired value of 50 L/s/m. The necessary width of the crest is calculated according to [EurOtop Team, 2007]. The selected armour layer of this mound is concrete cubes, as rubble is not available, the construction is the easiest and no expensive patents have to be used. A double layer is preferred over a single layer for a better overtopping reduction. As the slope is always relatively steep for an economic concrete armour units design, the slope is not of any influence for the overtopping, the steepest stable slope of 1:1.5 is used to minimize amount of material.

7.4.2 Overtopping

The overtopping over a combination of a mound and a vertical wall can be determined according to the Eurotop manual [EurOtop Team, 2007]. This approach describes three types of mounds, depending on the height of the mound:

- Small toe mounds which have no significant effect on the waves. Here the calculation is the same as for simple vertical walls and the toe may be ignored, see the section 4.2.

- Moderate submerged mounds which do affect wave breaking. Here a modified approach of the simple vertical wall is needed. For the Maleón seawall this leads to equation 7.2 (formula 7.14 of the Eurotop manual), valid for $0.05 < d_* \frac{R_c}{H_{m0}} < 1.0$ and $h_* < 0.3$. Because of the relatively low freeboard this equation not valid for the Maleón.

$$\frac{q}{h_*^2 \sqrt{g h_s^3}} = 7.8 * 10^{-4} * \left(d_* \frac{R_c}{H_{m0}} \right)^{-2.6} \tag{7.2}$$

- Emergent mounds in which the crest of the armour is located above still water level. Here the prediction may be adapted from rubble mound structures with a crown wall on it. For the Malecón seawall this leads to equation 7.3 (formula 6.5 of the Eurotop manual). This formula is valid for $\xi < 5$, which is true since it is a steep structure. Because of the steep nature of economic rubble mound structures, non-breaking waves determine the overtopping, and no form factor is included in the equation. Therefore, the slope of the structure does not directly influence the overtopping. The roughness reduction factor γ_f is 0.47 for double-layer cubes and 0.50 for single-layer cube. To achieve the highest reduction of overtopping, a double-layer has been selected.

$$\frac{q}{h_*^2 \sqrt{g H_{m0}^3}} = 0.2 * \exp \left(-2.3 * \frac{R_c}{H_{m0} * \gamma_f * \gamma_\beta} \right) \tag{7.3}$$

Section	1	2	2*	3&4*	4&5
small toe mound	345	427	384	754	384
emerged mound excluding wider crest	135	76	219	459	185
emerged mound including wider crest	50	50	50	50	50

Table 7.4: Overtopping rates of revetments [L/s/m]. No raising of the wall nor curved walls are included in these computations.

Crest width

The simplest version of a rubble mound structure is one with an armoured crest width of only $3 * k_t * D_n$. In which k_t is the layer coefficient of 1.1 for cubes and D_n the nominal stone diameter. This minimum width which is necessary for stability can be determined using the required stone size, calculated in section 7.4.3. A reduction of the overtopping is possible if the crest is widened, as more wave energy can be dissipated on a wider crest.It turns out that reducing the overtopping to 50 L/m/s needs a wider crest on the whole Malecón seawall, except for section 2. According to the Eurotop manual, the reduction factor on the overtopping discharge is given in formula 7.4.

$$C_r = 3.06 \exp\left(-1.5 \frac{G_c}{H_{m0}}\right) \tag{7.4}$$

Using this, the necessary crest width can be determined. Including a curved wall in this determination gives a smaller width, as a curved wall also reduces overtopping. The precise reduction is difficult to determine, especially in combination with a revetment. In this calculations the curved wall causes a reduction of overtopping of 27%, so the revetment has to reduce the overtopping to 63 L/s/m. This is based on the average reduction of the curved wall in the physical model test [Buccino et al., 2014]. Also the height of the wall induces differences in crest width as a higher wall will reduce overtopping.

The results of the calculations of the crest width are presented in table 7.5.

Section	1	2	2*	3	4&5	4*
Minimum crest width needed for stability	4.27	4.23	5.36	5.85	4.56	5.85
Including curved wall and raised wall	*2.60*	*2.21*	5.56	6.78	*2.98*	7.26
Including no curved wall and raised wall	*3.26*	*2.86*	6.37	7.65	*3.68*	6.47
Including curved wall and no raised wall	*3.77*	*2.53*	5.89	8.44	4.71	8.20
Including no curved wall and no raised wall	4.43	*3.19*	6.70	9.32	5.41	9.06

Table 7.5: Necessary crest width of the revetment [m]; for stability in row 1; for overtopping rates of 50L/s/m in row 2-5. Values in *italic* are not feasible because stability determines a wider crest.

7.4.3 Stability

The stability of two-layer cubes can be determined with formula 5.152 from the Rock Manual [CIRIA, CUR, CETMEF, 2007] derived by Van der Meer, see equation 7.5. On economic reasons, a steep slope is preferable. The steepest stable slope for concrete cubes is 1:1.5. As the overtopping of the revetment is not influenced by the slope, stated in section 7.4.2, a slope of 1:1.5 has been selected for the revetment.

$$\frac{H_s}{\delta * D_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} \tag{7.5}$$

In which N_{od} is the damage number, N is the number of waves and s_{om} is the fictitious wave steepness. Here a value of 0.2 is chosen for N_{od} which represents the start of damage of the cubes and is conservatively chosen from table 5.33 from the Rock Manual.
The results for the sizes of the cubes per section can be found in table 7.6.

	Section 1	Section 2	Section 2*	Section 3&4*	Section 4&5
D_n [m]	1.30	1.28	1.62	1.77	1.38
Mass [t]	5.1	4.9	10.1	13.1	6.2

Table 7.6: Necessary size of two-layer cubes armour layer of revetment

7.5 Breakwater

Two types of breakwater are designed, an emerged and a submerged breakwater. For the emerged breakwater a distinction can be made between the emerged breakwater and the low-crested emerged breakwater. The difference between the two is that the emerged breakwater has very limited overtopping over the breakwater itself, while the low-crested emerged breakwater is subject to wave overtopping over its crest.

7.5.1 General design choices

The design of the breakwater is based on the maximal transmitted wave through the breakwater, which leads to a wave overtopping of 50 L/s/m on the seawall. The initial design is based on stability of the armour layer and transmission through the breakwater. The stability of the armour layer is dependent on which type of armour layer is used. To decide on this, multiple types of armour layer are compared. The option rubble mound is not available in the (large) size needed for this project. Therefore concrete armour units are a better option. In this category a lot of options are possible, out of which the patented armour units are not suitable for this project due to high costs. The best option is cubes, because of simplicity in producing the units and relatively easy placement. Because of limited design experience with single layer cubes, double layer cubes are used. The slope is 1:1.5, since the stability then accomplished is good for cubes and the least material is used. The location of the breakwater is selected according to the length that is needed for the waves to break. According to section 5.1 of the Rock Manual [CIRIA, CUR, CETMEF, 2007], waves travel up to 80 per cent of the local wave length (L) before they complete the breaking process. The wave length is between 79 meters in section 1 up to 100 meters in section 3, which means that the waves break in a distance of approximately 63 meters in section 1 and 80 meters in section 3. Therefore when the breakwater is constructed between 60 and 120 meters offshore the waves have enough distance behind the breakwater to fully break.

7.5.2 Emerged breakwater

For the emerged breakwater the size of the armour layer is determined with the stability and the cross-sectional area is found using the transmission.

Stability

The stability of two-layer cubes can be determined with formula 5.152 for the Rock Manual [CIRIA, CUR, CETMEF, 2007], see equation 7.2. The number of waves is calculated with a storm duration of 20 hours, which results in 6600 waves for a mean wave period of 10.9 seconds. The storm duration of 20 hours is quite conservative. The density of the concrete is assumed at 2400 kg/m^3 and the density of seawater is 1025 kg/m^3.

In table 7.7 the results are presented.

Section	1	2	2*	3&4*	4	5
H_s 100 m offshore [m]	5.60	5.45	6.23	6.09	5.44	5.18
D_n [m]	2.35	2.28	2.64	2.58	2.28	2.16
Mass [t]	31.1	28.5	44.3	41.1	28.3	24.1

Table 7.7: Significant wave heights and necessary nominal diameter of two-layer cubes of breakwater

Transmission

The transmission of the emerged breakwater is determined with formula 5.59 from the Rock Manual [CIRIA, CUR, CETMEF, 2007], see equation 7.6. For breakwaters with a lower $\frac{R_c}{H_s}$ ratio the transmission coefficient C_t is 0.80 and for a higher ratio it is 0.10. It is however chosen to have the ratio in the range of equation 7.6, because this is the most realistic.

$$C_t = 0.46 + 0.3 * \frac{R_c}{H_s} \quad for - 1.13 < \frac{R_c}{h_s} < 1.2 \tag{7.6}$$

Since H_s and the needed C_t are known, crest height R_c can be calculated. In the inital design phase, it is not checked whether these crest height imply an emergent structure in terms of overtoppping. From the armour size found in table 7.7, the crest width can be deduced. According to the Rock Manual [CIRIA, CUR, CETMEF, 2007] this needs to be a minimum of three rows or armour units. The results are found in table 7.8.

Section	1	2	2*	3&4*	4	5
H_o [m]	1.73	1.78	1.78	1.86	1.84	1.84
C_t [-]	0.31	0.33	0.29	0.31	0.34	0.36
R_c [m]	0.94	0.54	1.74	1.26	0.33	-0.07
B [m]	7.1	6.8	7.9	7.7	6.8	6.5
Area in section [m^2/m]	381	248	402	467	527	338

Table 7.8: Results of transmission calculations emerged breakwater

7.5.3 Low-crested emerged breakwater

The crest height of the low-crested breakwater is zero, which means that the crest is at storm surge level.

Stability

Since no specific formula is known for the stability of the double cubes armour layer of a low-crested emerged breakwater, the same formula is used as for the emerged breakwater. Therefore the same results as found in table 7.7 apply for this type. The results for the sizes of the cubes per section can be found in table 7.7.

Transmission

The crest height R_c and width B of the breakwater are determined with the necessary transmission for the incoming and outgoing wave height. The incoming wave is known and the outgoing wave is based on the design criterion of 50 L/s/m wave overtopping at the wall. Formula 5.66 for the Rock Manual is used [CIRIA, CUR, CETMEF, 2007], since only narrow structures are considered, see equation 7.7. It is valid for low crested structures. The results of the calculation are displayed in table 7.9.

$$C_t = -0.4\frac{R_c}{H_s} + 0.64\left(\frac{B}{H_s}\right)^{-0.31}\left(1 - \exp\left(-0.5\xi_p\right)\right) \tag{7.7}$$

Section	1	2	2*	3&4*	4	5
H_o [m]	1.73	1.78	1.78	1.86	1.84	1.84
C_t [-]	0.31	0.33	0.29	0.31	0.34	0.36
B [m]	29	24	39	31	22	18
Area [m^2]	618	402	699	727	785	492

Table 7.9: Results of transmission calculations low-crested emerged breakwater

7.5.4 Submerged breakwater

The submerged breakwater has a crest height of 2 meters below storm surge level, which means that it is also below MSL and therefore cannot be seen under normal daily conditions. Unfortunately equation 7.2 cannot be used for this design, since it is only valid for emerged breakwaters. Therefore only transmission is discussed in the initial design.

Transmission

For the transmission the same method is applied as for the low-crested emerged breakwater. The submerged breakwater, however, does not fit the criterion narrow, so equation 5.67 from the Rock Manual is used, see equation 7.8.

$$C_t = -0.35\frac{R_c}{H_s} + 0.51\left(\frac{B}{H_s}\right)^{-0.65}(1 - \exp\left(-0.41\xi_p\right)) \tag{7.8}$$

The crest widths and areas for all sections can be found in table 7.10. As can be seen, the crest widths of the submerged breakwaters need to be much larger to meet the criterion of the outgoing wave height.

Section	1	2	2*	3&4*	4	5
H_o [m]	1.73	1.78	1.78	1.86	1.84	1.84
C_t [-]	0.31	0.33	0.29	0.31	0.34	0.36
B [m]	56	55	61	55	52	63
Area [m^2]	782	554	947	1090	760	799

Table 7.10: Results of transmission calculations submerged breakwater

7.6 Evaluation of breakwaters using a SWASH model

In this section the hydraulic response of the initial design of the breakwaters is reviewed with the numerical model SWASH. This is done because the transmission determined has a large uncertainty and the set-up due to the breaking waves is not taken into account.

For the low-crested emerged breakwater the formula for narrow low-crested breakwaters is used, which gives reasonable values for the crest width. However, the results for the submerged breakwater have a larger uncertainty, because the formula for wide low-crested breakwaters is used. This is only valid for crest widths over ten times the incoming wave height, which is around 6 meters. Therefore the crest width needs to be very large to meet the design criterion of the outgoing wave height. As mentioned the set-up due to breaking waves behind the breakwater is not taken into account for determining the overtopping at the seawall. To evaluate how much set-up is present and whether the results of the transmission are realistic the SWASH model is used. A description of this model can be found in appendix M. The input of the model is discussed in section 7.6.1.

Both the emerged and the submerged breakwater are evaluated with SWASH. During the design of the emerged breakwater, overtopping was calculated without taking into account what amount of set-up is present at the wall due to the breakwater. An estimate is made for this with SWASH, in order to compare the situations with and without set-up.

7.6.1 Model input

In this section the input of the SWASH model is described. This is split up into five subsections: grid, numerics, physics, boundary conditions and porous structures. All sections are based on the SWASH User Manual [SWASH team, 2015]. An example of an input file can be found in appendix M.

Grid

The SWASH model was used in non-stationary 1D mode. The computational grid has a length of 450 meters with 225 meshes, which results in a mesh size of 2 meters. For relatively high waves at least 100 grid cells per peak wave length are recommended. The peak wave length is approximately 200 meters in deep water, so the grid size is appropriate. In the vertical, two layers are used because we are interested in wave transformation. The number of layers is determined by the dimensionless depth kd, which has a value of approximately 0.5. For one layer this would result in an error of the normalised wave celerity ($= c/\sqrt{gd}$) of 3%, which is acceptable for many applications, but to be accurate two layers are used.

Physics

To represent wave transformation properly, also the command BREAKING is used, which imposes a hydrostatic distribution. This is used to initiate the onset of wave breaking and allows for the use of low-vertical resolutions (1-3 layers) throughout the domain. Furthermore the physical effect of FRICTION is applied. The Manning formula is used for a friction coefficient of 0.019 (source: [Zijlema, 2015]).

Numerics

Because short waves are present in the model, the command NONHYDROSTATIC needs to be applied. The space discretisation is done with the upwind scheme and because of breaking waves momentum conservation is applied (DISCRETE UPW MOM). The time integration is explicit and to be safe a minimum Courant number of 0.1 and a maximum Courant number of 0.5 are chosen, which are both lower that the default values.

Boundary conditions

The 1D model has the coastline on the west-side (left) and the incoming irregular waves from the east (right). At the east-side therefore a JONSWAP spectrum is assumed with a significant wave height of 5.95 m, a peak period of 12 seconds and a cycle time of 50 minutes. The significant wave height is obtained from the SWAN results 450 meters from the shore in front of section 3. The peak period is assumed to be the same as offshore and a cycle time of 250 peak wave periods is simulated. A weakly reflective boundary (command WEAK) is used at the east-side to avoid reflection of the waves, and second order bound long waves are added to the first order irregular waves (command ADDB). At the west-side the wall is implemented 20 meters in front of the boundary, which will be discussed further in subsection 7.6.1. Behind the wall at the boundary the Sommerfeld condition is applied, to simulate water flowing through the boundary and is not being totally reflected.

Porous structures

Porous structures can be implemented as porosity layers and placed inside the computational domain to simulate reflection and transmission effects.

Seawall
The wall is implemented as such a structure, but with a very low porosity of 0.11, so it is almost impermeable. For a porosity of 0.10 the structure would function as an impermeable structure and this would be comparable to implementing the wall in the bottom grid. The value of 0.11 is chosen, because the wall itself will not function as a dry point in the model and the water can cross the wall, like in reality.

Breakwater
The breakwater is also implemented as a porous structure. Because the armour layer is a double layer of cubes the porosity is 0.47, according to the Rock Manual [CIRIA, CUR, CETMEF, 2007].

7.6.2 Emerged breakwater

The emerged breakwater is represented as a structure with a porosity of 0.47. The dimensions are based on the dimensions needed for section 3 (see section 7.5). This section has the severest hydraulic boundary conditions and therefore the largest breakwater is needed here, which makes it the governing section for the breakwater design. A slope of 1:1.5 and a crest level at MSL with a crest width of 30 meters are implemented. For these conditions a set-up of approximately 0.9 meters is generated behind the low rested emerged breakwater and 0.7 for the emerged breakwater. The transmitted wave behind the breakwater that should induce a maximal overtopping of 50 L/s/m at the seawall is 1.86 meters according to overtopping formula 7.7 from the Overtopping Manual for plain vertical wall [EurOtop Team, 2007]. If this wave height is used again, together with a maximum set-up of 0.90 meters, the overtopping will become approximately 160 L/s/m instead of 50 L/s/m for section 3. This means that the initial design of the breakwater does not meet the design criteria according to the SWASH calculation. The wave height needed to establish an overtopping of 50 L/s/m, when taking into account a set-up of 0.90 meters, is 1.40 meters. According to the design formulas the breakwater crest should be raised with 1.5 meters to comply to this new condition using the transmission formula for low-crested breakwaters, see equation 7.7. However, this emerged breakwater still induces a set-up according to the SWASH model and extra design iterations are recommended.

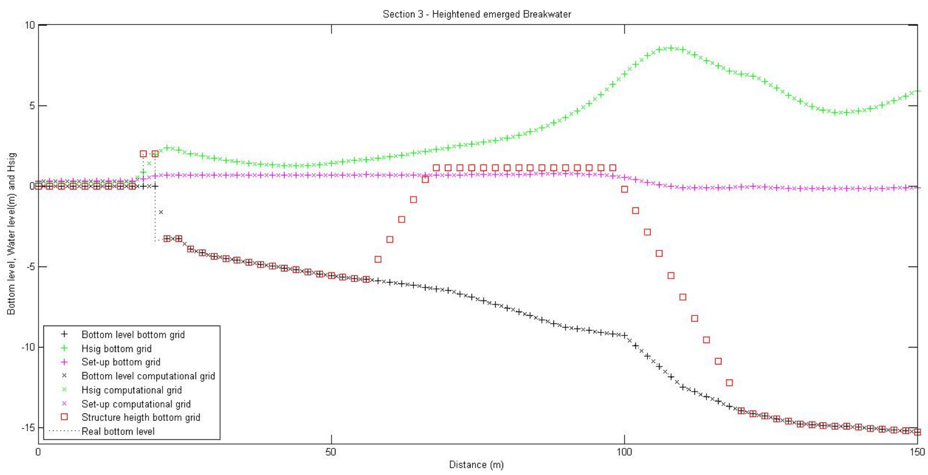

Figure 7.6: Results of the emerged breakwater in SWASH

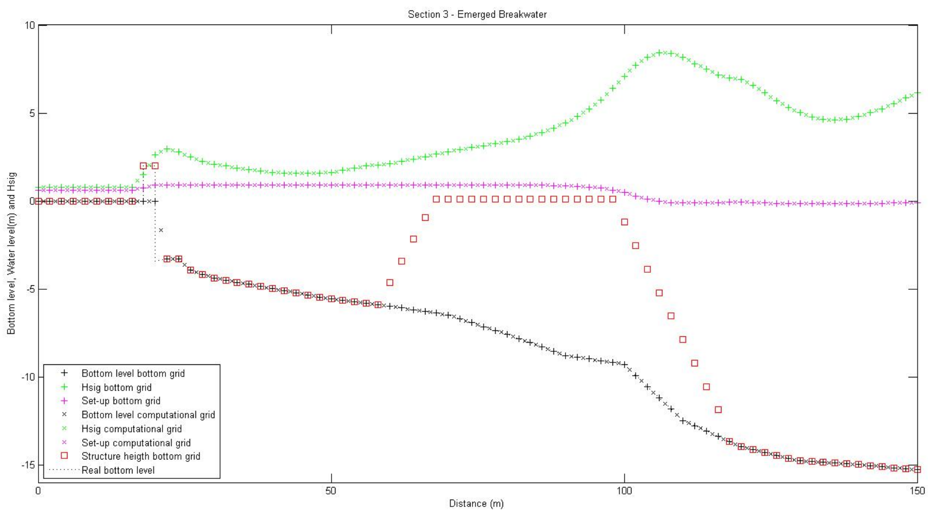

Figure 7.7: Results of the low-crested emerged breakwater in SWASH

7.6.3 Submerged breakwater

A submerged breakwater has been designed with a crest width of 60 meters and a slope of 1:1.5. From figure 7.8, it can be seen that the transmission is comparable to the emerged breakwater in subsection 7.6.2. This confirms the results of the transmission formula. The necessary material is therefore more than doubled, compared to the low-crested emerged breakwater, which makes the breakwater a very costly solution. The set-up becomes around 0.8 meters.

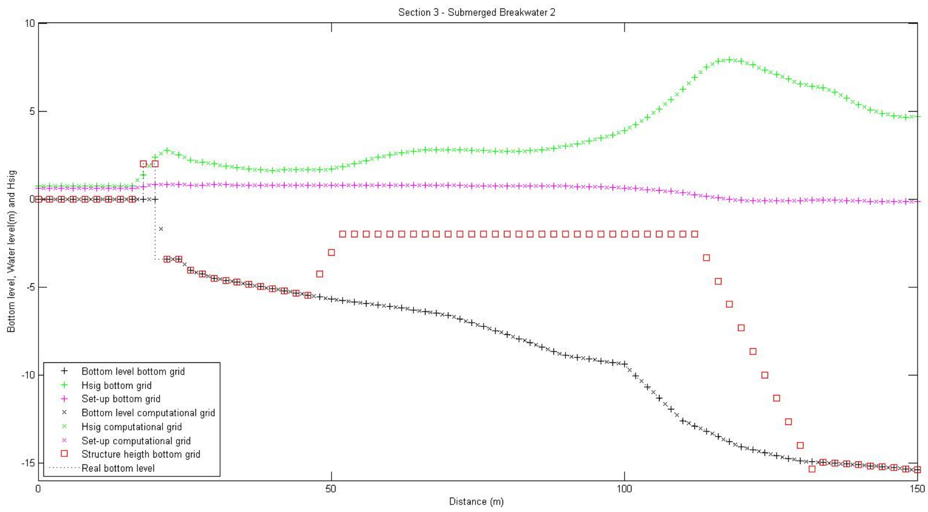

Figure 7.8: Results of the submerged breakwater in SWASH

7.7 Cost/benefit analysis

A cost/benefit analysis is used to evaluate the alternative solutions and find feasible combinations of the alternative solutions. This evaluation is done per section. The benefit of the solutions is expressed in terms of effectiveness in reducing the wave overtopping to 50 L/s/m. As indication of the costs, an estimation for the necessary material is made per alternative solution.

The resulting overtopping for all alternative solutions is displayed per section in figure 7.9. The green horizontal line corresponds to the design criterion of 50 L/s/m.
Unfortunately, it is not possible to calculate the overtopping of the raised wall for section 1, because it falls outside of the validity limits of the formulas from the Overtopping Manual. Therefore this value is not included in the graph. For all the sections the wall can be raised with half a meter, except for sections 2 and 2*, which can only be raised by 0.1 m. Therefore raising the wall in these sections does not lead to a large reduction. Overall, it can be concluded that it is impossible to reduce the overtopping enough to satisfy the design criterion by merely raising the seawall . It does, however, reduce part of the overtopping.

Using a curved wall also reduces overtopping, but although a prediction is hard to make, it will not be sufficient to fulfil the design criterion. Combining the curved wall with

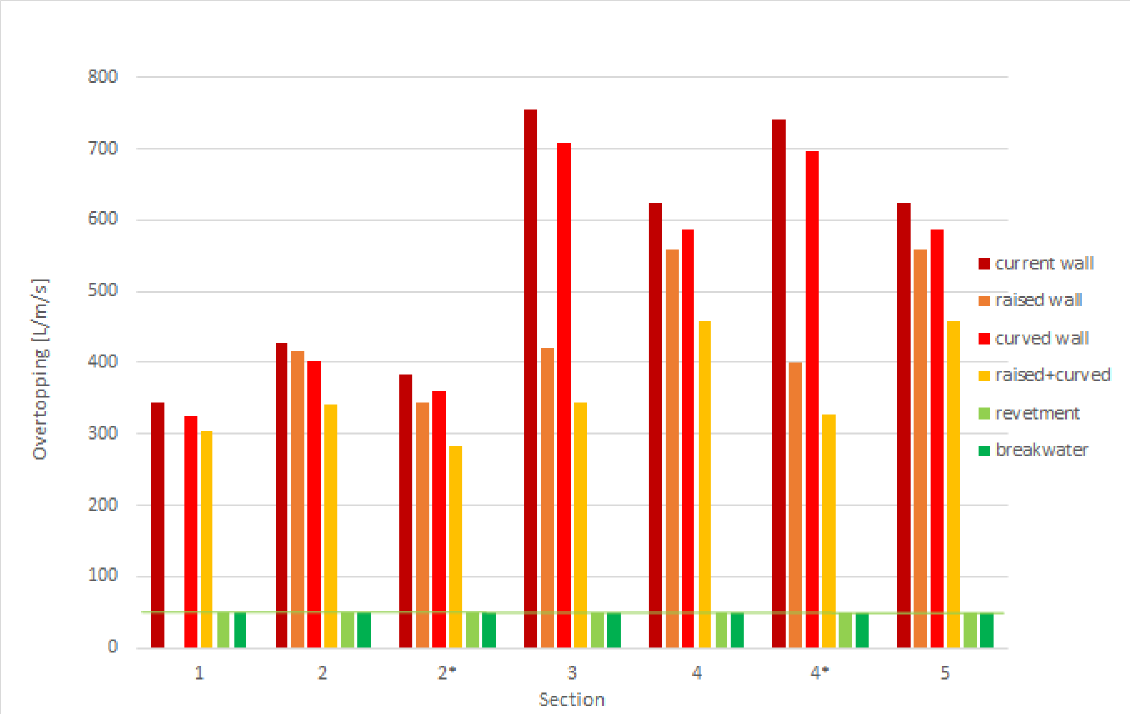

Figure 7.9: Resulting overtopping for all alternative solutions per section

raising the wall reduces the overtopping further, but still not enough. The revetment and breakwater can be designed in such a way that the overtopping is reduced to 50 L/s/m, thus fulfilling the design criterion.

From this overtopping analysis, it is clear that the revetment and breakwater are the most promising solutions, as their benefit (in terms of overtopping) is largest. Therefore these alternative solutions are compared, using an indicative estimate for the costs per solution based on the necessary material. In table 7.11 the necessary material for the cross-sectional area is given for the revetment and the breakwater in order to reduce the wave overtopping to the design criterion of 50 L/s/m. For the revetment, this is done for the current situation and for the case that the wall is raised to the maximum allowable height as given by the design criteria. For the breakwater, the cross-sectional area is only determined for the raised wall, because it is simply unrealistic to construct a breakwater without raising the wall. This is obvious from the values for necessary material.
The partial solution of a raised wall is most economic when this is combined with the necessary structural fortification, which differs per section. As section 4 and 5 have to be thoroughly strengthened for structural reasons, it makes sense to raise the top while doing this repair. Section 2 also needs thorough strengthening, but here the possible raise of the wall is limited. Section 1 and 3 do not need very extensive works for the structure. Still a raise of the wall is beneficial for section 1 and 3 as overtopping is reduced and a smaller revetment can be used.

	Raised vertical wall	Area revetment [m^2/m]	Area breakwater [m^2/m]
Section 1	0 m	23	
	+0.5 m	18	618
Section 2	0 m	18	
	+0.1 m	17	402
Section 2*	0 m	33	
	+0.1 m	32	699
Section 3	0 m	51	
	+0.5 m	43	727
Section 4	0 m	31	
	+0.5 m	23	755
Section 5	0 m	31	
	+0.5 m	23	755

Table 7.11: Area needed per solution alternatives per section

The curved wall can only fulfil the design criterion if used in combination with a revetment or a breakwater. For all sections of the Malecón seawall, constructing a revetment turns out to be the more feasible solution compared to a breakwater, based on the necessary material given in table 7.11. When the revetment is combined with a curved wall, a smaller revetment can be used, since the curved wall resolves part of the overtopping.

Only for section 3 it is necessary to combine a raise of the wall with a curved wall. In section 1, 2, 4 and 5 the revetment cannot be made smaller due to stability requirements of the revetment when one of these measures is taken. In sections 2* and 4* it is not feasible to construct a curved wall due the small section lengths. Construction for such small individual sections is not recommended to change the construction procedure multiple times. Moreover, the uncertainty of the reduction of overtopping by a curved wall, it is decided to only raise the wall in these sections.

7.7.1 Overview solutions

The best solution per section is displayed in table 7.12.
Since the chosen solutions require a raise of the seawall for all sections, the partial replacement alternative is recommended since the other alternatives are not economic or do not require the strength requirements, as was explained in section 7.2.5.

Section	1	2	2*	3	4	4*	5
Revetment	x	x	x	x	x	x	x
Breakwater							
Major structural fortification		x	x		x	x	x
Minor structural fortification	x			x			
Significant raise of the wall	x			x	x	x	x
Curved wall				x			

Table 7.12: Overview of the combinations of solutions per section

Chapter 8

Final Design

This chapter describes the design of the final solution. This follows from the outcome of the cost/benefit analysis in 7.7. In every section the final design is a combination of a overtopping solution by a revetment and a structural solution of the concrete wall. One typical cross-section is given in 8.1, in Appendix N all cross-sections are presented. Both the revetment and the structural aspects are considered in respectively section 8.3 and section 8.2.
As a breakwater might meet the specifications of the client but at a higher price, also a detailed design of this type of structure is worked out. This however is not the preferred solution of the cost-benefit analysis. A detailed design of a breakwater is included Appendix O.

8.1 Cross-section of the design

The recommended solution is a combination of placing a revetment and modifying the seawall. Per section the details of this solution differ and for each section a cross-section is presented in N. In this paragraph two cross-section are given, to visualize the differences that exist between sections along the Malecón. The first one is section 2, where only a small raise of the wall can be made and the waves are relatively small. The second one is section 3, where the highest waves hit the wall and heavy armouring is needed.

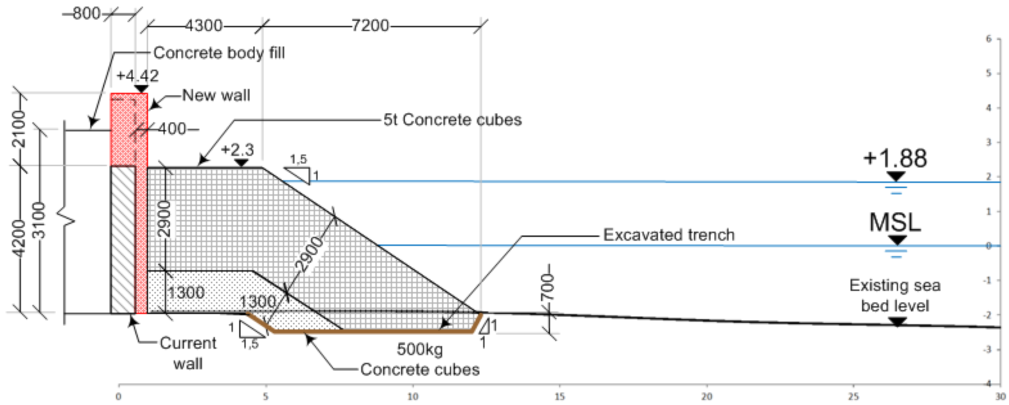

Figure 8.1: Cross section section 2 Notes: 1. All dimensions are in mm. 2. Levels are in meters relative to Mean Sea Level. 3. Storm Still Water Level is indicated at MSL +1.88 m.

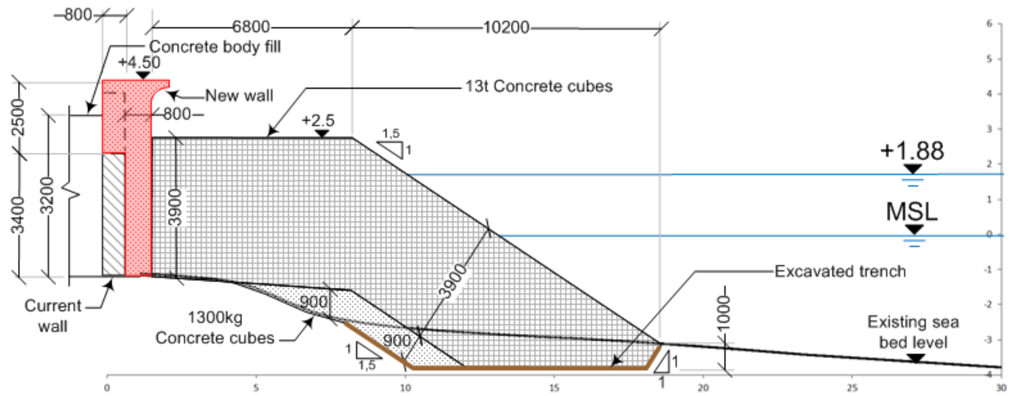

Figure 8.2: Cross-section section 3. The notes of Cross-section 2 of figure 8.1 also apply for Cross-section 3

8.2 Detailed structural design of the Malecón seawall

First the detailed design of the recommended partial replacement alternative will be discussed for every section of the wall. Then the construction method of the new Malecón seawall is described and the costs of the construction of the wall will be evaluated.

8.2.1 Detailed design

The elaborate computation to arrive to the design as presented in this section can be found in appendix K. First the detailed design for a vertical raised wall for sections 1, 2, 2*, 4, 4* and 5 is presented. Than the design approach and points of notice for the design of the curved wall for section 3 are given.

Sections 1, 2, 2*, 4, 4* and 5

The results for the different sections are presented in table 8.1. The cross sections of the solutions can be found in appendix N.

Section	1	2 & 2*	4 & 4*	5
Height remaining part old wall [m]	2.3	4.20	2.3	2.3
Thickness bottom part of new wall [m]	0.40	0.40	0.40	0.40
Height top part of new wall [m]	2.0	2.1	2.2	2.2
Thickness top part of new wall [m]	1.20	1.20	1.20	1.20
Cross section [m^2]	3.3	4.2	3.6	3.6
Longitudinal reinforcement	$\oslash$20 - 90 mm			
Shear reinforcement	$\oslash$12 - 200 mm and $\oslash$12 - 150			

Table 8.1: Results detailed structural design per section

The longitudinal reinforcement used complies with the strength requirement in equations K.4 and K.3 due to stresses in the concrete during ULS conditions, the minimum reinforcement ratio and the crack width criterion as given in section K.1.5 in the appendix. This leads to a reinforcement of $\oslash$20 with a spacing of 90 mm in the vertical direction. In horizontal direction only a minimal amount of reinforcement should be placed. It is recommended for the constructor to determine the exact amount to ensure customary reinforcement cages.

The shear reinforcement can be limited to the minimum value as the shear stresses in the new concrete are not much higher than the shear strength of the concrete itself, as was evaluated with the use of ANSYS. The minimum shear reinforcement ratio is 0.09% as computed in section K.1.3 in appendix K. For the top part of the wall with a thickness of 1200 mm this leads to a reinforcement of $\oslash$12 with a spacing of 150 mm. For the bottom part of the new wall the thickness is 400 mm the reinforcement used will be $\oslash$12 with a spacing of 200 mm. Both reinforcement spacings used meet the maximum spacing requirement as discussed in K.1.3 in the appendix. A visualisation of the reinforcement design can be found in figure 8.3.

Section 3

As stated in section 7.7 it was discovered that it is useful to build a curved wall in section 3. For the design of the curved wall the detailed design of the partial alternative is taken and a curvature is added. The dimensions of the curvature can be found in section 7.3. Pearson et al discussed the forces on the curved wall. From experiments is follows that the pressures on the curved wall will be twice the pressures on the straight vertical wall. Given the pressures on the vertical wall in section 4.4, the pressures on the curved wall can be determined. With these pressures the stress distribution of the curved wall is computed in ANSYS for section 3. The dimensions computed in section K.4 in the appendix are used for the curved wall as well. Which entails a height of the interface of 4.0 m and a thickness of the new bottom part of 0.40 m. The new stress distributions for the curved wall are given in figure L.21, L.22 and L.23 in appendix L.

It can be concluded that the shear stresses exceed the maximum allowable shear stress in the concrete. Especially for the old wall the exceeded shear stresses cause a problem, where the maximum allowable shear stress is 0.22 MPa. The shear stresses of the old wall can be seen in figure 8.4. Therefore it can be concluded that the bottom of the old wall will fail. Also it can be questioned whether the shear reinforcement in the new concrete will be enough in order to resist the shear stresses. It is recommended to have a closer look at the actual pressures on a curved wall and compute a proper shear reinforcement distribution, before a final design is made. From figure L.21 it follows that the tensile stresses do exceed the strength of the concrete. Constructing a curved wall with a greater thickness solves this problem. With a thickness of 0.80 m the tensile stresses do not exceed the strength which can be seen in figure L.25 in the appendix L. As the thickness of the wall changes a new detailed design for the longitudinal reinforcement is recommended as well. The cross section of the solution can be found in appendix N. The dimensions are presented in table 8.2.

	Section 3
Height remaining part old wall [m]	3.4
Thickness bottom part of new wall [m]	0.80
Height top part of new wall [m]	2.5
Thickness top part of new wall [m]	1.20
Cross section [m^2]	6.7
Longitudinal reinforcement	*Should be evaluated in more detail*
Shear reinforcement	*Should be evaluated in more detail*

Table 8.2: Results detailed structural design for section 3

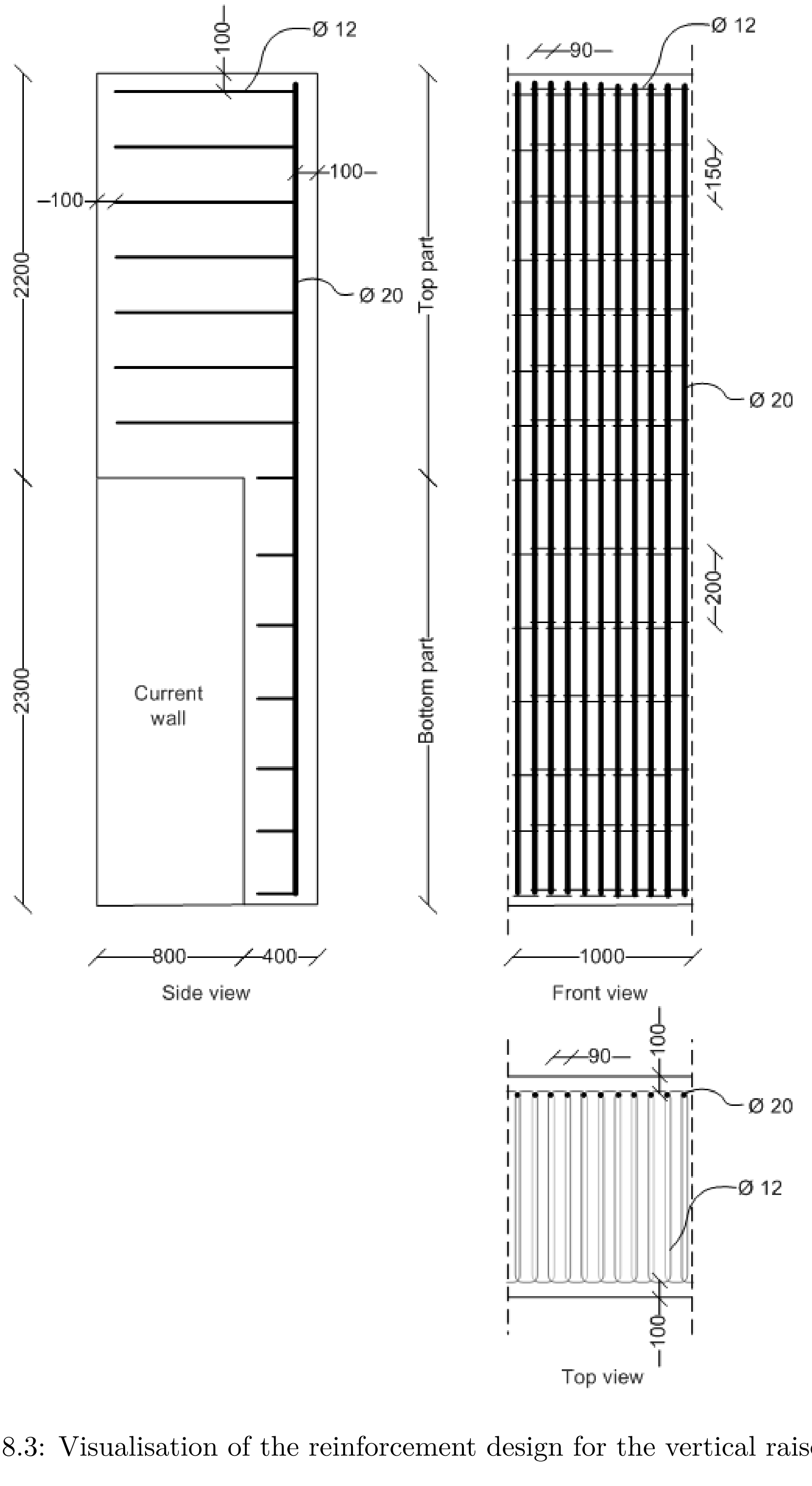

Figure 8.3: Visualisation of the reinforcement design for the vertical raised wall

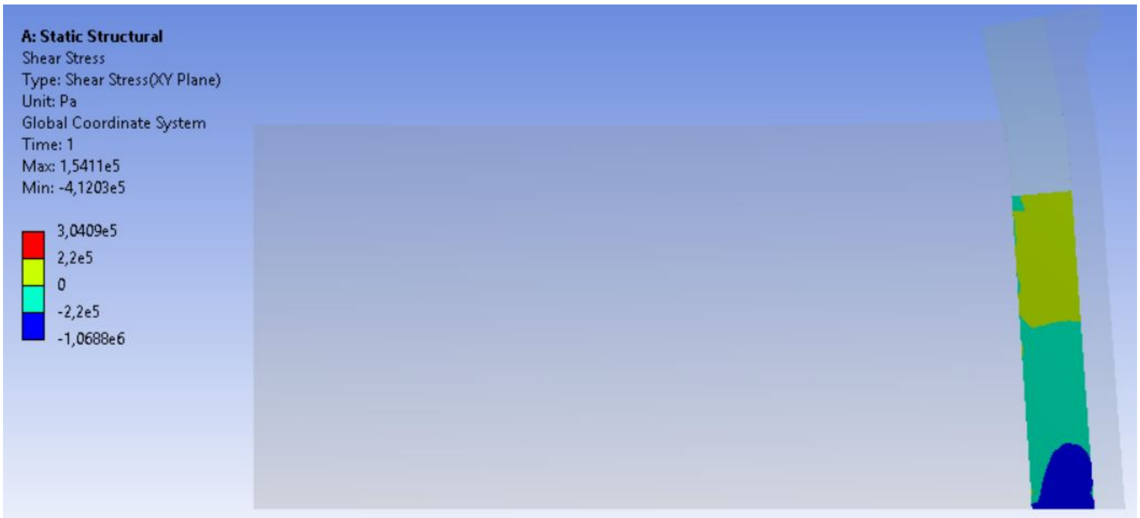

Figure 8.4: Shear stresses in the old wall with a curved wall for section 3

8.2.2 Construction method

The partial replacement construction procedure in more detail can be:

1. Work around the hurricane and cyclone season,
2. Remove scour protection located immediately adjacent to the seawall and carefully check the state of the concrete wall behind the protection,
3. Pressure wash the seaward face of the seawall,
4. Remove the zones of soft and weakened concrete along the seaward face of the seawall,
5. Remove top part of the wall,
6. Either sand blast or water blast the new interface surface of the existing concrete substrate to achieve satisfactory bond preparation,
7. Drill holes in the top and seaward surface as necessary to install and epoxy grout new steel reinforcing dowels and install dowels,
8. Place total formwork (possibly curved) and remove water inside,
9. Place reinforcement cages,
10. Construct a new concrete seawall,
11. Let the concrete cure sufficiently before removing the formwork.
12. Remove and replace the sidewalks where necessary,
13. Remove formwork and place additional stone riprap around the seaward perimeter of the new seawall.

The construction steps that consider repairing of the old seawall are based on the maintenance works of the Low Battery seawall in Charleston, South Carolina [Cummings and McCrady, 2004]. This seawall is built in a similar climate and period as the Malecón Tradicional and several sections are in a comparible condition as the Low Battery seawall. The placement procedure of the top part of the wall is based on the procedure proposed for the Isle of Man Sea Defence Options [Dane, 2014].

8.2.3 Construction costs

The cost estimation in Cuba differs from cost estimation in the Netherlands. The prices and methods are adapted from a manual distributed by the Cuban department of construction [Dirección de Presupuestos y Precios del Ministerio de la Construcción, 2005]. Appendix P describes this method for cost estimation in Cuba. The total construction costs for the new seawall amount between 6 and 9 million CUC. Figure 8.5 shows the distribution of the costs over the different items. It can be seen that the majority of the costs consists of material costs (roughly 50 %). Other important costs are labour (15 %) and indirect costs (20 %) for preparations of the site, etc. The total overview of the cost calculation can be found in figure P.1 in the appendix.

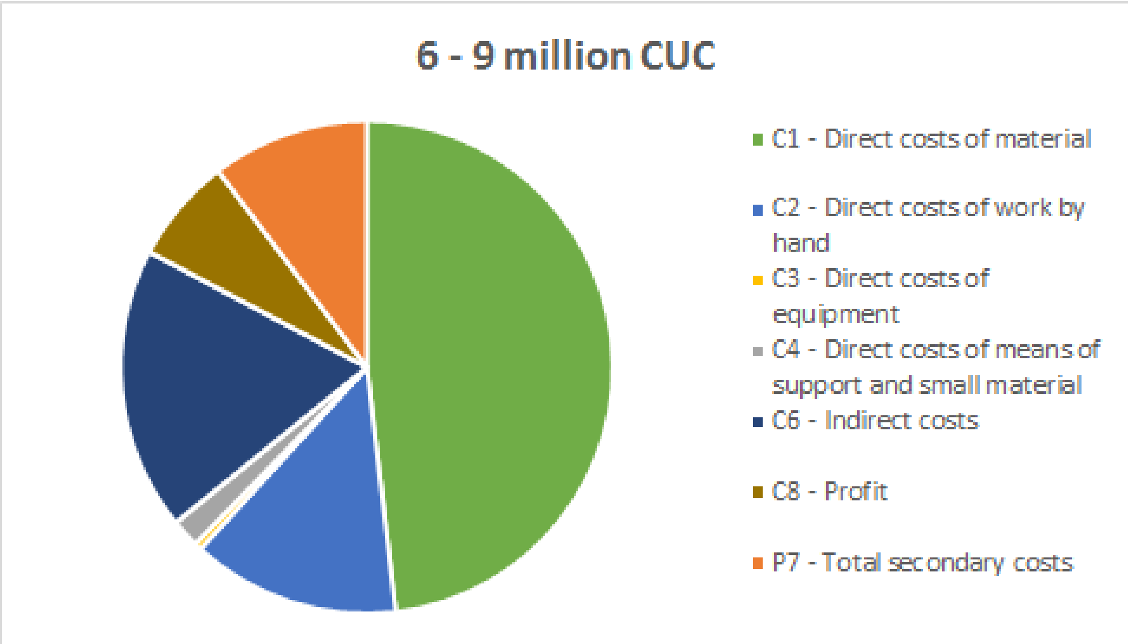

Figure 8.5: The split of the costs for the wall

8.3 Revetment

This section specifies details of the design of a revetment as solution to the overtopping problems. The cross section, dimensions and construction method are presented. The dimensions of the revetment continue on the initial design of the revetment described in section 7.4. This means the revetment has a two-layer concrete cubes armour layer, an 1:1.5 slope and a crest width based on necessary overtopping reduction.

8.3.1 Dimensions

The design details in this section are based on information from section 6.3 of the Rock Manual, Design of shoreline protection and beach control structures [CIRIA, CUR, CETMEF, 2007].

Armour layer

The method to design the armour layer is described in paragraph 7.4.3. The cubes will be randomly placed, as this configuration gives the best reduction of overtopping and reflection.

Crest

According to [CIRIA, CUR, CETMEF, 2007] the material of the crest is the same as the armour layer also placed in a double layer. The width of the crest is based on overtopping requirements and a minimum crest width of $3 * k_t * D_n$ for stability requirements. Although the construction is land-based, no widening of the crest is necessary, see also the construction method in section 8.3.2. The crest has to protrude above still water level during design conditions to be effective to reduce overtopping, as stated in section 7.4.2. If an armour layer placed on the rockbed does not fulfil this requirement, an underlayer is placed of one or two layers so that the crest protrudes the design storm still water level by at least half a meter.

Underlayer

An underlayer is necessary in sections 2, 2* and 3. The ratio between the mass of underlayer units and armour units should be $\frac{1}{10}$, as recommended by the Rock manual chapter 6.[CIRIA, CUR, CETMEF, 2007]. Assumed is that that the weight of individual units in an underlayer of concrete cubes does not differ from stones in an underlayer of rock. The underlayer is constructed as a double layer where possible.

Contact with the bed

Core material is not necessary, also an underlayer is some section not necessary. Due to the relatively large armour stones and underlayer compared to the structure height, the layer thickness of the armouring (and underlayer) combined is sufficient to place directly on the rockbed. In section 1, 3, 4, 4* and 5 the armour cubes are even so large that the armouring can be placed on the bottom, only a few underlayer cubes have to be placed to ensure a random placement of the armour layer. Also if large irregularities of the rockbed exist, like the cut-outs described in 5.1, this needs filling by underlayer material. Even though the lack of need of core material is economic, there might be problems with the friction between the bed and the structure. Although this probably will not be a problem, it is wise to study this behaviour in more detail and take measures to increase friction when necessary.

Toe

The toe stability is essential for the whole armour layer stability. The same armour units will be used as for the slope, no reduction in armour size is possible because of the limited depth. The sloping rock foreshore provides limited sliding resistance for the toe. Therefore to prevent brittle failure the toe support is achieved by excavating a trench of a depth of 0.5 D_n armour stone, which accommodates the armour layer and the underlayer. This avoids the need to drive piles and no further maintenance is needed. Hence, there is no offset.

Drainage pipes passing through the revetment

On multiple places drainage pipes pass through the Malecón. This creates discontinuities in the rock structure and are potential weak spots, so they should be considered carefully. The pipes should be renewed and extended and fortified from its current location at the wall into the structure. It is not possible to stop the pipe in the underlayer and prevent weak spots in the defence, but it has to protrude through the armour layer. This is because of the poor quality of the drained water and it should wash out properly for environmental reasons. It is more appropriate to use concrete around the pipe to increase the stability of the armour and underlayer to prevent settlement. Currently, a substantial amount of water flows into the city during storms. To prevent this from happening, non return valves should be implemented during the construction of the drainage system. It is recommended to make a more detailed design of these points, which is beyond the scope of this project.

Overview

An overview of the materials that are used for the revetment elements is given in table 8.3.

section	1	2	2*	3	4&5	4*
D_n armour layer and crest [m]	1.30	1.28	1.62	1.77	1.38	1.77
Mass armour layer and crest [t]	5.1	4.9	10.1	13.1	6.2	13.1
t_a armour layer and crest [m]	2.9	2.9	3.57	3.90	3.04	3.90
B crest width [m]	4.3	4.3	6.4	6.8	4.6	7.3
D_n underlayer [m]	-	0.59	0.75	0.82	-	-
Mass underlayer [kg]	-	495	1007	1309	-	-
t_u underlayer	0	1.3	0.8	0.9	0	0

Table 8.3: Dimensions of the revetment per section

8.3.2 Construction method

The design details in this section are based on information from section 9.7 of the Rock Manual, Work methods [CIRIA, CUR, CETMEF, 2007].

A land-based construction is selected, because this is normally more cost-efficient than marine placing. Also the limited water depth in front of the wall is an obstacle for water-borne equipment.

As the revetment is located besides Malecón seawall, there may be a possibility to place the construction equipment on the Malecón road and work over the wall. The practical execution, however, is questionable. The method is described based on the assumption that this will be possible. Machinery will be placed on the road of the Malecón, which will lead to (partial) closure of the road. By doing so, the design of the revetment does not have to be revised. The crest does not have to accommodate equipment, so the crest width remains limited.

The underlayer of concrete cubes can be placed by excavators. Excavators place and trim the material in the desired position. Where needed, gaps in the current natural berm are filled with underlayer material. The toe requires excavation in bedrock, here a specialist rock breaker is needed. The armour layer has to be placed by crawler cranes and the blocks have to be gripped with clamps. The cranes are placed on the road of the Malecón and special attention has to be on preventing accidental demolition of the wall.

Note if execution turns out to be impossible, or closing of the road is very undesirable, the revetment can be constructed otherwise. A first option would be to construct using water-borne equipment. This would increase the costs, but the design and construction method of the breakwater will be the same. Another option is to work with equipment based on the core of the revetment itself. However, since this has a major impact on the total design (not only the revetment), this method is most likely to be more expensive. When this method is selected, a core should be placed up to mean sea level plus 0.5 meter with a core crest width up to 8.50 meters for safe and smooth use. This would lead to a crest level of the revetment higher than the crest of the wall, which conflicts with the design criterion of preserving the view of the city Havana.

8.3.3 Construction costs

A rough calculation is made for the costs of the revetment. The total costs of the revetment amount between 16 and 20 million CUC. Figure 8.6 shows the distribution of the costs over the different items. It can be seen that the majority of the costs consists of material costs (roughly 40 %). Other important costs are labour (10 %), equipment (10 %) and indirect costs (20 %) for preparations of the site, etc. The total overview of the cost calculation for the revetment can be found in figure P.2 in the appendix.

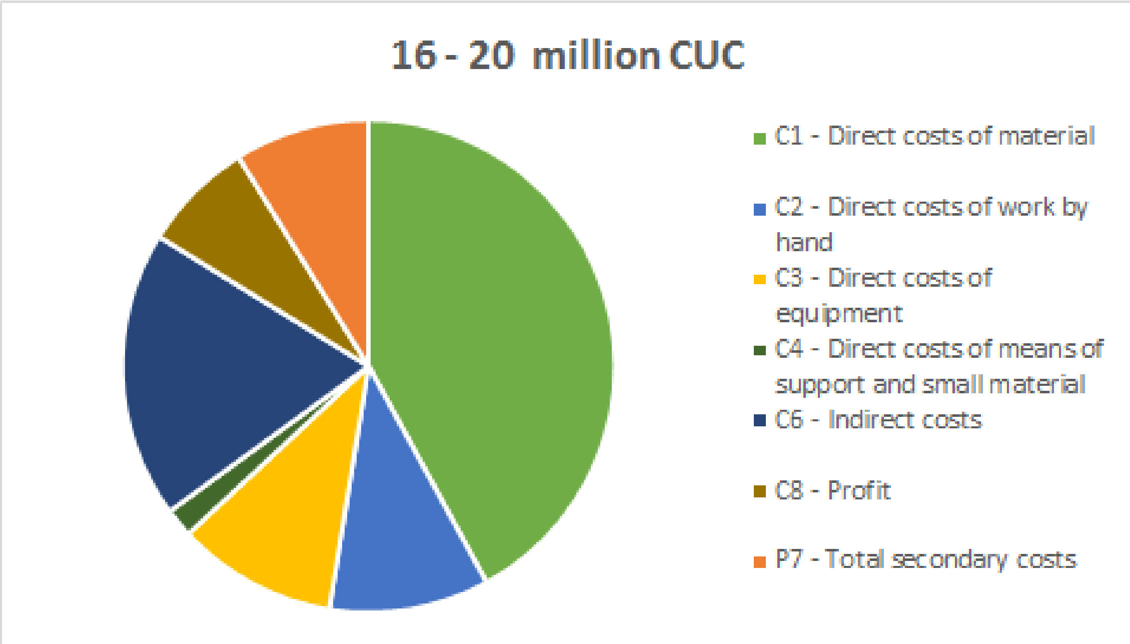

Figure 8.6: The split of the costs for the revetment

8.4 Evaluation of overtopping reduction

The resulting overtopping and effectiveness of the solutions are displayed in table 8.4. These are cummulative solutions, so it is assumed that the solutions are implemented from left to right. For example, in section 3 the reduction effectiveness of 58 % entails the reduction after raising and curving the wall. If the revetment would also be constructed, it would lead to an effectiveness of 100 %, thus satisfying the design criterion of 50 L/s/m.

The results for the curved wall are only included in the table for section 3, as this is the only section where the use of a curved wall is advised.

The overtopping for the raised wall could be computed but, due to the transition between overtopping formulas, it resulted in larger overtopping quantities. Since this is not realistic, the no reduction in overtopping is included in the table for only applying the raised wall in section 1.

Section	Current wall	Raised wall		+Curved wall		+Revetment	
	q	q	effectiveness	q	effectiveness	q	effectiveness
1	345	345	0 %			50	100 %
2	427	416	3 %			50	100 %
2*	384	344	12 %			50	100 %
3	754	420	47 %	344	58 %	50	100 %
4	624	558	11 %			50	100 %
4*	741	400	49 %			50	100 %
5	624	558	11 %			50	100 %

Table 8.4: Resulting overtopping in L/s/m and effectiveness for cummulative solutions

It is possible to divide the construction works of the partial solutions. For example, first the new (raised and recurve) wall can be constructed in order to reduce part of the overtopping. The revetment can then be constructed at a later stage. The resulting overtopping per section from table 8.4 can be used, when making this decision. If, for example, it is decided to construct the revetment five years after constructing the new (raised) wall, the design life of the temporary structure would be five years. However, it should be noted that if the revetment is not constructed, a scour protection is necessary in front of the seawall to prevent further damage to the wall. This introduces extra costs.

Also, it is important to consider the design life of this temporary solution. Next, the probability of failure can be determined for a storm with a certain return period. If the design life of the temporary construction is chosen wisely, the probability of failure can be minimised.

Chapter 9

Conclusion & recommendations

9.1 Conclusion

The Malecón seawall does not fulfil its functional requirements in its current state. Flooding of the city of Havana would be unacceptable and measures are needed to protect the city. Furthermore, the structure of the seawall is known to have failed during severe weather conditions. Therefore, reduction of the wave overtopping and structural fortification of the wall are necessary to improve the living conditions at the Malecón, and also to make it more attractive for tourists. The provided solution is designed for a lifetime of 50 years with a maximum allowable wave overtopping of 50 L/s/m for a storm event with a return period of 50 years.

The hydraulic boundary conditions with a combined return period of 50 years have been established. The governing offshore significant wave height equals 7.80 m. The governing storm surge level equals 1.88 m, including astronomical tides and sea level rise. Using a SWAN model the wave height is transformed, resulting in a maximum significant wave height of 4.27 m nearshore. The wave height differs considerably along the seawall. Therefore the seawall has been divided into five sections and two subsections. Then, a representative significant wave height was selected per section.

From the initial structural assessment it was concluded that the seawall in its current condition cannot withstand a storm with a return period of 50 years. The wall fails locally due to high tensile and shear stresses at sidewalk level, which may cause the top of the wall to break off.

Based on a multi criteria analysis four feasible alternatives have been selected: a curved wall, a revetment, a submerged breakwater and an emerged breakwater. The option of raising the wall was included as partial solution, as it can easily be combined with another alternative to increase the effectiveness. It was concluded that it is impossible to reduce the overtopping enough to satisfy the design criterion by merely raising or curving the seawall. The revetment and breakwater can be designed in such a way that the overtopping is reduced to 50 L/s/m.

The partial solution of a raised wall is most economic when it is combined with the necessary structural fortification. The curved wall can only fulfil the design criterion if used in combination with a revetment or a breakwater. For all sections of the Malecón seawall, the design of a revetment turned out to be a more economic solution than the design of a breakwater. Therefore, it is recommended to construct a revetment in all sections. When the revetment is combined with a raised or curved wall, a smaller revetment can be used, since part of the overtopping is resolved by raising or curving the wall. The best solution per section is displayed in table 9.1.

Section	1	2	2*	3	4	4*	5
Revetment	x	x	x	x	x	x	x
Breakwater							
Major structural fortification		x	x		x	x	x
Minor structural fortification	x			x			
Significant raise of the wall	x			x	x	x	x
Curved wall				x			

Table 9.1: Overview of the combinations of solutions per section

Both the top part and the seaside face of the Malecón seawall need a structural fortification in order to withstand the design conditions. For this fortification a partial replacement alternative is recommended for all sections. This entails partly removing the top of the current wall and replacing it with a new wall, which is extended in seaward direction. The solution assures that crack width is limited to prevent corrosion, since the structure is placed in salt water. The total costs for the implementation of the new seawall along the entire study area amount to approximately 6-9 million CUC.

A detailed design was presented per section, including dimensions, types of materials and a construction method. Concrete cubes are used as armour material of the revetment with a slope of 1:1.5. The total costs for the implementation of the revetment along the entire seawall amount to approximately 16-20 million CUC.

Concluding, the overtopping will be reduced to 50 L/s/m for a storm with a return period of 50 years along the entire seawall, if the recommended solutions are implemented. Both for a storm with a return period of 50 years and for the ultimate limit state, the proposed solution satisfies the strength requirements.

9.2 Recommendations

In order to reduce the overtopping to 50 L/s/m for a storm with a return period of 50 years, it is recommended to implement the solutions per section as proposed in this report. The transitions between the sections have not been designed yet. Therefore it is recommended to look into the design of these transitions. Additionally, there is a substantial uncertainty in the overtopping quantities as computed using the formulas from the Overtopping Manual. Therefore it is recommended to perform a physical model test to confirm the overtopping quantities for the proposed design.

In chapter 7 it was already mentioned that the overtopping was investigated with the numerical model SWASH for an emerged and a submerged breakwater. It is recommended to look into this result further and validate the numerical model with a physical model test. That way, the model can be developed into a tool that can be used to evaluate the overtopping for the breakwaters, as well as other structures. For example, the design of the revetment could be evaluated with the SWASH model as well.

The solutions presented in this report are based on a maximum overtopping of 50 L/s/m. This greatly reduces the amount of overtopping during the design storm and therefore lowers the flood risk. This flood risk reduction should be evaluated more thoroughly, taking into account overtopping as well as rainfall and drainage. The overtopping used in this report is a peak value. In order to obtain representative overtopping volumes for the entire storm duration, the behaviour of the storm could be researched. Based on the development of wave heights during the storm, the overtopping volumes can be found for the entire storm duration. Using topographic data of the land area behind the Malecón, the area that is subject to flooding can be evaluated both for the current situation as well as for the presented solutions.

The design criteria demand a design for a storm with a return period of 50 years and a design lifetime of 50 years. It is important to note that the return can be selected separately from the design lifetime. Based on advisement by an engineering firm, a 50 year return period is normal in developing countries for areas that are not of high economic or cultural importance. The area protected by the Malecón seawall does not fit in this category. On the other hand, since Cuba is located in a hurricane area, the hydraulic loads increase rapidly for higher return periods. In Cuba, the increase in construction costs is so large that it may be unrealistic to design for a higher return period. In order to quantify the differences between design lifetime and return period, the probability of exceedance of a storm can be computed for the given design criteria. This probability of exceedance can be determined using equation 9.1 (rewritten from equation 4.5) and figure 9.1 with T_R being the return period n the design lifetime of the structure in years.

$$T_R = \frac{1}{1 - (1 - \frac{p}{100})^{1/n}} \tag{9.1}$$

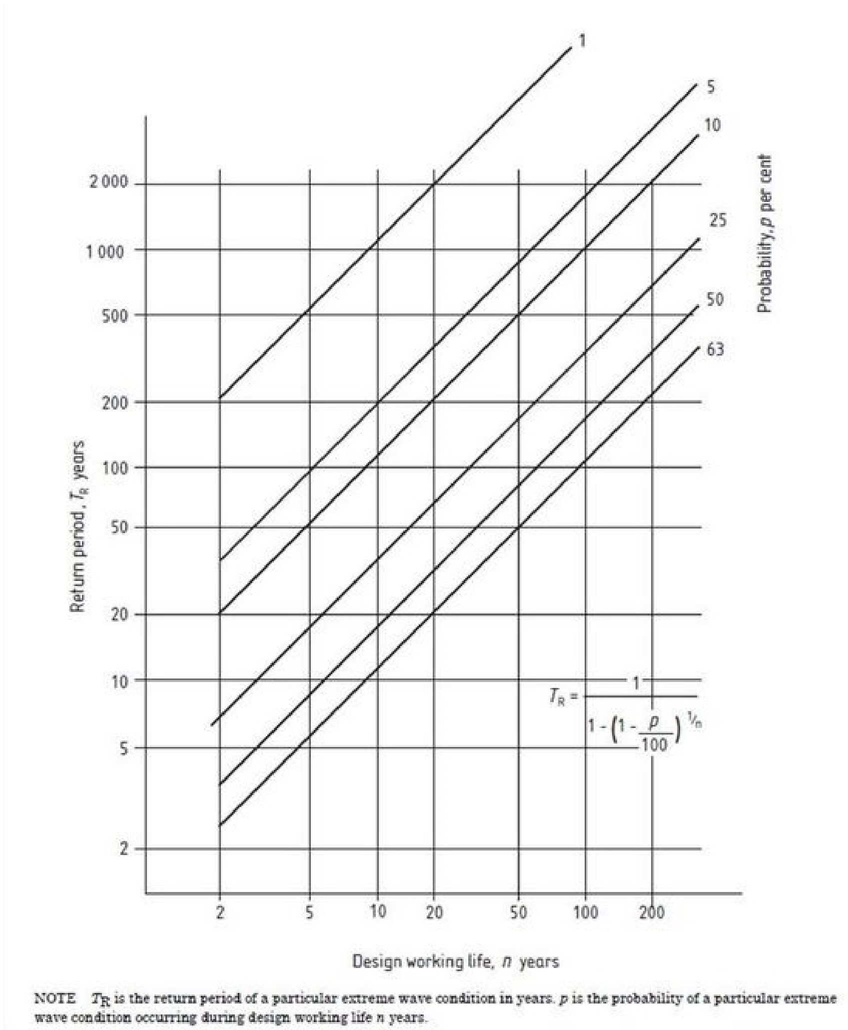

Figure 9.1: Relationship between design lifetime, return period and probability of wave heights exceeding the design condition (courtesy of British Engineering Standards)

The probability of exceedance for a storm with a return period of 50 years equals 63% during the design lifetime of 50 years. So the probability that a more severe storm occurs during these 50 years is equal to 63%. It is important to be aware of this probability of exceedance, when selecting a design return period. It may be worthwhile to select a larger return period in order to increase safety during the lifetime. Therefore, it is recommended to take the relation between design lifetime and return period into consideration.

Furthermore, for the structural fortification of the Malecón seawall quite some assumptions have been made. First of all it is advised to verify the used modulus of elasticity for the existing wall. Especially the stiffness of the concrete body, on which the Malecón road has been constructed, needs a more elaborate investigation. Also it is recommended to do a more elaborate study on the pressures of a curved wall. In the article 'Effectiveness of recurve walls in reducing wave overtopping on seawalls and breakwaters' [Pearson et al., 2005] it was stated that the pressures on a curved wall are twice as high as the pressures on a vertical wall. As there are limits on the maximum stresses in the concrete, this results into a greater thickness for the curved wall. Therefore the curved wall may become a less economic solution even though it will help to reduce the overtopping.

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Appendix A

Storm events

This appendix describes wave data for storm events used in section 3.3. Table A.1 and table A.2 contain data for tropical cyclones and cold fronts respectively. Figure A.1 contains a wave rose, showing the frequency of occurrence of each wave direction.

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

Table A.1: Historical wave data on hurricane events

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

Table A.2: Historical wave data on cold front events

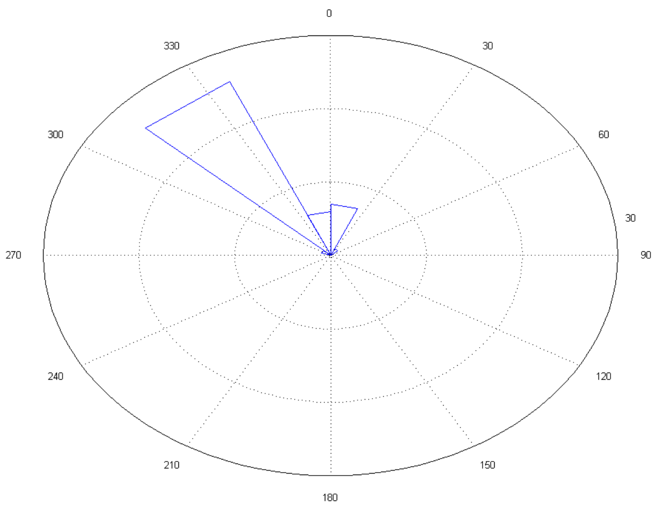

Figure A.1: A wave rose of the frequency of occurrence of each wave direction

Appendix B

Extreme value determination

B.1 Peak-over-threshold

In order to extrapolate the wave data from section 3.3 a peak-over-threshold method is used with a threshold of 2.50 m. First the events are categorised into discrete 0.50 m bins of wave heights based on the highest wave per event. In table B.1 the results are listed with N being the total amount of recorded waves per category.

Bin no.	**Range of wave heights** [m]	**No. storms**
1	2.50 - 3.00	1
2	3.00 - 3.50	6
3	3.50 - 4.00	17
4	4.00 - 4.50	3
5	4.50 - 5.00	5
6	5.00 - 5.50	7
7	5.50 - 6.00	6
8	> 6.00	0

Table B.1: Categorisation of storm events into discrete 0.50 m bins with a threshold of 2.50 m

B.2 Distribution calculations

Now that the number of waves is known for each category, the extreme wave height can be calculated for a given design condition and corresponding return period. This will be estimated by fitting different probability distributions, i.e. an exponential, a Gumbel and a

	Probability of non-exceedance of a wave:	$P = P(H_s \leq \hat{H}_s)$
	Probability of exceedance of a wave:	$Q = 1 - P$
Weibull distribution.	Probability of exceedance of a storm:	$Q_s = N_s * Q$
	Gumbel reduced variable:	$G = -\ln\left(\ln\left(\frac{N_s}{N_s - Q_s}\right)\right)$
	Reduced Weibull variable:	$W = -\ln Q^{1/\alpha}$

B.2.1 Exponential distribution

The storm wave heights are plotted in a graph against the probability of exceedance of a storm Q_s. In order to extrapolate the data a logarithmic line can be fitted using the excel function for trendlines. Combined with an inverse logarithmic axis the resulting distribution is an exponential distribution. Thus, the storm wave can be found for a certain return period. Figure B.1 represents the exponential fit with storm wave heights plotted against the storm exceedance probability on inverse logarithmic scale.

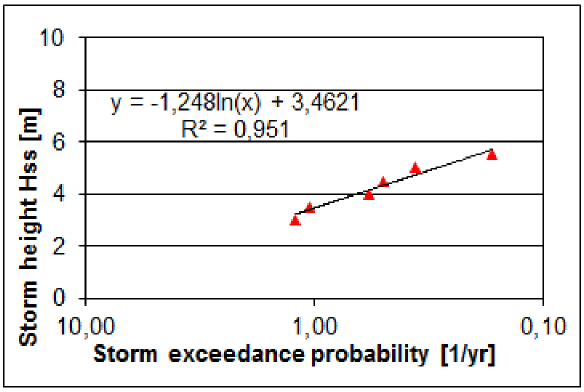

Figure B.1: Storm wave heights plotted against the storm exceedance probability on inverse logarithmic scale

B.2.2 Gumbel distribution

Using a Gumbel distribution the storm wave height is calculated as follows:

$$Hss = \gamma - \beta * \ln(\ln(\frac{N_s}{N_s - Q_s})) = \gamma + \beta * G \tag{B.1}$$

In which:

N_s = number of waves per year
Q_s = design condition

Here the excel function for trendlines is used to fit a linear line through the data. γ and β are found by use of this extrapolation. Figure B.2 represents the storm wave heights plotted against the Gumbel reduced variable.

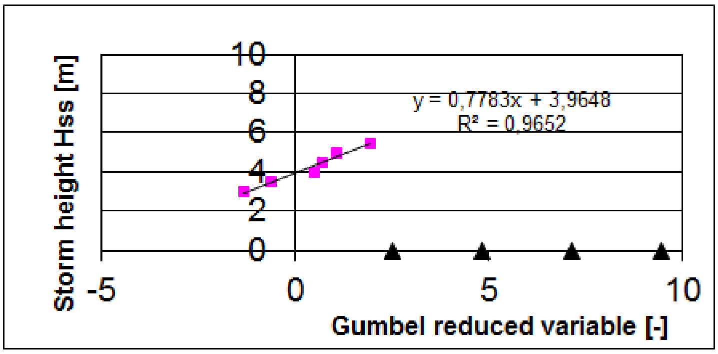

Figure B.2: Storm wave heights plotted against the Gumbel reduced variable

B.2.3 Weibull distribution

Using a Weibull distribution the storm wave height is calculated as follows:

$$H_{ss} = \gamma + \beta * -\ln(\frac{Q_s}{N_s})^{1/\alpha} = \gamma + \beta * W \tag{B.2}$$

When the wave height is plotted as a function of W, γ and β represent the value of y for which x equals zero and the gradient of the function respectively. The value α produces a correlation between W and the wave height of approximately one.

Appendix C

Finding a combination of H_s and ζ for a fixed return period

C.1 Correlation between variables

There are three possible relations between significant wave height H_s and water level elevation ζ from a statistical point of view: *I* full dependence, *II* full independence or *III* a partial correlation. These relations determine how the significant wave height and the water level elevation have to be combined for a fixed return period. This appendix describes how these combinations are determined. Figure C.1 shows these combinations and the figure highlights the governing combination that is selected.

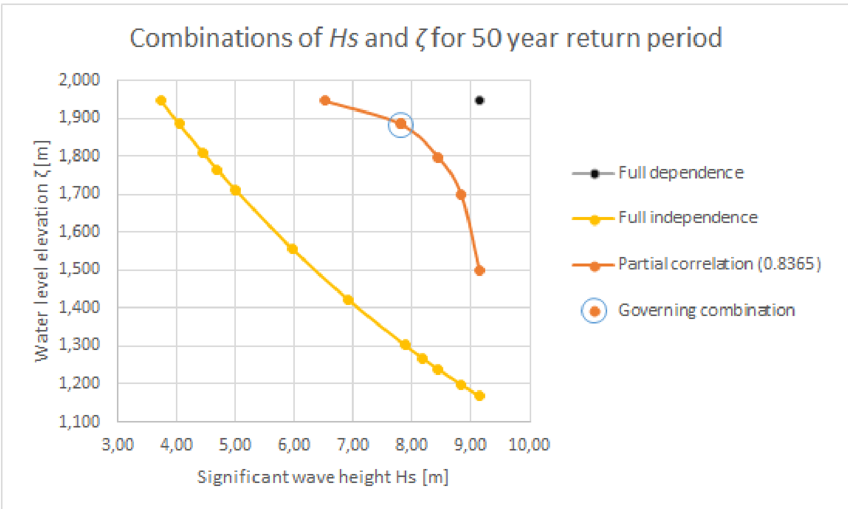

Figure C.1: Combinations of significant wave heights [m] and water level elevations [m]

I Full dependence Full dependence implies that the waves are always accompanied by water levels of the same probability. So for a design event with a probability of $\frac{1}{50}$ the only possibility is that both the wave height and the water level have values corresponding to

a probability of $\frac{1}{50}$.
This leads to the most conservative values, but in reality it is very unlikely that the moment of a $\frac{1}{50}$ wave height occurs simultaneously with the $\frac{1}{50}$ water level. [1]

$$P\left(E_1 \cap E_2\right) = min[P\left(E_1\right), P\left(E_2\right)] P\left(E_1 \cap E_2\right) = 1/50 = min[1/50, 1/50] \tag{C.1}$$

II Full independence Full independence implies that the waves and water levels are not related to each other. The probability of occurrence of one variable does not change when the other variable is given. To demonstrate this in our case: in the event of a $\frac{1}{50}$ wave height, the water level as a function of the probability is the same as the water level as a function of the individual return period. So to select an event (as a combination of wave height and water level) with a combined probability of $\frac{1}{50}$, now a range of possibilities is possible. There are two combination containing the maximal wave height or water level. Combination 1 is an $\frac{1}{50}$ H_s with $\frac{1}{1}$ ζ and combination 2 is a $\frac{1}{1}$ H_s with $\frac{1}{50}$ ζ. Of course other possibilities exist, as a $\frac{1}{25}$ H_s with $\frac{1}{2}$ ζ, but for now these are neglected as they do not lead to the highest H_s or ζ.
In reality it is however questionable whether wave heights and water levels are completely independent since meteorological phenomena responsible for the generation of waves are also able to cause a water level elevation.

Combination	$P(H_s)$	H_s	$P(\zeta)$	ζ
1	$\frac{1}{50}$	9.2 m	$\frac{1}{1}$	0.90 m
2	$\frac{1}{1}$	3.7 m	$\frac{1}{50}$	1.67 m

$$P\left(E_1 \cap E_2\right) = P\left(E_1\right) * P\left(E_2\right) \tag{C.2}$$

$$P\left(E_1 \cap E_2\right) = 1/50 = 1/50 * 1/1 = 1/25 * 1/2 etc. \tag{C.3}$$

III Partial correlation In reality there will be some sort of statistical relationship between the variables. This implies that the probability of a variable is changed once the other variable is known. So in the event of a very high wave, there is probably a high water level, but the exact value of the water level is not exactly certain. When the correlation ρ is known, an interval of probabilities can be found using the theory of Ditlevsen on boundaries of probabilities of correlated events Ditlevsen [1977]. To find an event with a fixed probability, a return period of 50 years, the upper limit of the interval calculated with Ditlevsen is used to avoid an underestimation of the variables. As for the full independence, more combinations of variables have the same probability. But to find maximal wave heights and water levels, the return period of the design event should be equal to the return period of one of the variables. So one combination for an event with

[1]Negative dependence is neglected, as this is both physically not true and this leads to an underestimation of the design criteria.

a combined probability of $\frac{1}{50}$ would be an $\frac{1}{50}$ H_s with 1/x ζ, where x is between 1 and 50 and is depending on the correlation.
The question remains what the exact value for this correlation is, but fortunately there is some data on this. The meteorological institute described combinations of events in [Meteorological Institute, 2015]. Besides in [Meteorological Institute, 2013] the event of Wilma is described. Using this data a correlation of 0.44 is found. In the previously described example this would lead to an 1/**12** water level.

$$P\left(E_1 \cap E_2\right) = f(P\left(E_1\right), P\left(E_2\right), \rho) \tag{C.4}$$

A safe correlation between H_s and ζ From [Meteorological Institute, 2015] a correlation of 0.44 can be calculated. However the dataset that is used to compute this correlation is relatively small. Therefore it is wise to introduce an upper limit of the correlation that seems realistic. Besides, the data of [Meteorological Institute, 2015] gives a theoretical certainty of 98% for an upper limit of 0.8365 of the correlation.
The value of 0.8365 is used in further computations. This is still a conservative approach as the real correlation will be around 0.44. Unfortunately, better data is lacking and it is expected that more data will convert to a correlation of the same order, but in order to be on the safe side 0.8365 is used.

C.2 Ditlevsen

Using the found correlation of 0.8365 an interval of probabilities can be computed. In 1977 Ditlevsen found a formula to find these probabilities. Ditlevsen stated that when the correlation ρ is known, an interval of probabilities for a combined event can be found based on the probabilities of the individual events. For more information on background of the method the reader is referenced to Ditlevsen [1977].
Ditlevsen formulas:

$$\Phi(-\beta_1^*)\Phi(-\beta_2) \leq P\left(E_1 \cap E_2\right) \leq \Phi\left(-\beta_1\right)\Phi\left(-\beta_2^*\right) + \Phi\left(-\beta_1^*\right)\Phi\left(-\beta_2\right) \tag{C.5}$$

$$\beta_i = -\Phi^{-1}(P(E_i)) \tag{C.6}$$

$$\beta_1^* = \frac{\beta_1 - \rho\beta_2}{\sqrt{1-\rho^2}} \tag{C.7}$$

$$\beta_2^* = \frac{\beta_2 - \rho\beta_1}{\sqrt{1-\rho^2}} \tag{C.8}$$

$$\rho_{X_1X_2} = \frac{Cov(X_1, X_2)}{\sigma_{X_1}\sigma_{X_2}} \tag{C.9}$$

For this project the upper boundary is the governing situation as this leads to the highest loading combinations. Using this for a fixed probability of $\frac{1}{50}$ a range of combinations between wave heights and water levels exists. The maximal individual return period of one of these variables is equal to the return period of the combined event. So for an $\frac{1}{50}$ event the maximal $P(H_s) = \frac{1}{50}$ and $H_s = 9.2$ m combined with an elevation of $P(\zeta) = 1/\mathbf{7.5}$ and $\zeta =$**1.50** m. Vice versa $P(\zeta) = \frac{1}{50}$ and $\zeta =$1.67 m combined with $P(H_s) = 1/\mathbf{7.5}$ and $H_s =$**6.52** m.

C.3 Schematic overview of the method

The description of the method of finding a combination of H_s and ζ for a fixed return period is rather extensive. To summarise a shorter schematic overview is presented below.
Targets:

- Determine realistic governing combination of two variables, for a fixed return period;
- Choose return period based on processed outcome.

Given:

- Return periods (= exceedance probabilities per year) for separate variables;
- Small dataset of combinations of variables.

Approach:

- Set theoretical boundaries for a fixed return period: full independence (no correlation or lower bound) and dependence (full correlation or upper bound) of variables;
- Describe realistic approach: use the actual correlation and Ditlevsen boundaries;
- Use dataset for a reference correlation;
- Search for better references for correlation, find realistic empirical upper boundary;
- Statistical analyses of dataset for a theoretical certainty of the upper boundary;
- Set realistic upper boundary;
- Use Ditlevsen to find combinations of variables;
- Process found combinations (amount of overtopping) to find governing combination(s);

Appendix D

SWAN model

This appendix gives background information of the SWAN model that is used to compute the wave transformation from deep to shallow waters. A SWAN model description is given in section D.1. In section D.2 the input files for the SWAN model are given. Section D.3 shows the results for different load combinations and wave directions.

D.1 Program description SWAN

This model description is based on information from the SWAN user manual [SWAN team, 2015]. SWAN is a numerical, third-generation wave model that can be used to compute wave transformation from deep water to nearshore values, based on a wave action balance. The model does not solve for individual waves, but transforms an offshore wave spectrum to a wave spectrum nearshore. These spectra are defined by a spectral shape, wave height, peak period and frequencies. Other input parameters that should be defined are the bottom level, initial water level, wind speed and wind direction.
The physical phenomena that are included in SWAN are generation of waves, quadruplet wave-wave interactions, depth-induced breaking, friction, triad wave-wave interactions, whitecapping and wave-setup. Based on an algorithm SWAN iterates the wave conditions for each point in the grid. SWAN can thus generate output in any desired point of the grid.
The relevant spatial scales vary for wind waves propagating from deep to shallow waters, which can require local refinement of the grid mesh size near the coast. This can be achieved by employing a nesting approach: first compute the waves on a coarse grid for a relatively large region and then on a finer grid for a smaller region.

D.2 SWAN input files

The SWAN input file and corresponding grid are given in figures D.1 and D.2 for the largest grid, in figures D.3 and D.4 for the first nested grid and in figures D.5 and D.6.

```
$*********************** HEADING ************************
PROJ 'combination' '302'
$
$ Malecon seawall in Havana, Cuba
$ Wave propagation towards shore (2 October 2015)
$ Coarsest grid
$******************** MODEL INPUT ***********************
SET 1.884 NAUT
MODE STAT TWOD
COORDINATES CART
$
CGRID REG 3.2385e+5 2.5418e+6 5 72000. 27000. 100 37 CIRCLE 36 0.03 1. 31
$ SECTOR 260. 45. 72 0.05 1. 31
$
INPGRID BOT REG 3.2385e+5 2.5423e+6 5 100 37 720. 729.73 EXC -1.0000000e+003
READINP BOT 1. 'z-table_101x38.bot' 3 0 FREE
$
WIND 25.0 347.5
$
BOUN SHAPE JON 3.3 PEAK DSPR DEGREE
BOUN SIDE N CCW CON PAR 7.80 12.0 347.5 30
BOUN SIDE W CCW CON PAR 7.80 12.0 347.5 30
BOUN SIDE E CCW CON PAR 7.80 12.0 347.5 30
$
$************ PHYSICS ************************
GEN3 KOMEN
QUAD
BREAKING
FRICTION
TRIADS
WCAP
SETUP
$************ OUTPUT REQUESTS ************************
$
BLOCK 'COMPGRID' NOHEAD 'mal302.mat' LAY 4 XP YP HS BOTLEV
NGRID 'MALECON' 3.5536e+5 2.5556e+6 5 12e+3 12e+3 100 100
NESTout 'MALECON' 'nes302.spc'
$
TEST 0 0
COMPUTE
STOP
```

Figure D.1: SWAN input file for the large and coarse grid

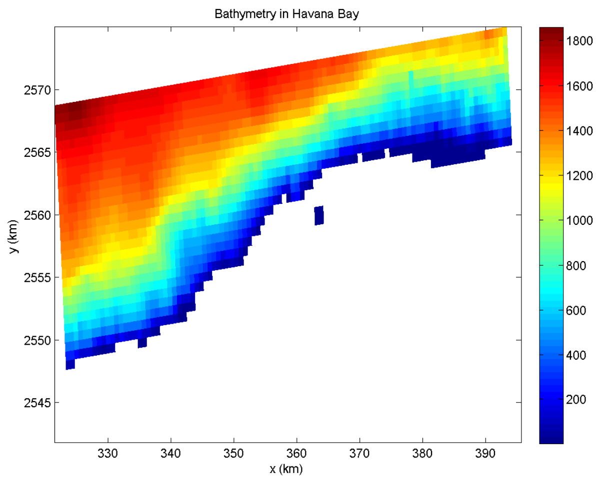

Figure D.2: Bathymetry for the large, coarse SWAN grid offshore

```
$*********************** HEADING ************************
PROJ 'nested' '302'
$
$ Malecon seawall in Havana, Cuba
$ Wave propagation towards shore (5 October 2015)
$ medium grid for the combination 'run id.'
$******************** MODEL INPUT ************************
SET 1.884 NAUT
MODE STAT TWOD
COORDINATES CART
$
CGRID REG 3.5536e+5 2.5556e+6 5 12e+3 12e+3 100 100 CIRCLE 36 0.03 1. 31
$
INPGRID BOT REG 3.5536e+5 2.5556e+6 5 100 100 120. 120. EXC -1.0000000e+003
READINP BOT 1. 'nest-table_101x101.bot' 3 0 FREE
$
WIND 25.0 347.5
$
BOUndnest1 NEst 'nes302.spc' CLOSed
$
$************ PHYSICS ************************
GEN3 KOMEN
QUAD
BREAKING
FRICTION
TRIADS
WCAP
SETUP
$************ OUTPUT REQUESTS ************************
$
BLOCK 'COMPGRID' NOHEAD 'nes302.mat' LAY 4 XP YP HS BOTLEV
NGRID 'NESNES' 3.5766e+5 2.5590e+6 5 5.6e+3 3.2e+3 300 171
NESTout 'NESNES' 'nesnes302.spc'
TEST 0 0
COMPUTE
STOP
```

Figure D.3: SWAN input file for the first nest

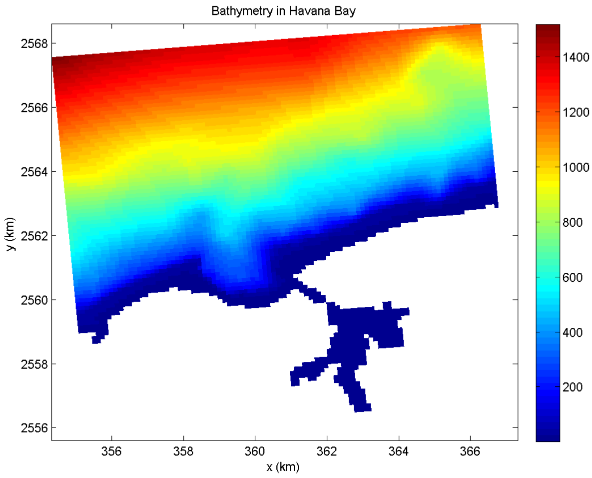

Figure D.4: Bathymetry for the first nested SWAN grid at Havana Bay

```
$*********************** HEADING ************************
PROJ 'nested' '302'
$
$ Malecon seawall in Havana, Cuba
$ Wave propagation towards shore (5 October 2015)
$ Smaller, finer grid using nested input
$******************** MODEL INPUT ***********************
SET 1.884 NAUT
MODE STAT TWOD
COORDINATES CART
$
CGRID REG 3.5766e+5 2.5590e+6 5 5.6e+3 3.2e+3 300 171 CIRCLE 36 0.03 1. 31
$ SECTOR 260. 45. 72 0.05 1. 31
$
INPGRID BOT REG 3.5766e+5 2.5590e+6 5 300 171 18.67 18.71 EXC -1.0000000e+003
READINP BOT 1. 'swash-table_301x172.bot' 3 0 FREE
$
WIND 25.0 347.5
$
BOUndnest1 NEst 'nesnes302.spc' CLOSed
$
$************ PHYSICS ************************
GEN3 KOMEN
QUAD
BREAKING
FRICTION
TRIADS
WCAP
SETUP
$************ OUTPUT REQUESTS ************************
OUTPut OPTIons '%' TABle 16
$
CURve 'BREAKW' 3.589e+5 2.5601e+6 5 3.5935e+5 2.5599e+6 8 3.6039e+5 2.55992e+6 2 3.6052e+5 2.56e+6 4 3.608e+5 2.5603e+6
TABLE 'BREAKW' NOHEAD 'breakwater302.tab' XP YP HS BOTLEV SETUP RTP DIR PDIR
BLOCK 'COMPGRID' NOHEAD 'nesnes302.mat' LAY 4 XP YP HS BOTLEV SETUP RTP DIR PDIR
$
TEST 0 0
COMPUTE
STOP
```

Figure D.5: SWAN input file for the second nest. This is the smallest and finest grid.

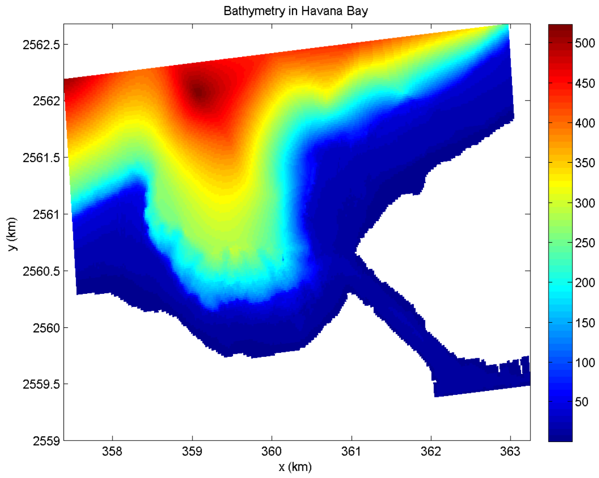

Figure D.6: Bathymetry for the finest, double nested SWAN grid at the study area

D.3 SWAN results

This section shows the results of the different runs that were performed. First the difference between open and closed east and west boundaries is shown in section D.3.1. The results for runs with different load combinations are shown in section D.3.2. For the governing load combination, different incoming wave direction were assessed. The results of this assessment are shown in section D.3.3

D.3.1 Difference between open and closed east and west wave boundaries

The resulting wave climate for the a run with only an incoming wave boundary on the north are displayed in figure D.7. The mean wave direction is north north west. From these results it is clearly visible that the closed boundary on the west creates a shadow zone. This shadow zone is minimised as much as possible. Due to lack of bathymetric data further westward, it is impossible to move the west boundary any further. The results for a run with incoming wave boundaries on the north, east and west can be seen in figure D.8. This prevents a shadow area from developing. This may however introduce an error, as the wave spectrum is imposed on the entire boundary, including the shallow area nearshore.

D.3.2 Results for different load combinations

Table 4.1 shows the results of the wave transformation computations for all load combinations. For completeness, the load combinations and corresponding nearshore wave height are listed again in table D.1. Figures D.9 upto D.13 show the resulting wave heights for combinations 1, 2, 3, 4 and 5 respectively. These results are displayed at the Malecón seawall, as well as at a location just away from the wall, in order to obtain values of the wave

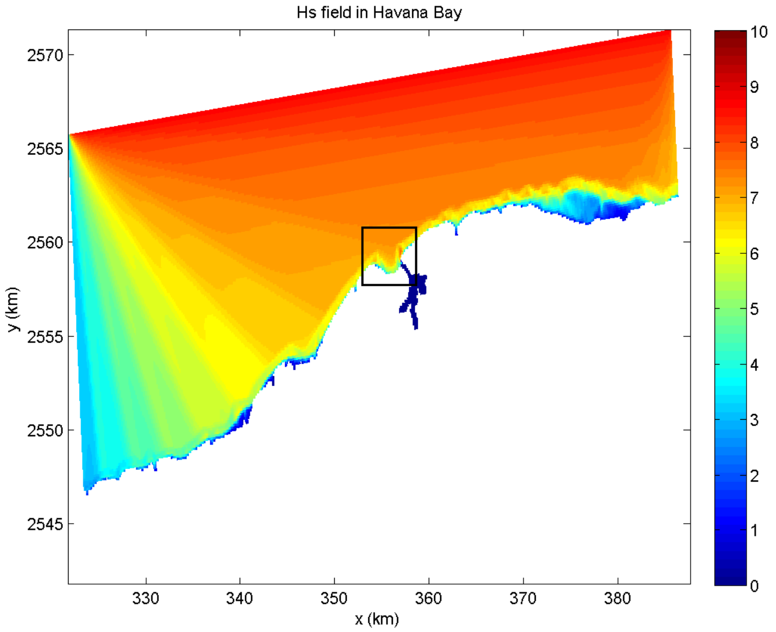

Figure D.7: Resulting wave climate in Havana Bay with closed east and west boundary

height for the design of the breakwater. The largest significant wave height is displayed in the title of each graph. All circled cells have wave height that is close to the maximum wave height.

Combination	Offshore ζ [m]	Offshore H_s [m]	Nearshore H_s [m]
0	1.95	9.14	4.35
1	1.95	6.52	4.08
2	1.88	7.80	4.36
3	1.50	9.14	4.35
4	1.70	8.84	4.37
5	1.80	8.44	4.32

Table D.1: Wave transformation for different combinations of H_s and ζ

D.3.3 Results for different incoming wave angles

In section 4.2 the overtopping is determined for all wave combinations. From those computations, it follows that combination 2 is the governing load combination. Therefore, this load combination is assessed for different wave directions. Figures D.14 upto D.20 show the wave field of Havana Bay for incoming wave angles between west and north east. As can be seen from the results, waves from the west, west north west, east north east and north east do not lead to the highest waves at the Malecón Sea wall. Waves from the north west, north north west and north result in the largest waves, so these directions are

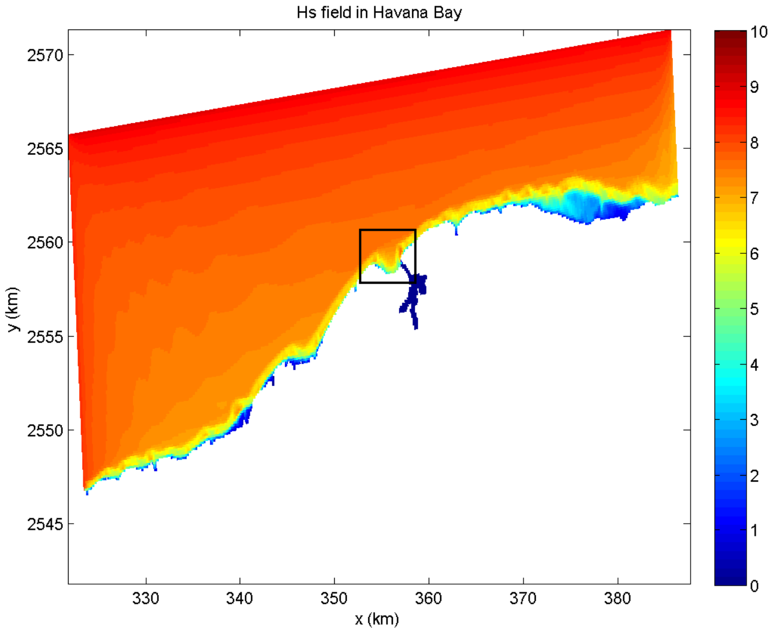

Figure D.8: Resulting wave climate in Havana Bay with opened east and west boundary

used in the assessment.

Figures D.21, D.22 and D.23 show the resulting wave heights at the Malecón seawall for incoming wave angles north west, north north west and north respectively. It can be seen that the absolute maximum wave height is obtained for waves coming from the north. However, when selecting a representative wave height per section, it was found that the governing wave direction depends on the location. Therefore all three wind direction are considered.

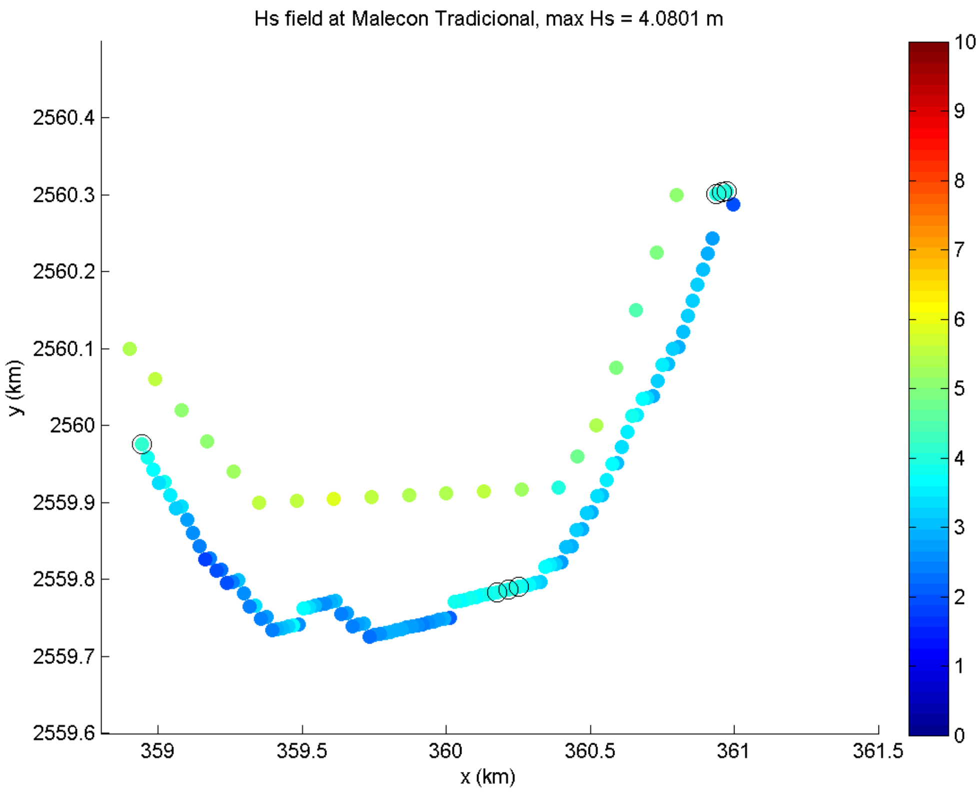

Figure D.9: Resulting wave heights at the Malecón seawall for combination 1

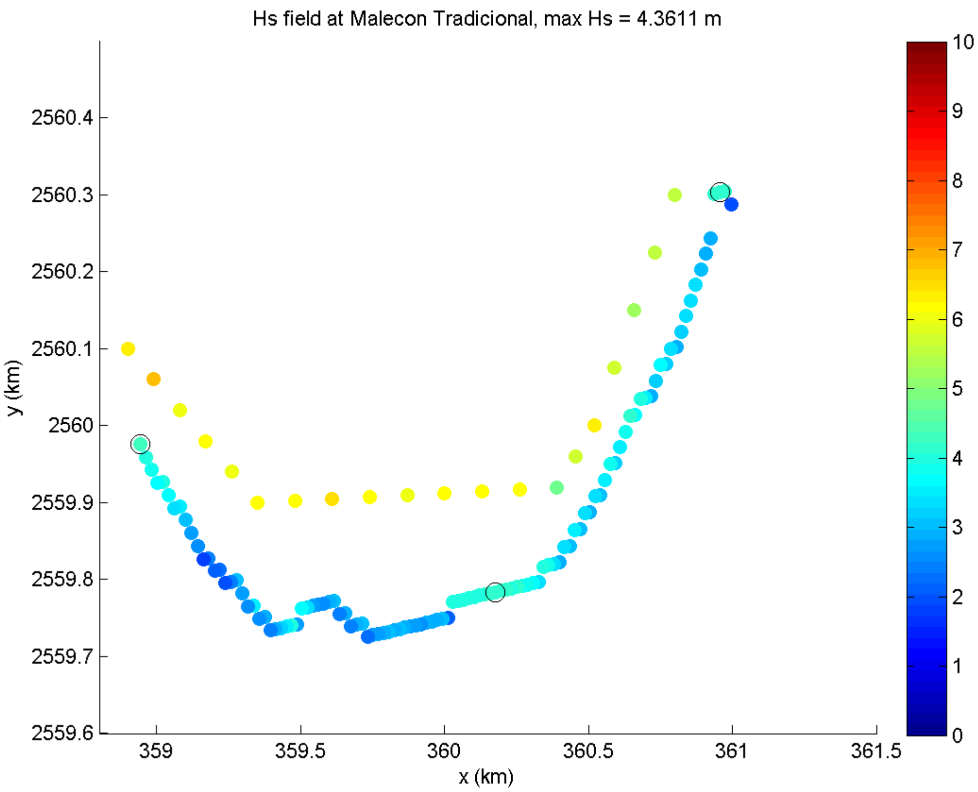

Figure D.10: Resulting wave heights at the Malecón seawall for combination 2

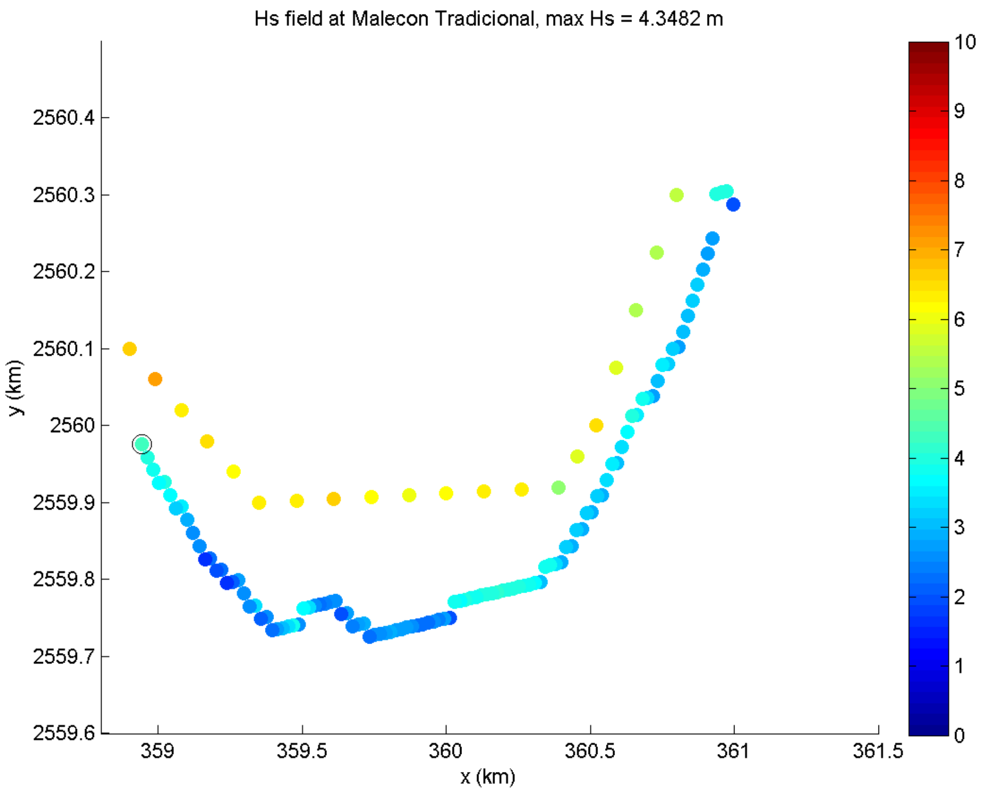

Figure D.11: Resulting wave heights at the Malecón seawall for combination 3

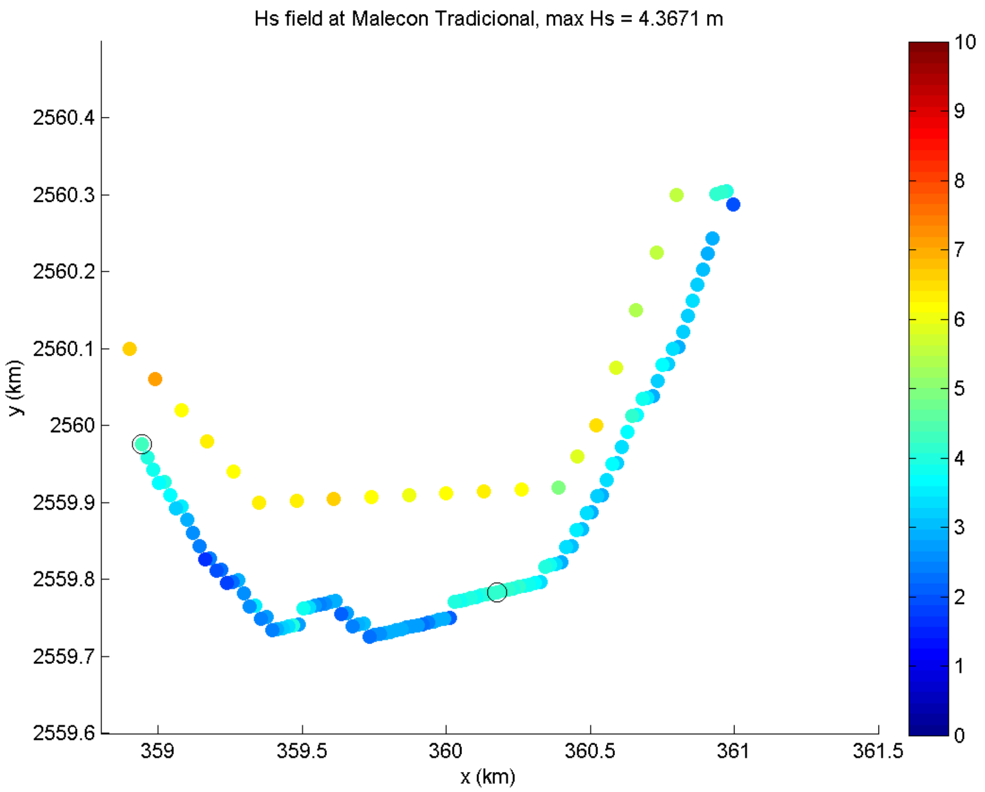

Figure D.12: Resulting wave heights at the Malecón seawall for combination 4

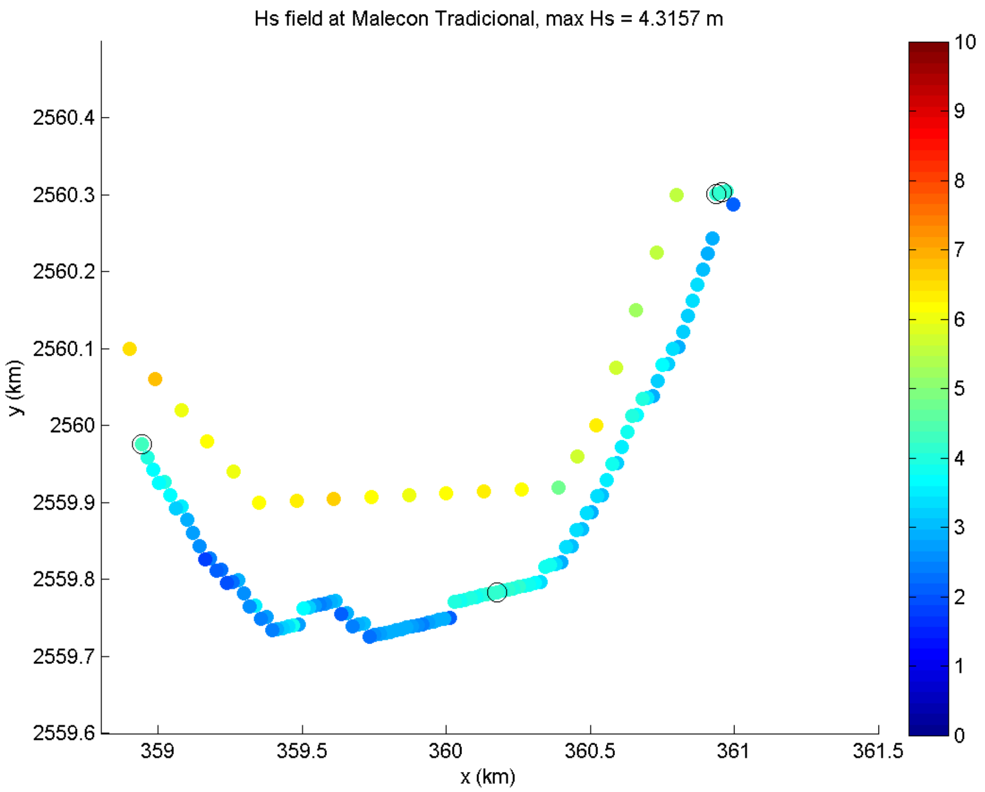

Figure D.13: Resulting wave heights at the Malecón seawall for combination 5

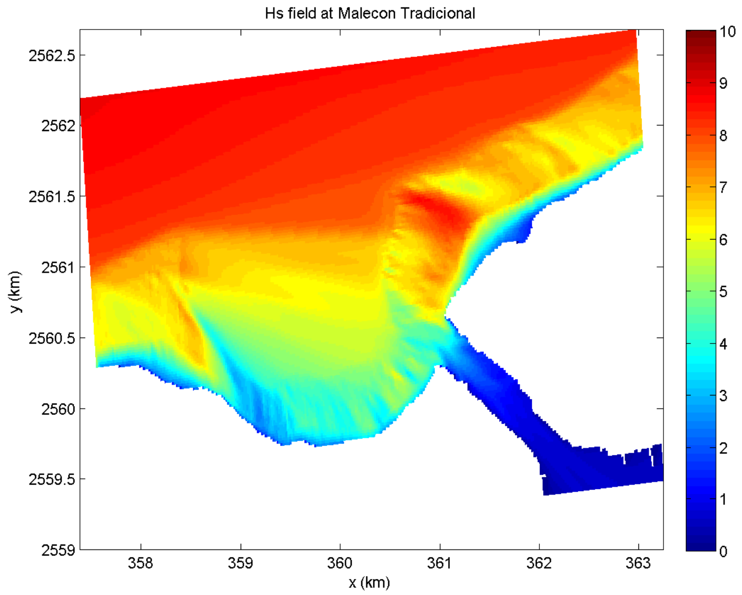

Figure D.14: Resulting wave field for incoming waves from the west

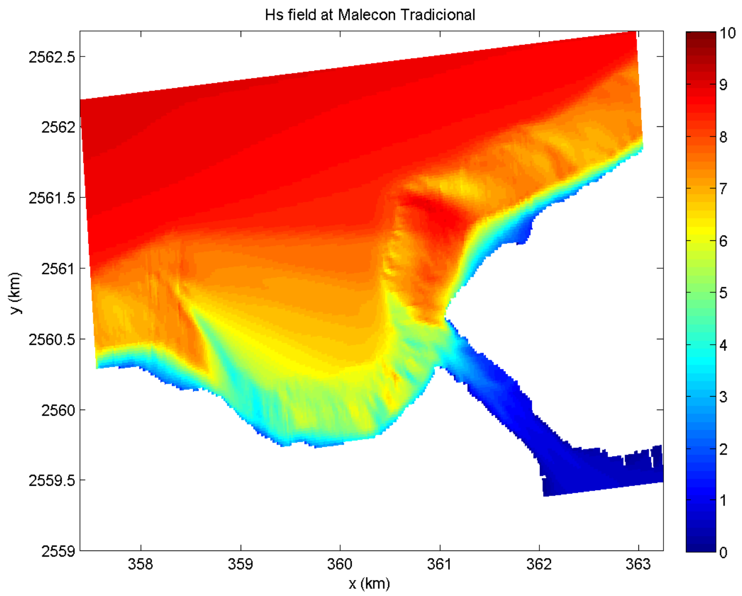

Figure D.15: Resulting wave field for incoming waves from the west north west

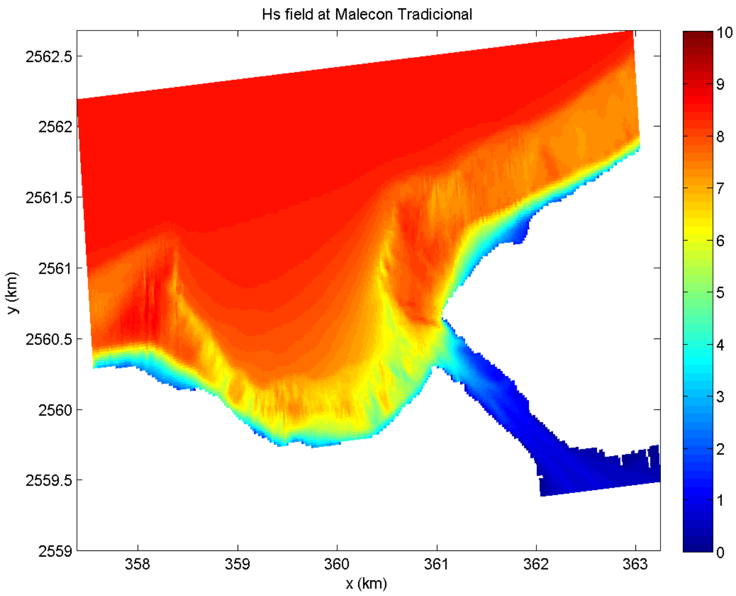

Figure D.16: Resulting wave field for incoming waves from the north west

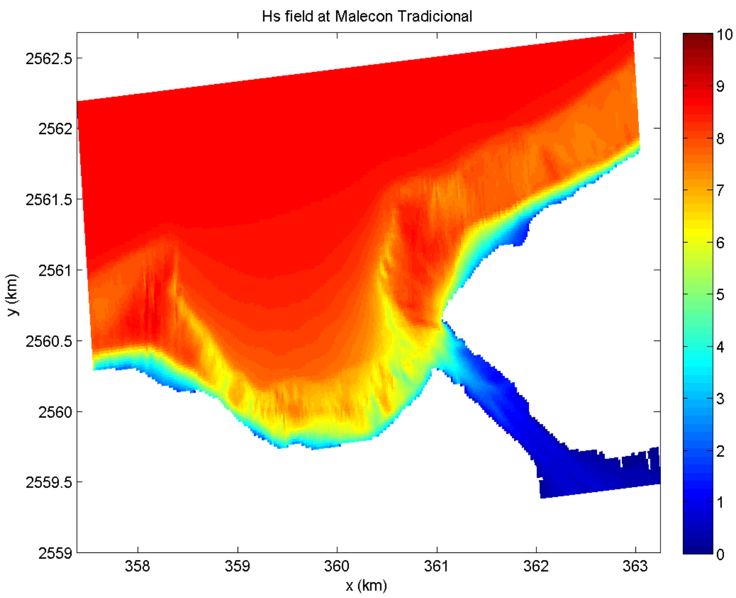

Figure D.17: Resulting wave field for incoming waves from the north north west

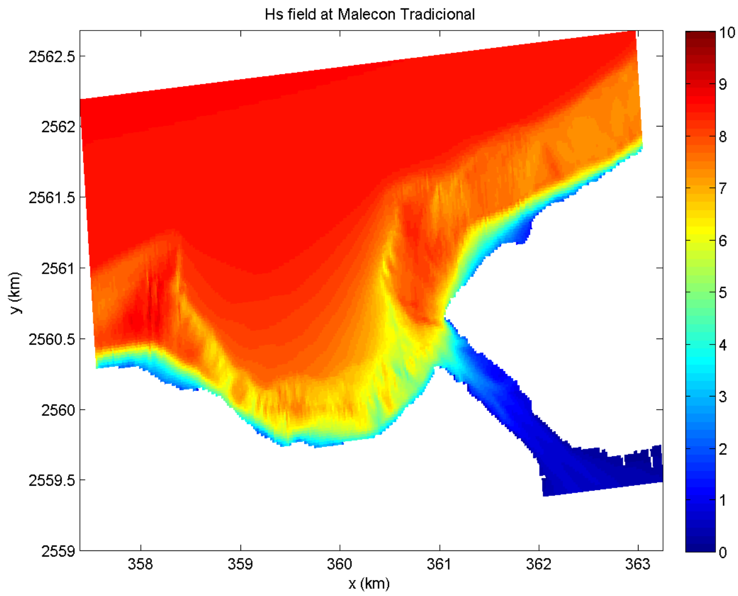

Figure D.18: Resulting wave field for incoming waves from the north

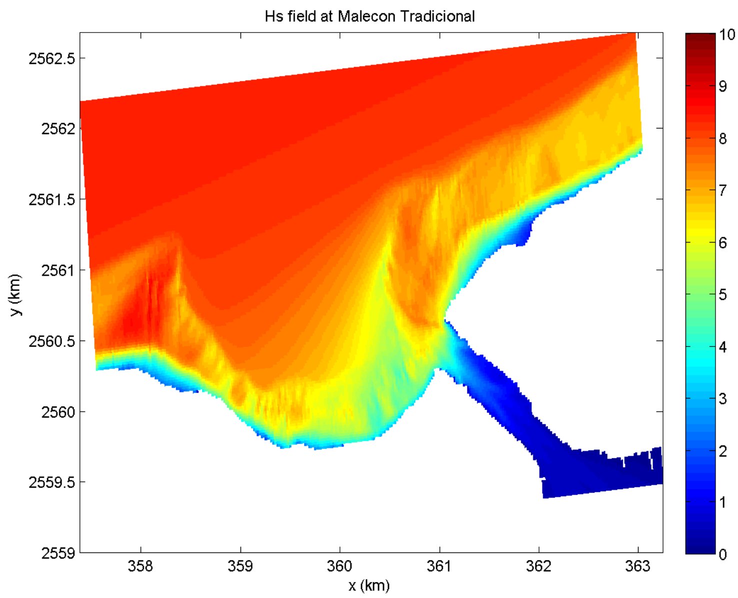

Figure D.19: Resulting wave field for incoming waves from the north north east

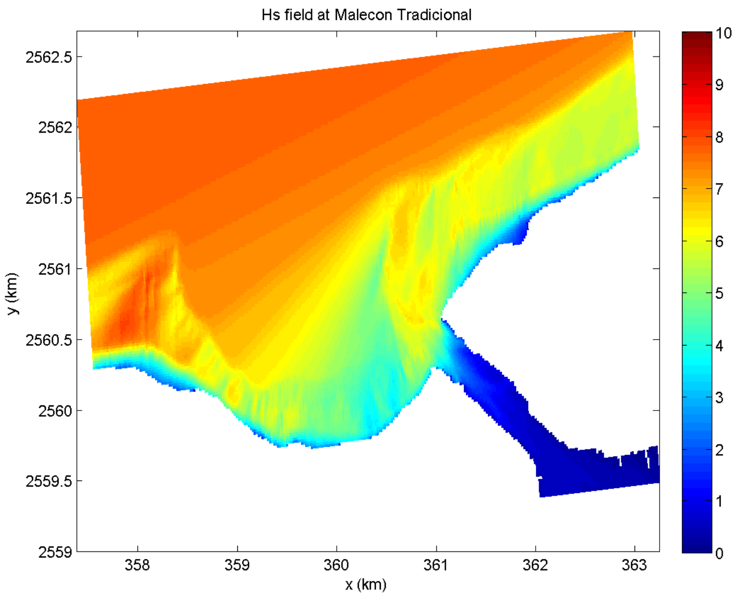

Figure D.20: Resulting wave field for incoming waves from the north east

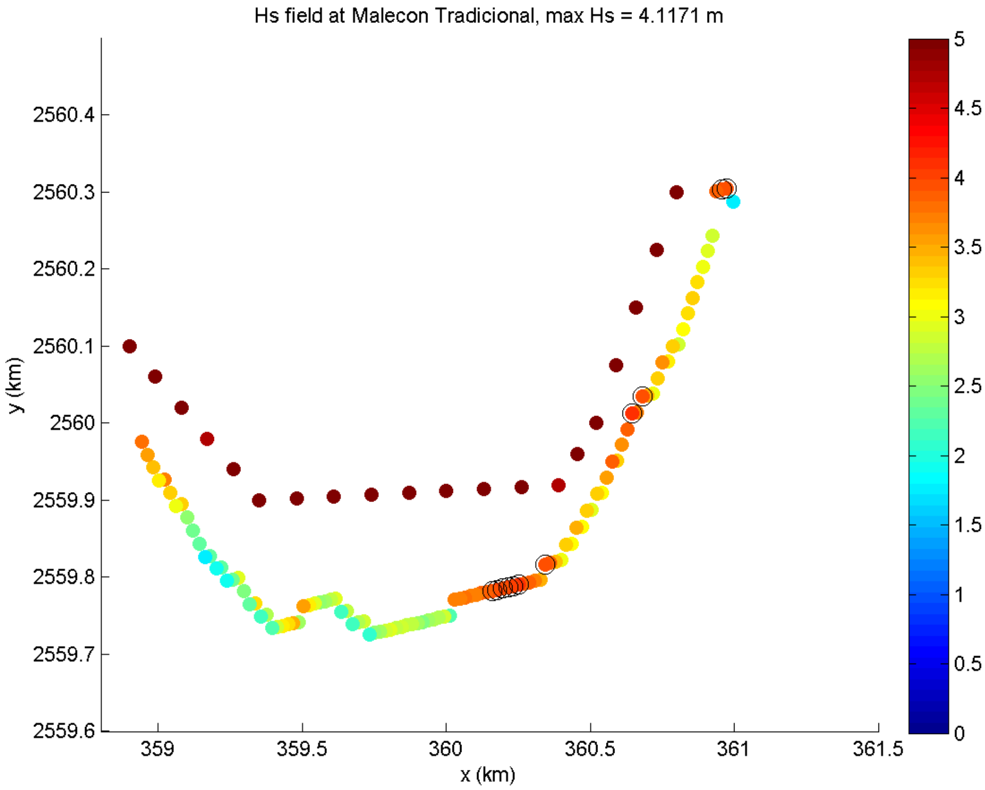

Figure D.21: Resulting wave heights at the Malecón seawall for incoming waves from the north west

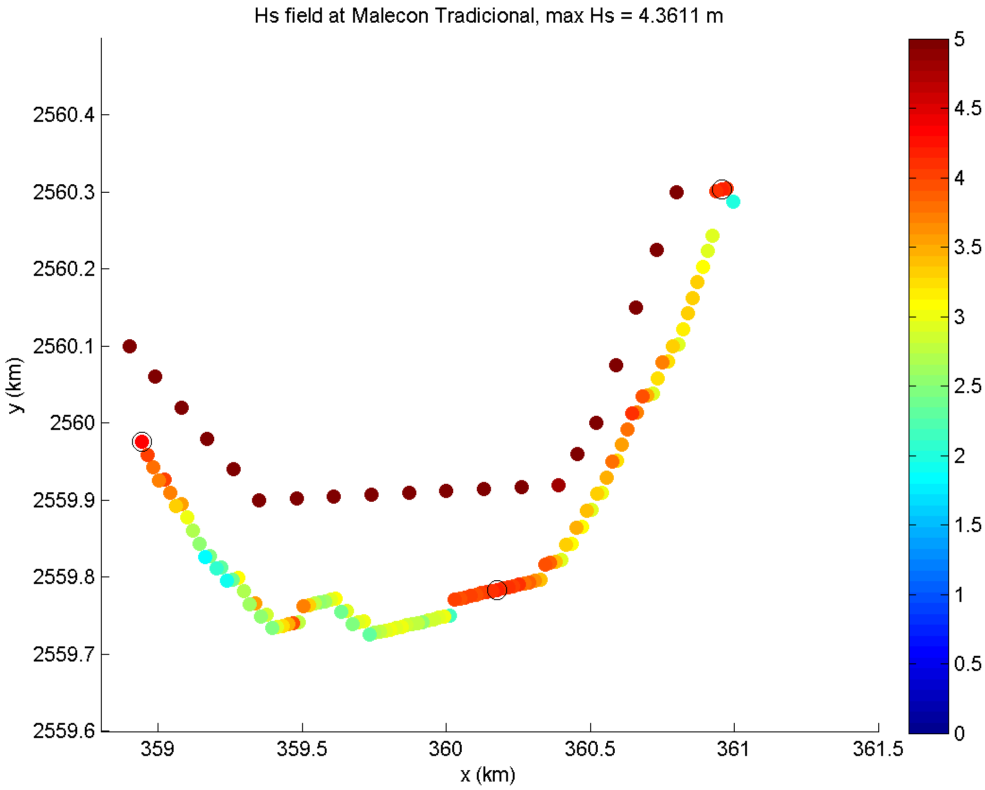

Figure D.22: Resulting wave heights at the Malecón seawall for incoming waves from the north north west

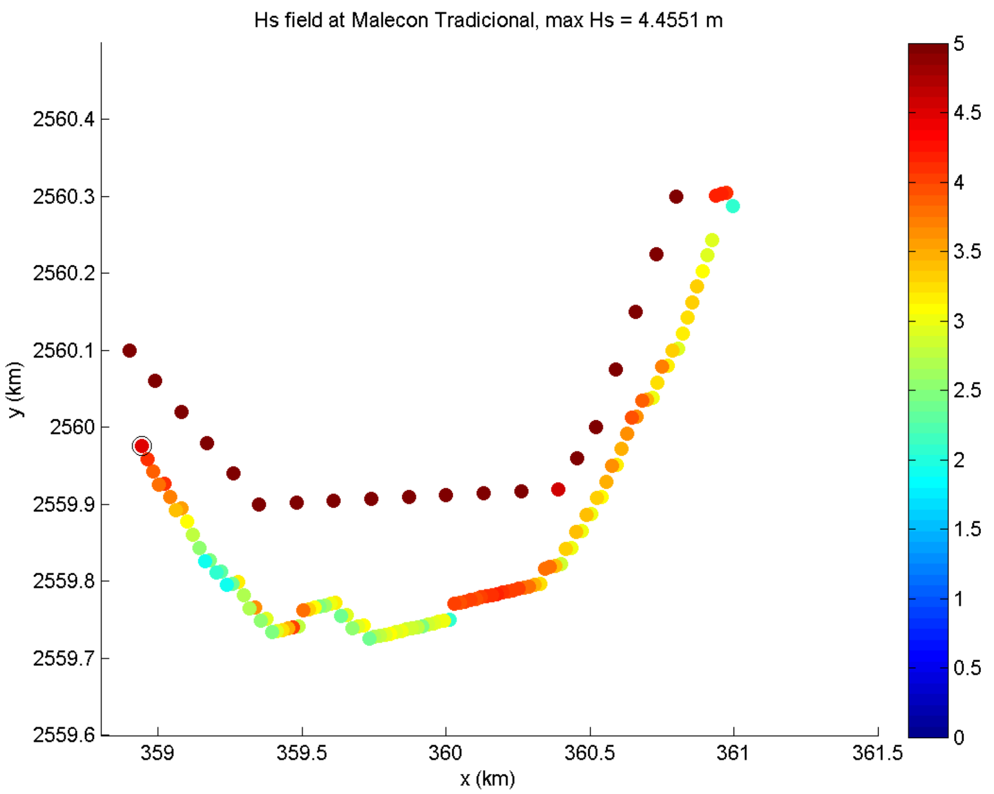

Figure D.23: Resulting wave heights at the Malecón seawall for incoming waves from the north

Appendix E

Forcing on the Malecón seawall

In this appendix the background information of the coupling methods is given, together with the results of applying these methods to the study area, the comparison with the physical models and the determined pressure distributions.

E.1 Goda

In the Coastal Engineering Manual the Goda formula, modified by Takahashi, Tanimoto and Shimosako for an impermeable vertical wall is given [U.S. Army Corps Engineers, 2003]. It is modified to include impulsive forces from head-on breaking waves. In figure E.1 the schematisation of the pressures is shown.

The pressures at significant points on the wall are given as:

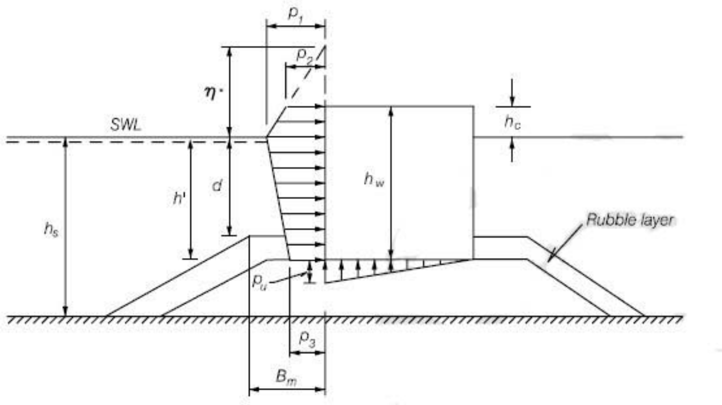

Figure E.1: Pressure distribution Goda [U.S. Army Corps Engineers, 2003]

$$\eta^* = 0.75(1 + \cos\beta)\lambda_1 H_{design} \tag{E.1}$$

$$p_1 = 0.5(1 + \cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* \cos^2\beta)\rho_w g H_{design} \tag{E.2}$$

$$p_2 = \begin{cases} \frac{1-h_c}{\eta^*} & for \quad \eta^* > h_c \\ 0 & for \quad \eta^* \leq h_c \end{cases} \tag{E.3}$$

$$p_3 = \alpha_3 \tag{E.4}$$

Where: $\alpha_* =$ largest of α_2 and α_I

$\alpha_1 = 0.6 + 0.5(\frac{4\pi h_s/L}{\sinh(4\pi h_s/L)})^2$

$\alpha_2 =$ the smallest of $\frac{h_b-d}{3h_b}(\frac{H_{design}}{d})^2$ and $\frac{2d}{H_{design}}$

$\alpha_3 = 1 - \frac{h_w-h_c}{h_s}(1 - \frac{1}{\cosh(2\pi h_s/L)})$

$\alpha_I = \alpha_{I0} * \alpha_{I1}$

$$\alpha_{I0} = \begin{cases} H_{design}/d & for \quad H_{design}/d \leq 2 \\ 2.0 & for \quad H_{design}/d > 2 \end{cases}$$

$$\alpha_{I1} = \begin{cases} \frac{\cosh \delta_2}{\cosh \delta_2} & for \quad \delta_2 \leq 0 \\ \frac{1}{\cosh \delta_1 * (\cosh \delta_2)^2} & for \quad \delta_2 > 0 \end{cases}$$

$$\delta_1 = \begin{cases} 20 * \delta_{11} & for \quad \delta_{11} \leq 0 \\ 15 * \delta_{11} & for \quad \delta_{11} > 0 \end{cases}$$

$\delta_{11} = 0.93(\frac{B_m}{L} - 0.12) + 0.36(\frac{h_s-d}{h_s} - 0.6)$

$$\delta_2 = \begin{cases} 4.9 * \delta_{22} & for \quad \delta_{22} \leq 0 \\ 3 * \delta_{22} & for \quad \delta_{22} > 0 \end{cases}$$

$\delta_{22} = -0.36(\frac{B_m}{L} - 0.12) + 0.93(\frac{h_s-d}{h_s} - 0.6)$

β: Angle of incidence of waves

H_{design}: Design wave height defined as the highest wave in the design sea state at a location just in front of the breakwater. Goda recommends for practical design a value of 1.8 H_s, which corresponds to $H_{1/250}$.

L: Wavelength at water depth h_b corresponding to the significant wave period.

h_b: Water depth at a distance of $5H_s$ seaward of the wall.

λ_1, λ_2: Modification factors. For vertical wall structures, $\lambda_1 = \lambda_2 = 1$.

E.2 Physical model test

In figure E.2 and figure E.3 the dimensions of the prototype and the placing of the transducers can be seen.
To test if this method is valid for this situation, the dimensionless pressure is calculated

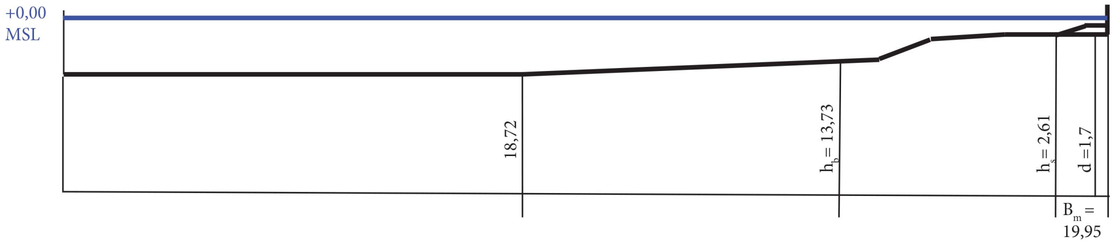

Figure E.2: Dimensions of the slope of the prototype used for the physical model test

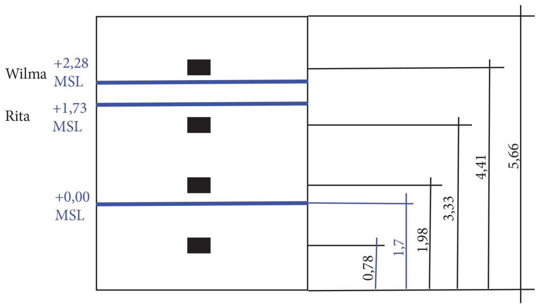

Figure E.3: Water levels and transducer locations of the physical model test

for the variables of the physical model test [Buccino et al., 2013]. Sixteen tests were done, of which four are evaluated. These four tests are selected on basis of relevance and variety. The test is relevant when set-up is found at the structure, which implies breaking waves, and different peak periods, wave heights and storm surge levels are chosen for variety. Therefore tests 2, 4, 8 and 11 are selected.

The validity of this method for the used test results is tested for the parameter H_s/h_s in front of the structure. For the given range of tested parameters the range of this parameter is $0.32 - 0.90$. In table E.1 the variables of the tests and the parameter H_s/h_s can be seen. Since the values are within the range, it is assumed that the formula is valid.

In table E.2 the pressures that are calculated with Goda are compared with the results from the physical model tests. Goda has been calculated with the $H_{s_{at\ the\ wall}}$ that follows from the physical model test. This to keep the results comparible.

	Test 2	Test 4	Test 8	Test 11
Storm surge [m]	2.28	2.28	2.28	1.73
Set-up [m]	0.09	0.27	0.27	0.35
H_s [m]	2.99	3.55	3.26	2.32
h_s [m]	4.89	5.16	5.16	4.69
H_s/h_s [-]	0.61	0.69	0.63	0.49

Table E.1: Variables physical model test

	Wilma			Rita
Test	2	4	8	11
Storm surge [m]	2.28	2.28	2.28	1.73
Set up [m]	0.09	0.27	0.27	0.35
$H_{s_{at\ the\ wall}}$ [m]	2.99	3.55	3.26	2.32
$H_d = 1.8 * H_s$ [m]	5.8	4.4	3.3	4.7
T_p [s]	12.16	12.16	10.46	12.87
$\frac{p_{transducer0}}{\gamma * H_{s_{toe}}}$ $(calculated)$ [-]	9.2	10.3	13.2	7.2
$\frac{p_{transducer0}}{\gamma * H_{s_{toe}}}$ $(test)$ [-]	34.5	11.2	36.9	34.0

Table E.2: Used parameters and results from the modified Goda formula and test results

E.3 Determined pressure distributions

The impact pressure distribution around SWL according to physical model tests as described in [Cuomo et al., 2010] is given in figure E.4. The pressures on the current seawall for the ULS condition are given in figure E.5. The pressures on the elevated seawall for a return period of 50 years are given in figure E.6. The pressures on the current seawall for the ULS condition are given in figure E.5.

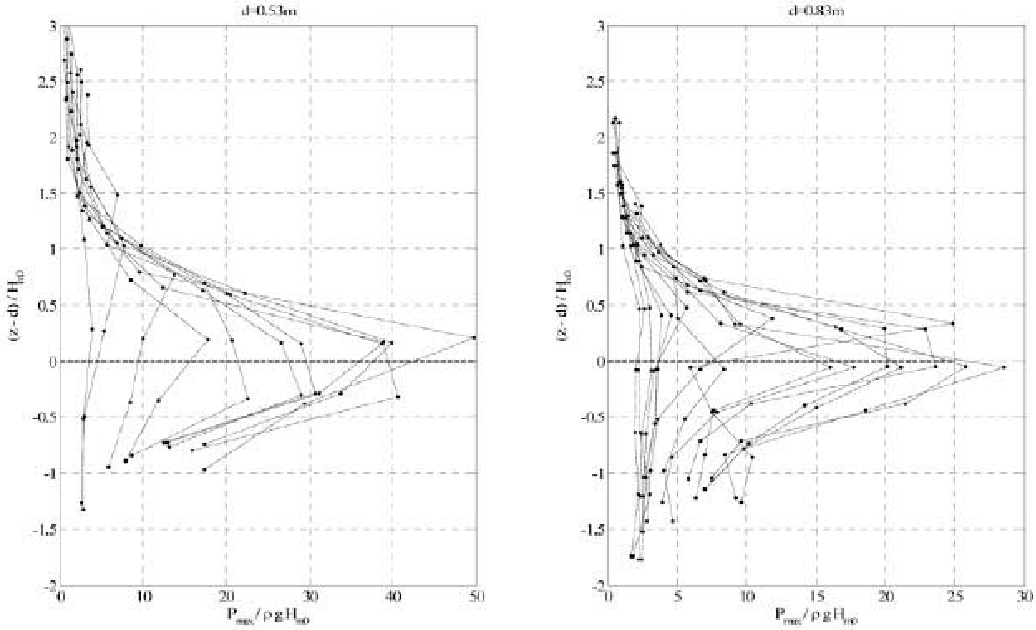

Figure E.4: Impact pressure distribution around SWL for a physical model test performed by [Cuomo et al., 2010]

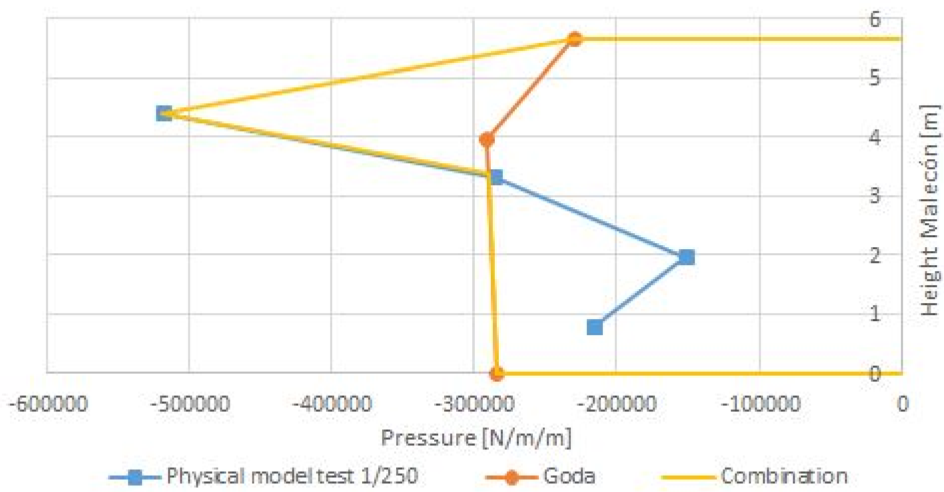

Figure E.5: Pressures on the current seawall for the ULS condition

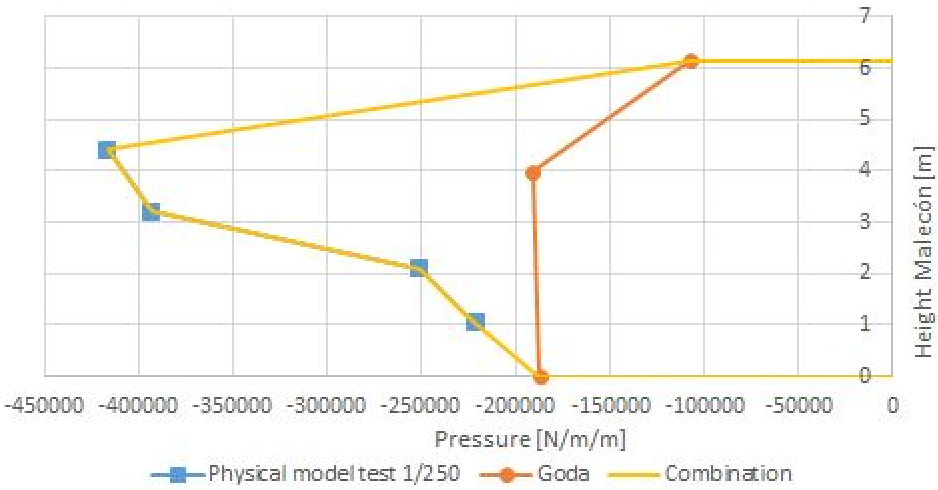

Figure E.6: Pressures on the elevated seawall for a return period of 50 years

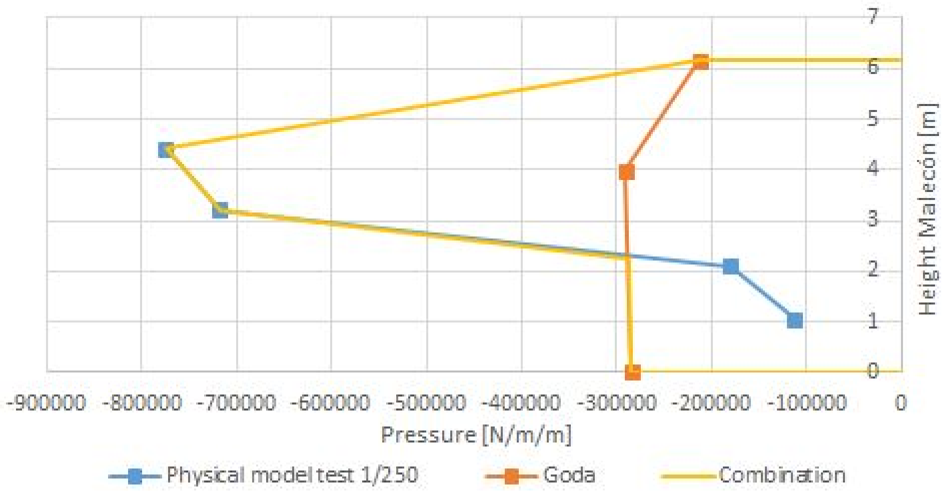

Figure E.7: Pressures on the elevated seawall for the ULS condition

Appendix F

Current cross sections

In this appendix the current cross sections is presented for each section. The following notes apply to all the cross-section:

- All dimensions are in mm.
- Levels are in meters relative to Mean Sea Level.
- Storm Still Water Level, for a governing $\frac{1}{50}$ year storm, is indicated at MSL +1.88 m.

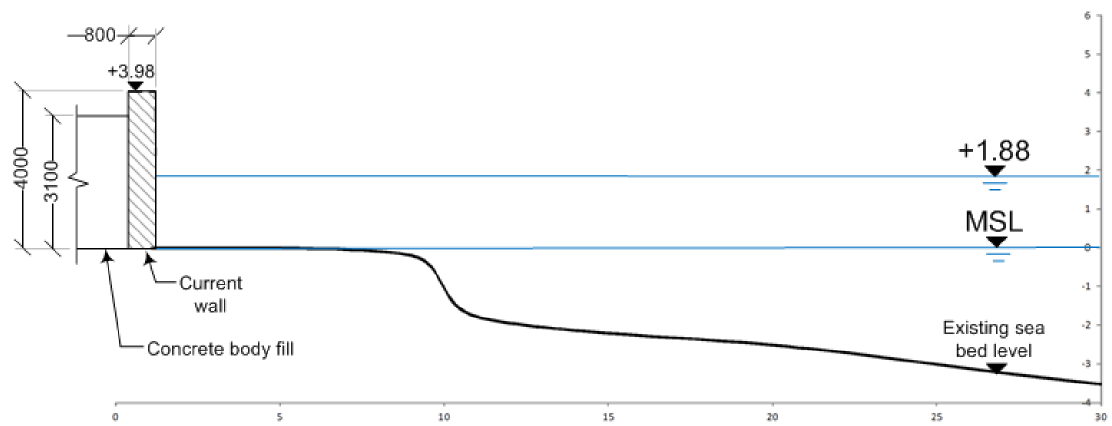

Figure F.1: Current cross sections for section 1

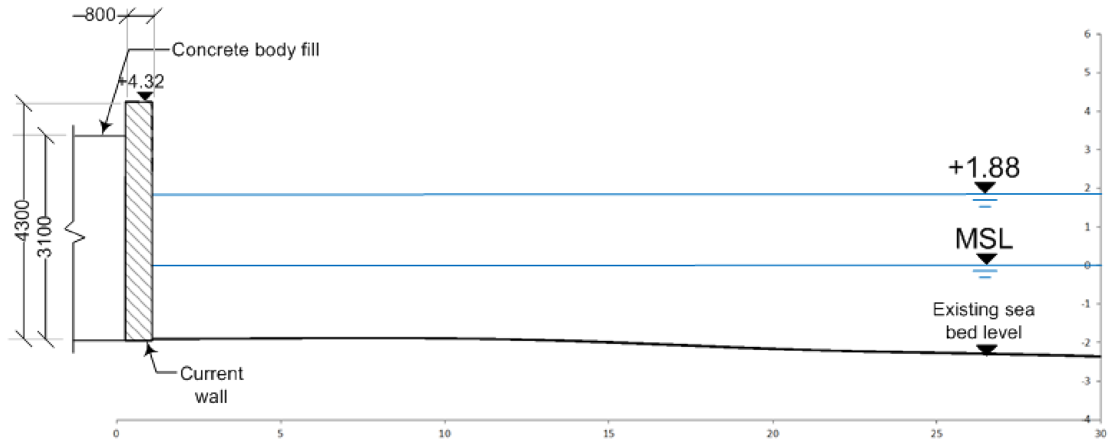

Figure F.2: Current cross sections for section 2

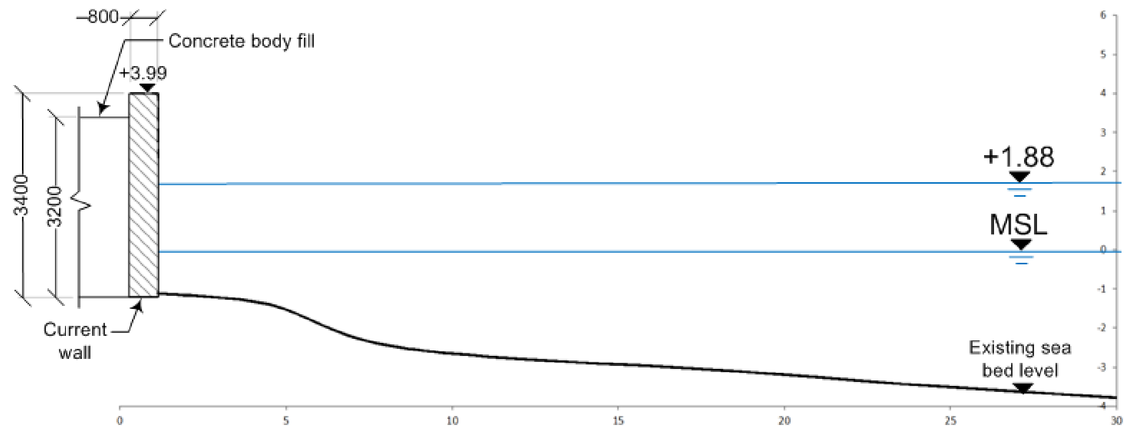

Figure F.3: Current cross sections for section 3

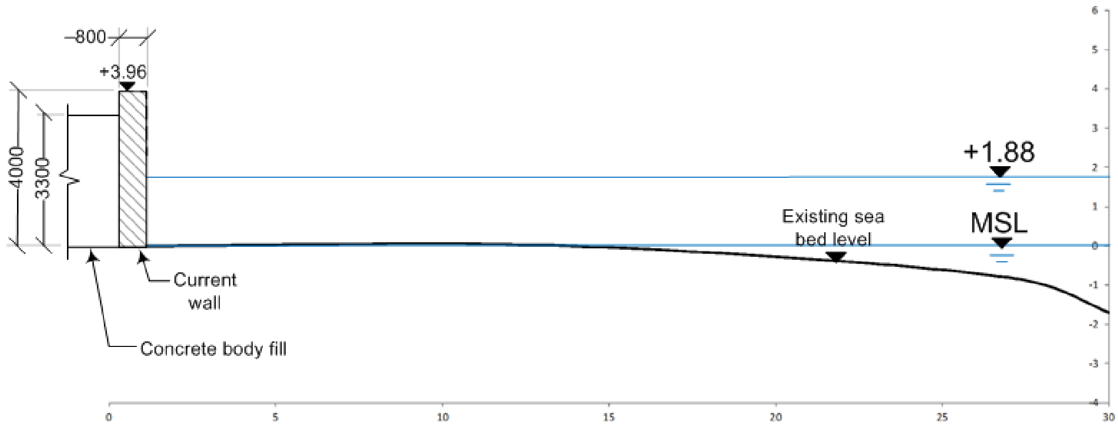

Figure F.4: Current cross sections for section 4

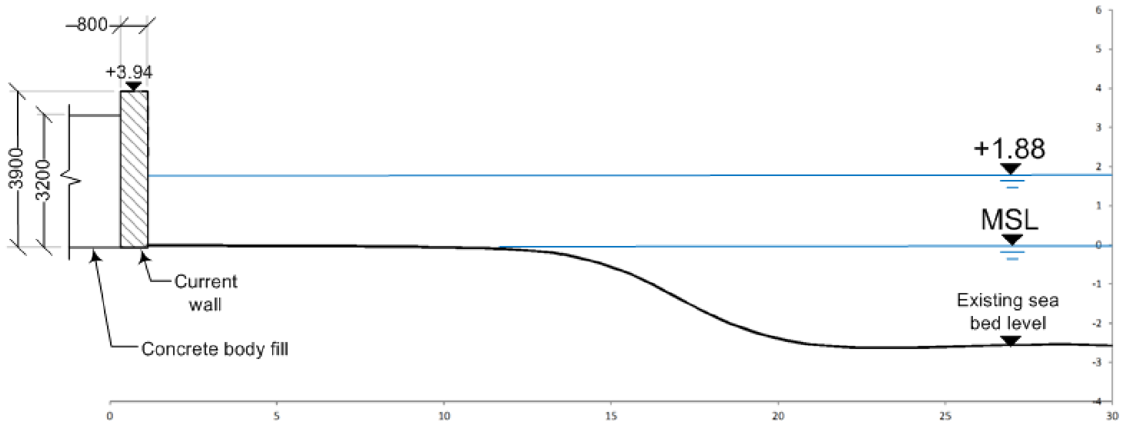

Figure F.5: Current cross sections for section 5

Appendix G

Construction of the Malecón

In order to be able to make a good model which represents the wall as accurately as possible, it is necessary to know how the seawall was constructed. Therefore this section will be dedicated to the construction of the wall. The information in this section is based on the power point presentation [de las Cuevas Toraya, 2013] and the accompanying story of prof. Córdova.

The construction of the Malecón seawall was initiated by the American government in 1900. Initially the Malecón was intended as a nice boulevard decorated with trees, see figure G.1, but due to the observed winds and rough sea conditions it was decided to use the wall as a flood defence as well.

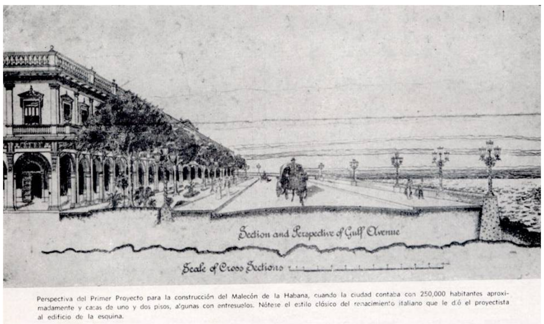

Figure G.1: General idea for the Malecón

Construction started at Castillo de la Punta in 1900. In the first two years the first 370 m of the wall were built up to Crespo. The construction of this part coincides with section 5 discussed in paragraph 5.1. As can be seen in figure G.3, first a wall was built consisting of concrete. For the concrete a wooden cast was made, so the concrete could be poured into place, this can also be seen in figure G.3 in the upper right corner.

Figure G.2: Coast line before construction

Figure G.3: First construction phase

The part which was constructed last is of great interest. This part coincides with section 2 of the wall in the first paragraph. This part was built between 1903 and 1909 and it is interesting because the wall was constructed directly in the water. Prior to the construction of the wall a small harbour was present at this location. This harbour was closed and the Batería de la Reina was demolished to have material to fill the harbour. Also the natural berm in front of the wall was removed in order to raise the ground level in the harbour. Therefore this part of the wall has no natural berm. The construction of the wall in water can be seen in figures G.4 and G.5.

Figure G.4: Constrution of the wall in water seen from above

Figure G.5: Construction of the wall in water seen from the side

After a section of wall was constructed the site behind the wall was filled with rocks and concrete to raise the land level. Care was taken to make sure this was done correctly, because on top of this raised land a new road was constructed. A rough sketch of the situation can be seen in figure G.6. After the road was constructed properly the space between the road and the wall was also levelled with the road. This space is used to make the sidewalk next to the Malecón seawall. This was not done with concrete, but with stones, and on top large plates were placed to create a sidewalk. Due to the method of construction and the scour at the toe of the wall, the sidewalk is cracked in a number of places nowadays.

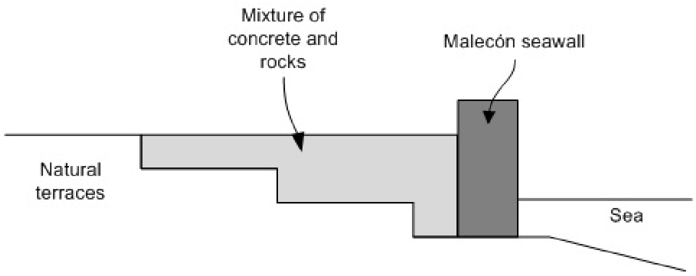

Figure G.6: Schematisation of the existing situation

Appendix H

ANSYS model

H.1 Program description

ANSYS is a professional program that allows an engineer to use the finite element method when evaluating a structure in search of a field quantity, such as the displacements. As formulated in Finite Element Modeling for Stress Analysis [Cook, 1995], a finite element method divides a structure into several elements that consist of a number of nodes and connections between the nodes. For each element the displacement vector is interpolated from values at the nodes. By reconnecting the elements this is interpolated for the entire structure. The values of the displacements that minimise a function (for example total energy) are the values used and displayed by the program. Because of the large amount of elements (and the corresponding amount of equations) used, a computer program like ANSYS is needed.

There are several types of elements. The program that is used, ANSYS Workbench, uses some of the best algorithms developed by ANSYS for the meshing according to Pierre Thieffry[Thieffry, 2010]. Most of the meshing is therefore automated, but mesh size and element size can be altered manually. Additionally a choice can be made between a quadrilateral and triangular element. Every node of such an element has six degrees of freedom, three displacements (x, y and z) and three rotations (θ_x, θy and θz) as can be seen in figure H.1. Each of these degrees of freedom can be constrained to determine the supports and prevent rigid body displacements. Other boundary conditions that can be applied are forced displacements and loads.

With the help of the element stiffness matrix and the given loads the displacements at the nodes can be computed. The displacements at the nodal locations are therefore exact. The material behaviour, like stresses and strains, is computed at the integration points (Gauss points). By using these displacement vectors and several matrix calculations, in which the material properties are implied, the stresses and strains are calculated at the Gauss points. These Gauss points do not coincide with the element surfaces, but are located inside the element volume (see figure H.2). Therefore the stresses and strains cannot be

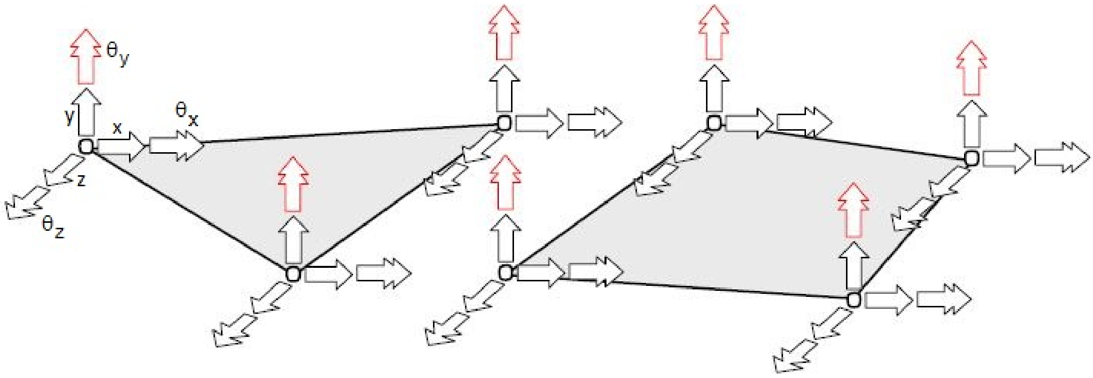

Figure H.1: Degrees of freedom of element nodes[Hoogenboom, 2012]

computed exact at the element nodes and surfaces, but they are inter- and extrapolated from values found at the Gauss points. Gauss integration over a small volume is used to calculate the internal and reaction forces[Lui et al., 2015].

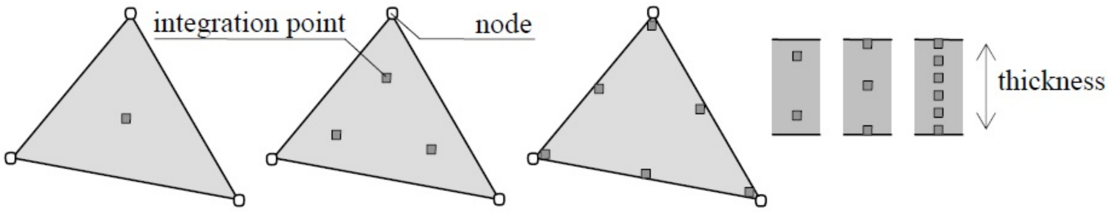

Figure H.2: Locations of Gauss points in triangular elements[Hoogenboom, 2012]

Things that have to be checked after the calculation

The following program checks are based on Finite Element Modeling for Stress Analysis [Cook, 1995].

- A lot of finite element programs work with averaging over the nodes, so the contour line will display no discontinuities. The program user should be careful with this. When there is a discontinuity in the contour line and none is found in the structure at the same place, if this occurs it should be concluded that a finer mesh should have been used. Therefore, when modelling the seawall, mesh refinement should be taken into account. However, refining the mesh increases the run time of the model, so this should be taken into consideration when deciding on a mesh size.

- The FE results have to checked for discrepancies as well. Approximate calculations have to be made before describing the seawall in ANSYS, because there will be a tendency to search for the approximation that agrees with the found FE results if these calculations are performed after finalizing the model. Some hand calculations have been made for simple loads on a wall, these hand calculations and verification of the ANSYS model can be found in section H.2.

- When the structures' supports are not correctly defined, it has a singular stiffness matrix and rigid body movements can take place. When this is the case the software will not be able to compute the nodal displacements.

H.2 Verification of ANSYS calculations

In this appendix a few very basic calculations will be done in order to verify the output of ANSYS. The most important kinds of loads for this project are the gravity load, the hydrostatic pressure and a normal pressure. The gravity load and the hydrostatic pressure will be checked with hand calculations. A normal pressure will not be checked because the hydrostatic pressure is a special kind of pressure, and if the hydrostatic pressure is correct the normal pressure will also be correct. If these calculations are in comparison to each other it can be assumed that a combination of these loads will also be correct. For these verification calculations a very simple block of concrete is used. The block only has a fixed support at the bottom as a support. The dimensions of this block are given in figure H.3.

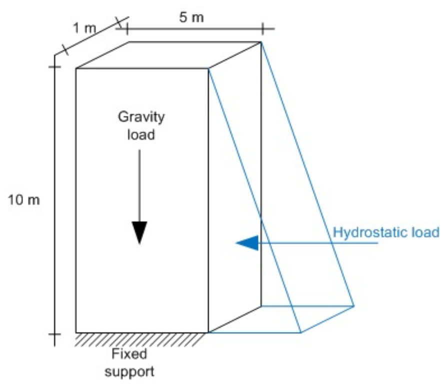

Figure H.3: The block used to verify calculations

Gravity load

The vertical support reaction due to the gravity load can be calculated as shown in equation

H.1

$$\begin{aligned} R_v &= m * a \\ &= l * b * h * \rho * g \\ &= 1 * 5 * 10 * 2300 * 9.81 \\ &= 1128150 \text{ N} \end{aligned} \tag{H.1}$$

The support reaction given by ANSYS is 1127800 N. There is a little difference in the exact value of this support reaction, this can be explained because ANSYS uses 9.8066 m/s^2 as gravity acceleration instead of the used 9.82 m/s^2.

Hydrostatic pressure

For this verification it is assumed the water level coincides with the top of the wall. It is also assumed that the water is salt water. The horizontal support reaction generated by the hydrostatic pressure can be calculated by equation H.2.

$$\begin{aligned} R_h &= 0.5 * \rho * g * h^2 * b \\ &= 0.5 * 1025 * 9.81 * 10^2 * 1 \\ &= 502763 \text{ N} \end{aligned} \tag{H.2}$$

The input needed in ANSYS is the density of the fluid, which is 1025 kg/m^3, the location of the free surface level, which coincides with the top of the wall and is therefore 10 m, and the last input needed is the hydrostatic acceleration, which is 9.81 m/s^2 in y direction and 0 in x and z direction. The horizontal support reaction given by ANSYS is 502750 N, which is almost exactly the same as the calculated horizontal support reaction. The hydrostatic pressure causes a moment and therefore the stresses are interesting as well. The stresses at the front and the back of the block are calculated by equations H.3, H.4 and H.5.

$$\begin{aligned} F_h &= 0.5 * \rho * g * h \\ &= 0.5 * 1025 * 9.81 * 10 \\ &= 50276 \text{ N} \end{aligned} \tag{H.3}$$

$$\begin{aligned} M &= F_h * a \\ &= 50276 * \frac{10}{3} \\ &= 1675888 \text{ Nm} \end{aligned} \tag{H.4}$$

$$\begin{aligned} \sigma &= \frac{M * z}{I} \\ &= \frac{1675888 * 5}{\frac{1}{12} * 1 * 5^3} \\ &= 80442 \text{ Pa} \end{aligned} \tag{H.5}$$

Since the block is symmetric the positive and negative stresses have the same magnitude. The stresses given by ANSYS are 81405 Pa and -77467 Pa. Again there is a slight difference between the hand calculations and the output of ANSYS. These slight differences can be explained because ANSYS computes the stresses at the Gauss points and the hand calculations compute the stresses at the edge of the block. Also the edge disturbance at the bottom of the block causes a slight difference.

H.3 Mesh elements

For the calculation in ANSYS an element size of 0.1 m is considered. The element mesh is shown in figure H.4.

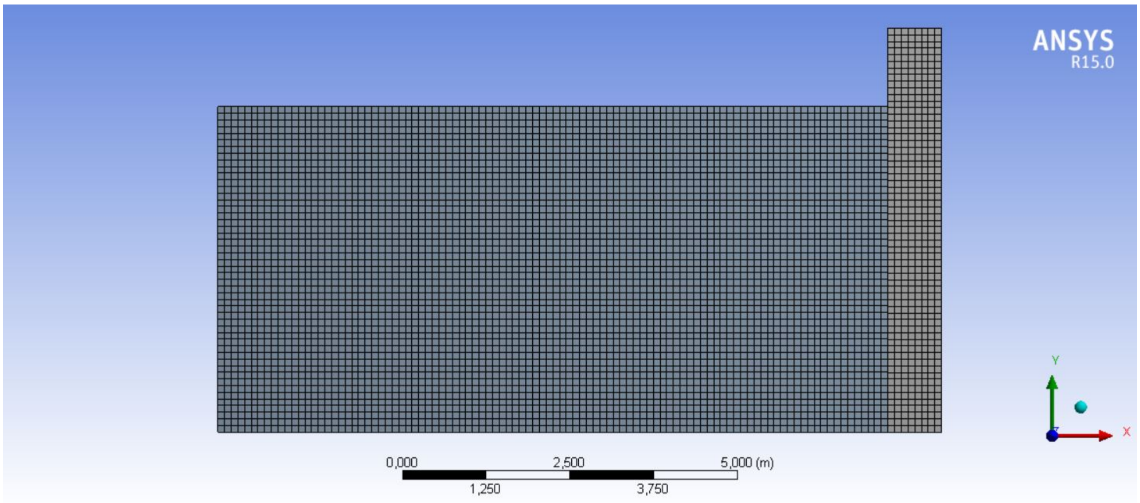

Figure H.4: Element mesh of the ANSYS model

Appendix I

ANSYS results current wall

In this appendix the deformations due to the loads imposed by hurricane Wilma will be shown in figure I.1. The figure I.2 displays the principal stresses in vector notation. The vector notation clearly shows how the structure behaves.

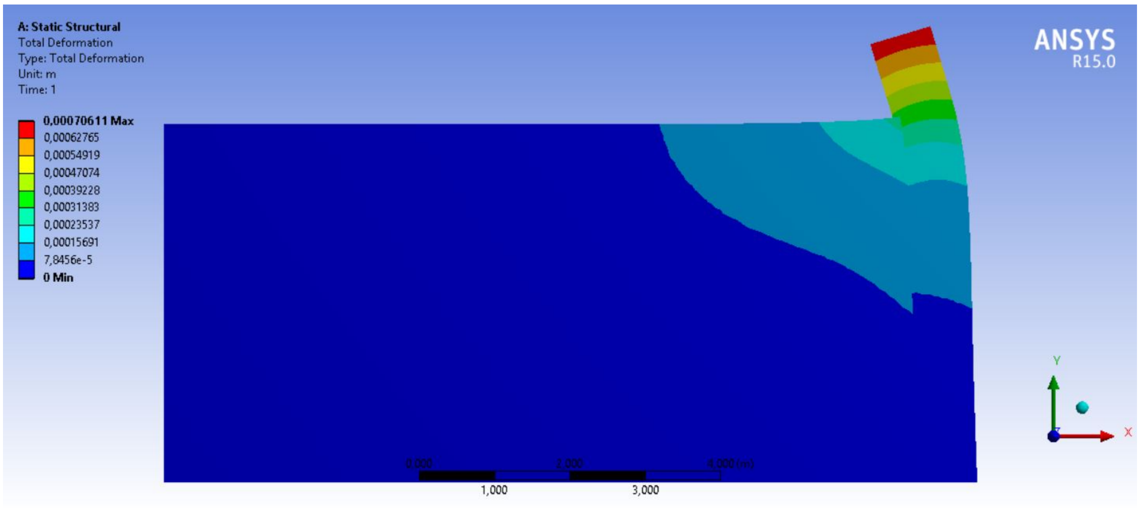

Figure I.1: Total deformation for section 2

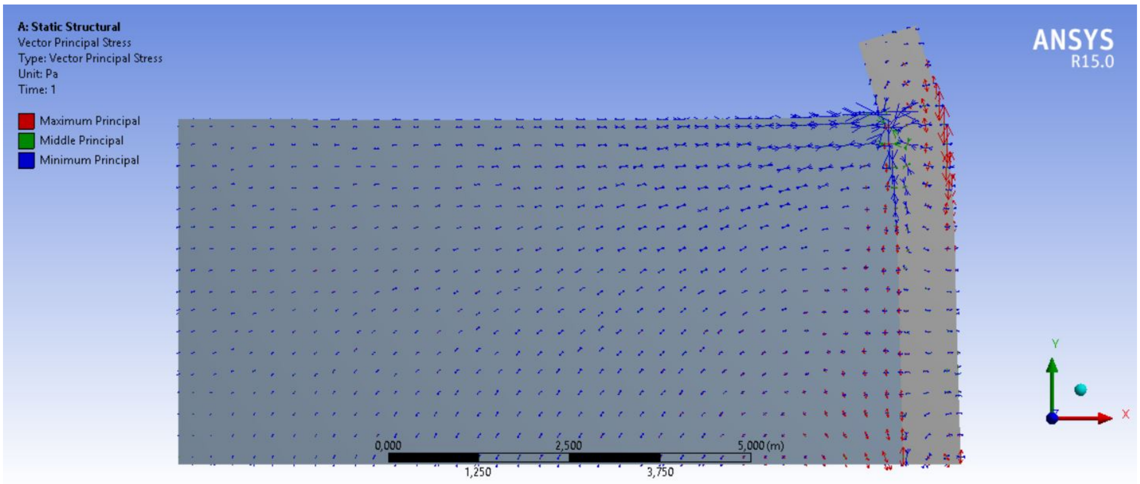

Figure I.2: Principal stresses displayed in vectors for section 2

Appendix J

Fault tree of a quay wall

In figure J.1 the fault tree of a gravity quay wall is presented.

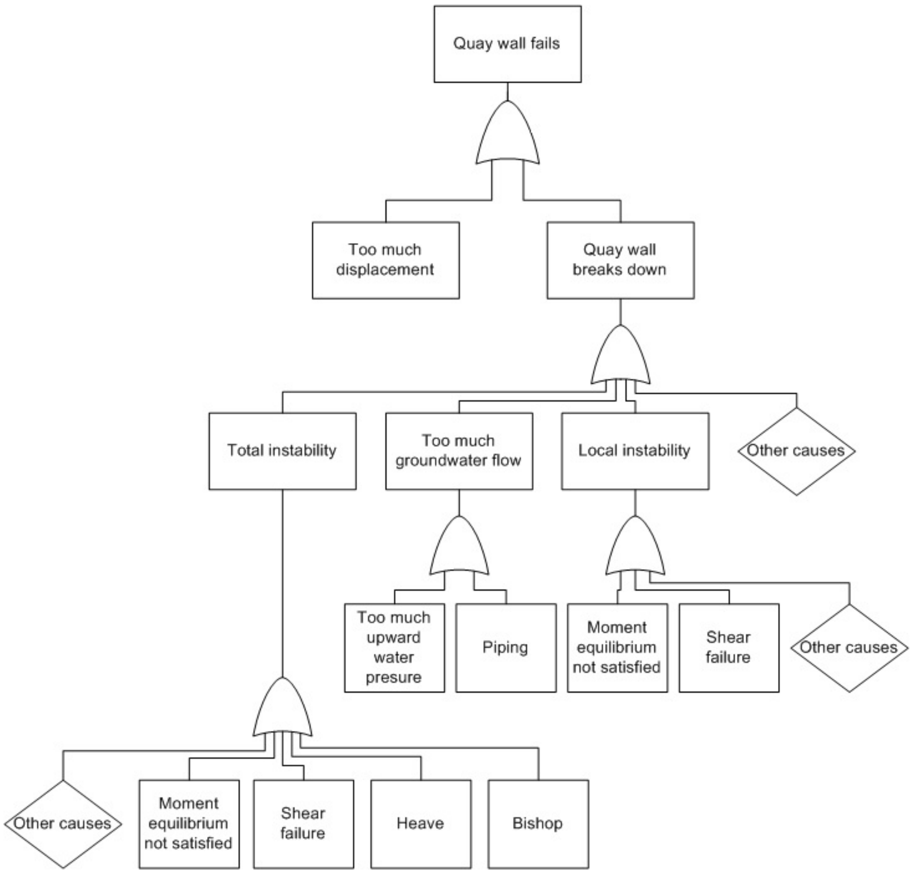

Figure J.1: Fault tree of a gravity quay wall Theunissen [2004]

Appendix K

Detailed structural design

K.1 General design procedures

All of the equations mentioned in this section are based on the EN-1991-1-1 For each subsection it is mentioned which section of the EN-1991-1-1 is used to determine the equation and its factors.

K.1.1 Material properties

C35/45	MPa
f_{ck}	35
f_{cd}	23.3
$f_{ctk,0.05}$	2.2
f_{ctd}	1.5
E_{cm}	34000
v_{min}	0.44

Table K.1: Strength properties concrete class C35/45

K.1.2 Slab thickness

For slabs there are rules of thumb to make a first design for the wall thickness. Formula K.1, based on the deflection criterion in EN-1992-1-1 cl. 7.4.1 is used during the design. The formula gives a conservative estimation for the effective thickness of the wall.

$$d \geq \frac{\sigma_s l}{9200(0.8 - 0.3\rho_l)} \tag{K.1}$$

With:

σ_s The tensile stress in the longitudinal reinforcement for which a first value of 1.3 MPa is taken. This is the maximum tensile stress found by ANSYS for the current seawall.

l_{eff} Effective length of the span. For l_{eff} a value of $2l$ is taken as the top of the wall is considered to be a cantilever slab with the supported bottom part of the wall acting as a fixed support. This is an approximation, but can be used for a first design. The free end of the wall has an average length, after elevation of the crest level, of 1.2 m.

ρ_l The reinforcement ratio in percentage. The value taken is 0.30% as explained in section 7.2.1

K.1.3 Reinforcement and cover

Longitudinal reinforcement

According to EN-1992-1-1 cl. 9.2.1.1 the minimum longitudinal reinforcement ratio in order to prevent brittle failure is given as:

$$\rho_{min} = 0.26 \frac{f_{ctm}}{f_{yk}} \tag{K.2}$$

For concrete class C35/45 ρ_{min} is 0.17%.

In Plate analysis, theory and application, Volume 1 it is advised to calculate the needed reinforcement on the basis of two reinforcement stresses $\bar{\sigma}_{zz}$ and $\bar{\sigma}_{yy}$. The reinforcement stresses should be equal to the product of the design yield stress f_yd of the reinforcement and the reinforcement ratio ρ_l. This is defined as:

$$\bar{\sigma}_{zz} = \sigma_{zz} + |\sigma_{zy}| = f_{yd} * \rho_z \tag{K.3}$$

$$\bar{\sigma}_{yy} = \sigma_{yy} + |\sigma_{yz}| = f_{yd} * \rho_y \tag{K.4}$$

From these equations ρ_x and ρ_y follow.

Shear reinforcement

According to EN-1992-1-1 cl. 9.2.2 the minimum shear reinforcement ratio is defined as:

$$\rho_{w,min} = \left(0.08\sqrt{f_{ck}}\right) / f_{yk} \tag{K.5}$$

For the concrete class C35/45 and a f_{yk} of steel of 500 MPa, the minimum reinforcement ratio $\rho_{w,min}$ becomes 0.09%. The maximum spacing of the shear reinforcement elements is given as:

$$s_{max} = 0.75d\,(1 + \cot\alpha) \tag{K.6}$$

With α being the angle between the longitudinal direction and the shear reinforcement.

K.1.4 Shear resistance at the interface between concrete layers of different ages

According to EN-1992-1-1 cl. 6.2.5 the shear resistance of the interface between two layers of concrete is described as:

$$v_{Rd} = c * f_{ctd} + \mu * \sigma_n + \rho * f_{yd} \left(\mu \sin\alpha + \cos\alpha\right) \leq 0.5 * \nu * f_{cd} \tag{K.7}$$

Where:

c and μ	are factors which depend on the roughness of the interface. A rough interface is assumed as the interface should be treated correctly by blasting.
f_{ctd}	is the design tensile strength of the material given as $\frac{f_{ctk_{0,05}}}{\gamma_c}$. For concrete class C35/45 $f_{ctk_{0,05}} = 2.2$ MPa and the material factor $\gamma_c = 1.5$.
σ_n	is the stress per unit area caused by the minimum external normal force or moment that can act simultaneously with the shear force, positive for compression and negative for tension. When σ_n is tensile $c * f_{ctd}$ should be taken as 0.
ρ	$= \frac{A_s}{A_c}$

The first term describes the adhesion. The second term depends on the surface roughness and describes the external normal force contribution. The third term represents the reinforcement contribution.

K.1.5 Crack width

According to EN-1992-1-1 cl. 7.3.1 the recommended value of w_{max} for an Exposure class of XS3 is 0.3 mm. The crack width can be calculated with the calculations given in EN-1992-1-1 cl 7.3.4:

$$w_k = s_{r,max} * (\epsilon_{sm} * \epsilon cm) \tag{K.8}$$

Where:

w_k	is the crack width in m
$s_{r,max}$	is the maximum crack spacing in m, determined by equation K.10
$(\epsilon_{sm} * \epsilon cm)$	is the difference in mean strain of the reinforcement and the concrete in between the cracks

$$\epsilon_{sm} * \epsilon cm = \frac{\sigma_s - k_t \frac{f_{ct,eff}}{\rho_{p,eff}} * (1 + \alpha_e * \rho_{p,eff})}{E_s} \geq 0.6 * \frac{\sigma_s}{E_s} \tag{K.9}$$

Where:

σ_s	is the stress in the reinforcement steel in Pa, calculated by $\sigma_s = \frac{N_s}{A_s}$ where N_s is determined using the results from ANSYS
α_e	is the ratio $\frac{E_s}{E_{cm}}$
$\rho_{p,eff}$	is the effective reinforcement ratio, because only reinforcement steel will be implemented in the new wall $\rho_{p,eff} = \frac{A_s}{A_c}$
k_t	is a factor of 0.6 for short term loading

$$s_{r,max} = \frac{k_3 * c + k_1 * k_2 * k_4 * \phi}{\rho_{p,eff}} \tag{K.10}$$

Where:

c	is the cover in longitudinal direction in m
k_1	a factor of 0.8 for high bond bars
k_2	is a factor of 0.5, for pure bending, which represents the actual case the best.
k_3	is a factor of 3.4
k_4	is a factor of 0.425

Together with the results given by ANSYS, the equations K.8, K.9 and K.10 divine the crack width. It the crack width w_k exceeds the w_{max} of 0.3 mm the amount of reinforcement needs to be increased until w_k is smaller than 0.3 mm.

K.2 Repairing alternative

K.2.1 Slab thickness

According to formula K.1 in section K.1.2 the slab effective thickness should be at least 480 mm. The current wall has a thickness of about 800 mm, so this is sufficient. This wall thickness is enough to fit the reinforcement steel and a concrete cover of 100 mm.

K.2.2 Reinforcement

From formulas K.3 and K.4 follows that the minimum reinforcement in the horizontal longitudinal direction is 0.1% and in the vertical direction 0.7%. For the vertical direction this leads to a longitudinal reinforcement of ⊘20 with a spacing of 55 mm. For the horizontal longitudinal direction a minimal amount to make a workable reinforcement cage is enough to meet the reinforcement criterion. It is recommended that the constructing company this workable condition.

The shear reinforcement ratio will be 0.09%. For the determined thickness of the wall s_{max} will be 520 mm. These two criteria lead to a reinforcement of ⊘12 with a spacing of 200 mm.

K.2.3 Shear resistance interface

Assuming that there will be a total reinforcement ratio of 1% when the dowels are added at the interface, the maximum shear resistance v_{Rd} will be around 2.5 MPa. The results from the ANSYS model in appendix L show that the maximum shear stress is around 0.06 MPa at the interface, so the resistance is sufficient.

K.2.4 Crack width

For the initial calculation a reinforcement distributed as discussed in K.2.2 is used. The steel stresses needed to determine the crack width is calculated with the help of the results from ANSYS. In figure L.3 in appendix L the normal stresses in y direction are shown. It is clear that half of the wall is in tension and the other half is in compression. As concrete can not take tensile stresses the tension zone is assumed to be cracked. Therefore all the tensile stresses have to be carried by the reinforcement steel. In order for the wall to be in equilibrium the tensile stresses in the steel and the compressive stresses in the concrete have to be the same. Thus with figure L.3 it can be said that the compressive concrete stresses are equal to $\sigma_{compressive\ concrete} = 1.14$. Given the reinforcement distribution, the tensile stresses in the steel are equal to $\sigma_{steel} = 79.8$ MPa, which is far below the design yield strength of the steel.

Using equations K.8, K.9 and K.10 it follows that the crack width $w_k = 0.13$ mm. Therefore it can be concluded that the distribution of reinforcement is enough to prevent large cracks from occurring during a $\frac{1}{50}$ year storm. Even though it follows from the calculations the concrete cover is enough for the reinforcement steel to be protected from the sea water, it is strongly recommended to investigate the effects of tropical waters, since the EuroCode is based on European conditions.

K.2.5 Evaluation of optimal interface location

The optimal location of the interface between the current wall and the new constructed top has been evaluated using ANSYS. It is advised to avoid tension in the old part of the wall, since this part is not reinforced and according to formula K.7 in appendix K the shear resistance of the interface is also reduced when the interface is exposed to tensile stresses. The results of the analysis are shown in figures L.1 and L.2 in appendix L. The ideal place for this intersection is about 0.70 meters above MSL. From the results of ANSYS it follows that no high tensile stresses occur at the interface location.

K.2.6 Evaluation of the design with ANSYS

Return period of 50 years The design has been evaluated with ANSYS and a raised wall designed as discussed is not strong enough to resist the loads of a storm with a return period of 50 years. The computed stresses in the structure can be found in figure L.4 and L.5 in appendix L. The results show that no high tensile and shear stresses are found in the old Malecón seawall so the resistance of the old wall will be sufficient to withstand the imposed loads. However, for the new top part of the wall the tensile stresses do exceed the tensile strength of the used concrete class, so cracking can occur. The reason for these high stresses is the type of connection of the new wall to the old wall. Constructing a thicker top part of the wall is no option as it should fit on top of the old wall, so a redesign of the cross section is not possible. The problem can be solved by using a higher class of concrete. With the use of concrete class C40/50 the new top will be able to resist the tensile stresses. The shear stresses in the new wall still exceed the shear strength of C40/50, but only in a small zone of the wall and the shear resistance has been raised by the use of shear reinforcement. Therefore the design is sufficient for this load condition.

ULS conditions The results of the computed stresses by ANSYS can be found in figure L.6 and L.7 in appendix L. When the ULS condition is analysed it is noted that the tensile stresses at the sidewalk level are relatively high and they exceed the tensile strength of the concrete. The longitudinal reinforcement prevents the structure for ultimate failure, but the concrete will be cracked. Large cracks will lead to corrosion of the reinforcement steel. As recommended in section K.2.4 the influence on tropical waters on the calculated crack width has to be checked. The shear stresses are concentrated at the area around sidewalk level, but with adequate shear reinforcement these shear stresses do not cause a problem, as they are not very high with a maximum value of 0.9 MPa.

For a not raised wall with a height in the same order as that of section 1, 3, 4 and 5, the stresses in this alternative do not exceed the strength of the design as can be seen in figure L.8 and L.9 in appendix L.

K.3 Additional alternative

K.3.1 Slab thickness

According to formula K.1 in section K.1.2, the effective thickness d of the slab of the seaside wall will be about 480 mm. With a concrete cover the total thickness will be around 600 mm. After evaluating the design with the use of ANSYS, it was clear that the tensile stress in the concrete slab σ_s is 1.5 MPa instead of the value of 1.3 MPa as described in section K.1.2, so the total thickness will be 700 mm.

K.3.2 Reinforcement and cover

From formulas K.3 and K.4 follows that the minimum reinforcement in the horizontal longitudinal direction is 0.02% and in the vertical direction 0.25%. For the vertical direction this leads to a longitudinal reinforcement of ⌀20 with a spacing of 150 mm. For the horizontal longitudinal direction a minimal amount to make a workable reinforcement cage is enough to meet the reinforcement criterion. It is recommended that the constructing company this workable condition.

The shear reinforcement ratio is 0.09%. For the determined thickness of the wall, s_{max} will be 420 mm. These two criteria lead to a shear reinforcement of ⌀10 with a spacing of 200 mm.

K.3.3 Shear resistance interface

The shear resistance at the interface is not of interest as there is no horizontal interface and the shear stresses are critical in horizontal direction.

K.3.4 Crack width

From the results of ANSYS as discussed in the next section it appeared that the tensile stresses do not exceed the strength of the additional wall. Therefore the crack width calculation is not of interest as the concrete will not be cracked for ULS conditions.

K.3.5 Evaluation of the design with ANSYS

Return period of 50 years The discussed design was evaluated with ANSYS and it follows that the newly designed wall is strong enough to resist the load of a $\frac{1}{50}$ year storm. The results can be found in figure L.10 and figure L.11 in appendix L. The results show that no high tensile and shear stresses are found in the old Malecón seawall so the resistance of the old wall will be sufficient to withstand the imposed loads. For the added wall the tensile strength is not exceeded at the critical zone at the sidewalk level, so cracking will be prevented. At the toe of the wall the tensile stresses are slightly higher, but this could also be due to edge disturbance. It is recommended to investigate this more carefully or apply an extra protection against corrosion of the reinforcement steel and scour. The shear stresses are within the given restrictions, so the design is sufficient. It has been investigated if a thinner seaside wall would be sufficient too, but then the shear stresses in the old wall would be too high and as it has no shear reinforcement, there would be a high possibility of failure.

ULS conditions The results of the computed stresses by ANSYS can be found in figure L.12 and L.13 in appendix L. When the design is evaluated for the ULS condition it can be seen that the shear stresses in the old seawall are critical. They exceed the shear strength of the wall in a large zone just above the sidewalk level. The maximum principal stresses are not critical for the design, but there are tensile stresses present in the old seawall. For this reason it is recommended to increase the thickness of the added wall. It was found that for a thickness of 800 mm these tensile stresses would be low enough in order for the old concrete wall to resist the maximum tensile stresses. For the toe of the wall it is recommended to investigate the stresses in the concrete in more detail as described in section K.3.5.

K.4 Partial replacement alternative

K.4.1 Slab thickness

As the old part of the wall can be used as a load bearing element in this alternative, the slab thickness has been evaluated in an alternative manner. The old wall is only present in the part that is under compression and since concrete is very strong in compression the old concrete can work together with the newly designed wall in this way. For the slab thickness at the bottom of the new wall therefore a minimum value of 400 mm is used. This thickness is needed to fit the reinforcement necessary and leave room for the recommended cover. This thickness of the bottom part leads to a thickness of the top part of the new wall of $800 + 400 = 1200$ mm.

K.4.2 Reinforcement and cover

From formulas K.3 and K.4 follows that the minimum reinforcement in the horizontal longitudinal direction is 0.02% and in the vertical direction 0.28%. For the vertical direction this leads to a longitudinal reinforcement of ⌀20 with a spacing of 90 mm. For the horizontal longitudinal direction a minimal amount to make a workable reinforcement cage is enough to meet the reinforcement criterion. It is recommended that the constructing company this workable condition.

The shear reinforcement ratio will be 0.09% as the minimum shear reinforcement is used for the design. For the top part of the wall with a thickness of 1200 mm, s_{max} will be 820 mm. This leads to a reinforcement of ⌀12 with a spacing of 150 mm. For the bottom part of the new wall the thickness is 400 mm, as the part of the wall where the reinforcement is placed, has a smaller thickness than at the top. For these values s_{max} will be 220 mm. So the reinforcement used will be ⌀12 with a spacing of 200 mm.

K.4.3 Shear resistance

Assuming that the total reinforcement ratio at the interface is about 1% when the dowels are incorporated, the shear resistance of the interface will be 2 MPa. The results from the ANSYS model in appendix L show that the maximum shear stress is around 0.06 MPa at the interface, so the resistance is sufficient.

K.4.4 Crack width

For the initial calculation a reinforcement distribution as discussed in K.4.2 is used. The steel stresses needed to determine the crack width are calculated with the help of the results from ANSYS. In figure L.16 the normal stresses in y direction are shown. From the figure it can be concluded that about 0.4 m is in compression and 0.8 m is in tension.

In order for the wall to be in equilibrium the tensile stresses in the steel and the compressive stresses in the concrete have to be the same. From figure L.16 it can be concluded that the average compressive stress in the concrete is equal to $\sigma_{compressive\ concrete} = 1.015$ MPa. Given the amount of reinforcement the tensile stress in the steel will be equal to $\sigma_{steel} = 116$ MPa, which is far below the design yield strength of the steel.

Using equations K.8, K.9 and K.10 it follows that the crack width $w_k = 0.19$ mm. Again the reinforcement is enough to prevent too large cracks. However it is still recommended to further investigate the influences of the tropical waters.

K.4.5 Evaluation of optimal interface location

For this design as well, the location of the interface between the current wall and the new top part of the wall has been optimized using ANSYS, as was described in section K.2.5. The results can be seen in figure L.14 and L.15 in appendix L. The location has been determined such that no high tensile and shear stresses occur in the current wall. The optimal location of the intersection is around 2.3 meters above MSL.

K.4.6 Evaluation of the design with ANSYS

Return period of 50 years Figure L.17 and figure L.18 in appendix L show the results of the partial replacement alternative for a storm with a $\frac{1}{50}$ year return period. It follows that the tensile strength of the concrete is not exceeded for this load condition. The shear stresses do exceed the shear resistance of the concrete cross section in a small zone just above sidewalk level, but the shear reinforcement added is sufficient to resist the loads.

Conditions of hurricane Wilma (or to be defined ULS condition) For the ULS condition, the stresses in the designed structure have been computed in ANSYS as well. These results can be found in figures L.19 and L.20 in appendix L. It is noticed that the tensile stresses in the new concrete wall slightly exceed the strength, therefore the concrete will be cracked. The crack width, however, will not be governing as is calculated in K.4.4.

The longitudinal reinforcement present in the cross section is sufficient to withstand this tensile force. At the toe of the wall the cracking of the concrete could lead to scour and corrosion easily. It is therefore recommended to investigate this more carefully and apply an extra protection against corrosion of the reinforcement steel and scour. The shear stresses are taken on by the shear reinforcement, as described for the return period of 50 years. In the old wall the tensile and shear stresses are low and do not exceed the resistance.

Appendix L

ANSYS results detailed structural design of the Malecón seawall

In order to model all the structural alternatives in ANSYS, the connection between the old wall and the new wall was modelled as bonded. Therefore no sliding or separation will occur between the faces of the old and the new wall. With the use of a bonded connection the contact area will not change and can therefore be seen as a linear solution [ANSYS, Inc., 2007].

L.1 Repairing alternative

L.1.1 Stresses at the interface between the old and new part of the wall

The stresses at the interface location are presented in figure L.1 and figure L.2.

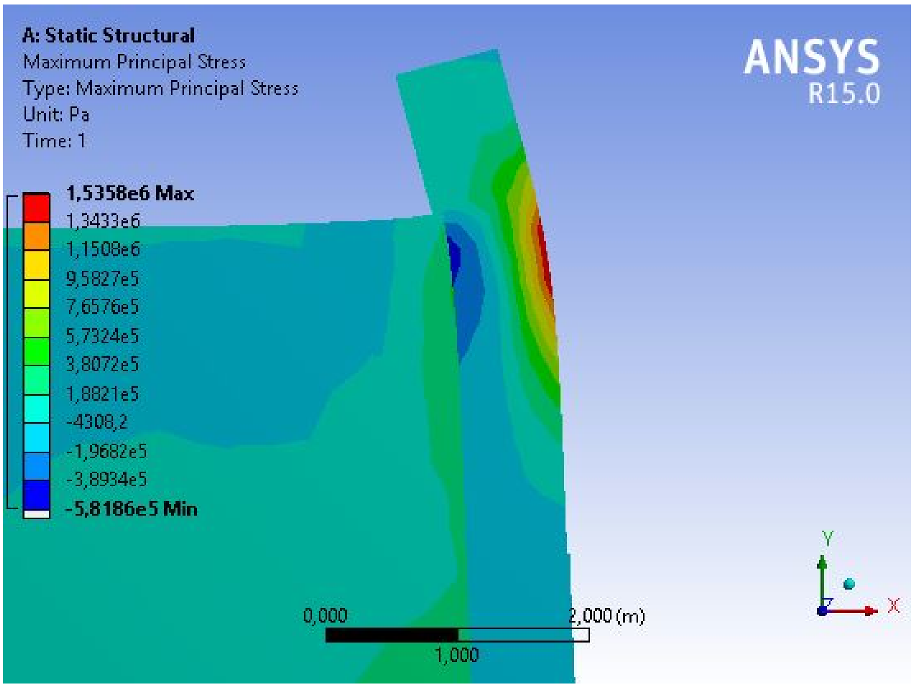

Figure L.1: Maximum principal stresses at the interface between the old and new part of the wall for the repairing alternative

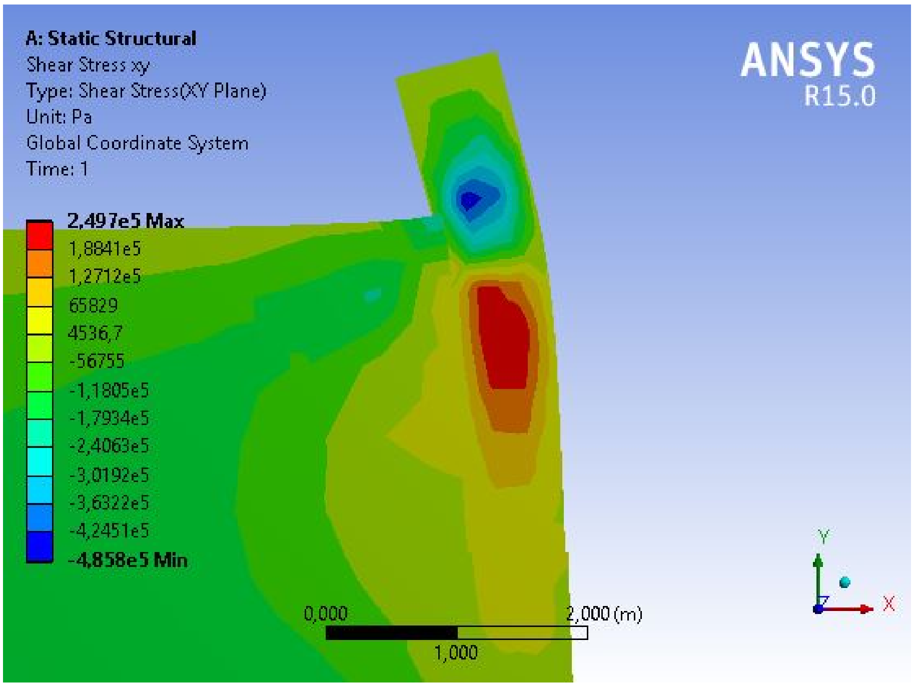

Figure L.2: Shear stresses at the interface between the old and new part of the wall for the repairing alternative

L.1.2 Crack width evaluation

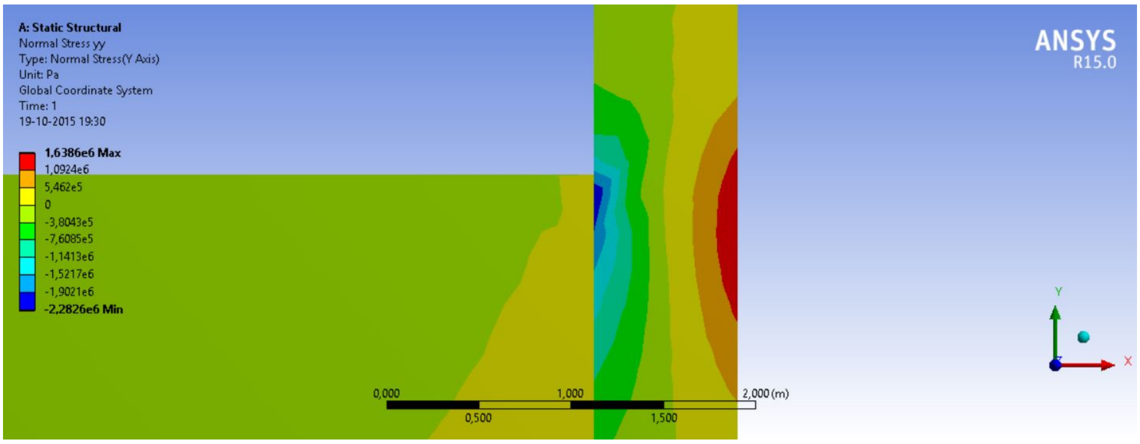

Figure L.3: Normal stress in y direction for a $\frac{1}{50}$ year storm

L.1.3 Stresses for a storm with a return period of 50 years

Figure L.4 shows the maximum principal stresses and figure L.5 the shear stresses in the structure.

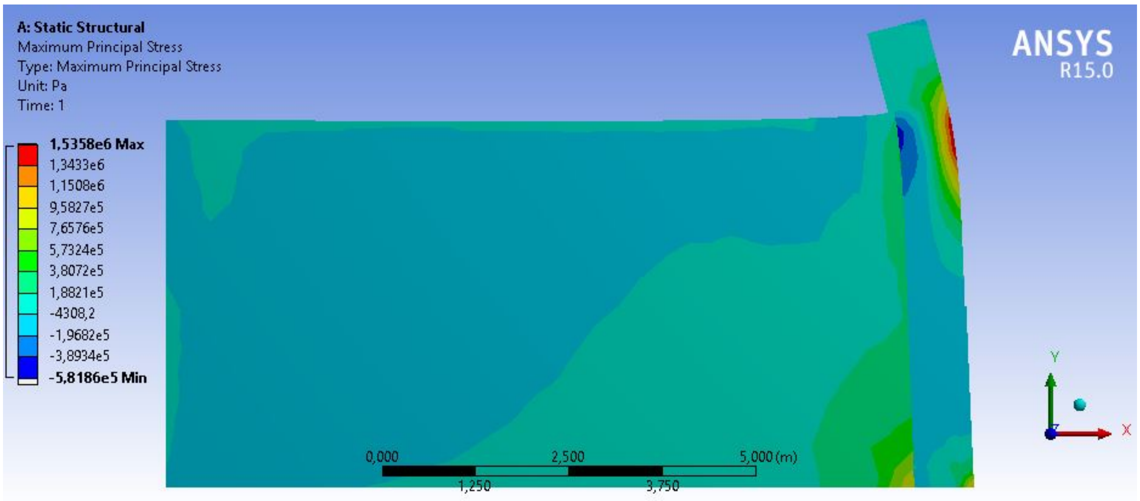

Figure L.4: Maximum principal stresses for the repairing alternative for $T_{return} = 50y$

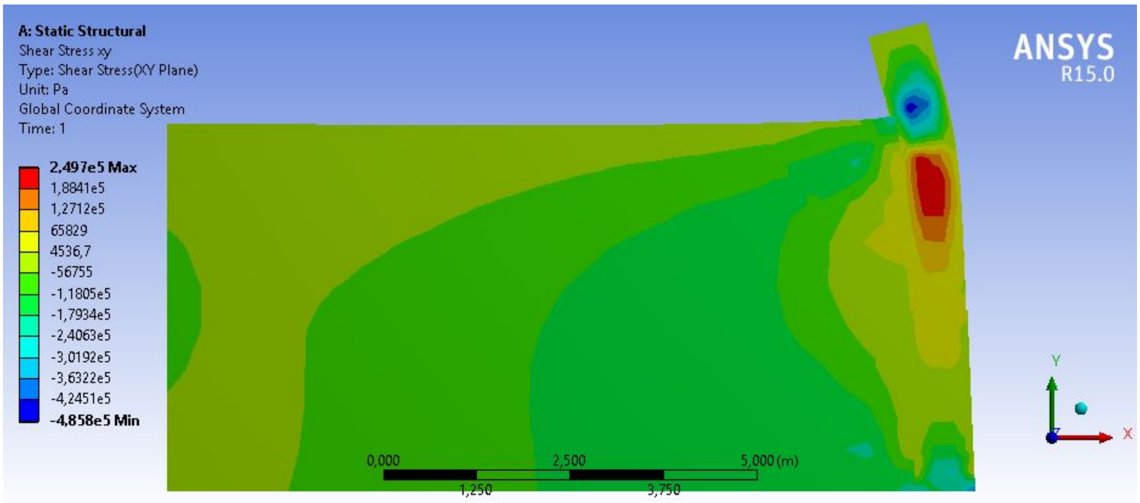

Figure L.5: Shear stresses for the repairing alternative for $T_{return} = 50y$

L.1.4 Stresses for the ULS condition

The maximum principal stresses are showed in figure L.6 and the shear stresses in figure L.7.

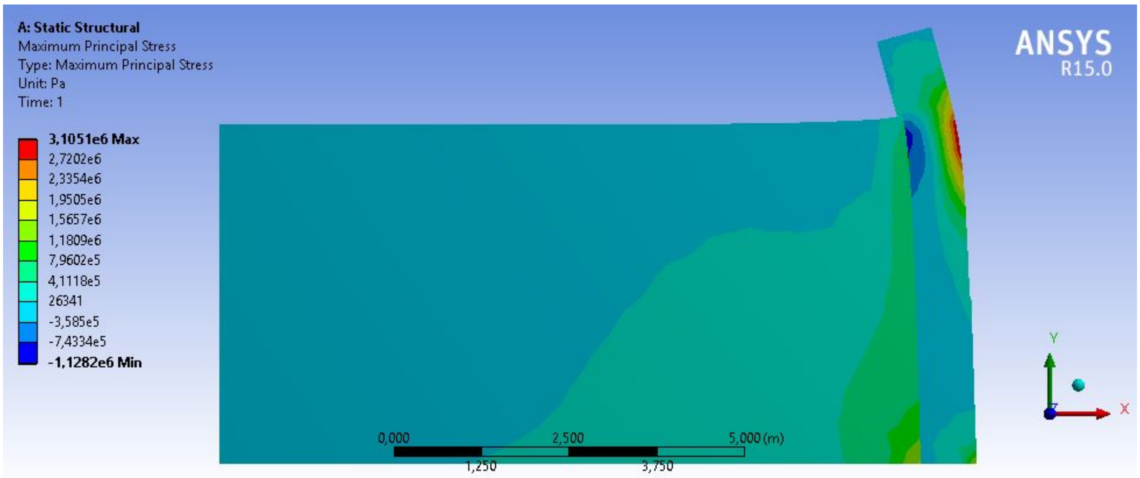

Figure L.6: Maximum principal stresses for the raised repairing alternative for the ULS condition

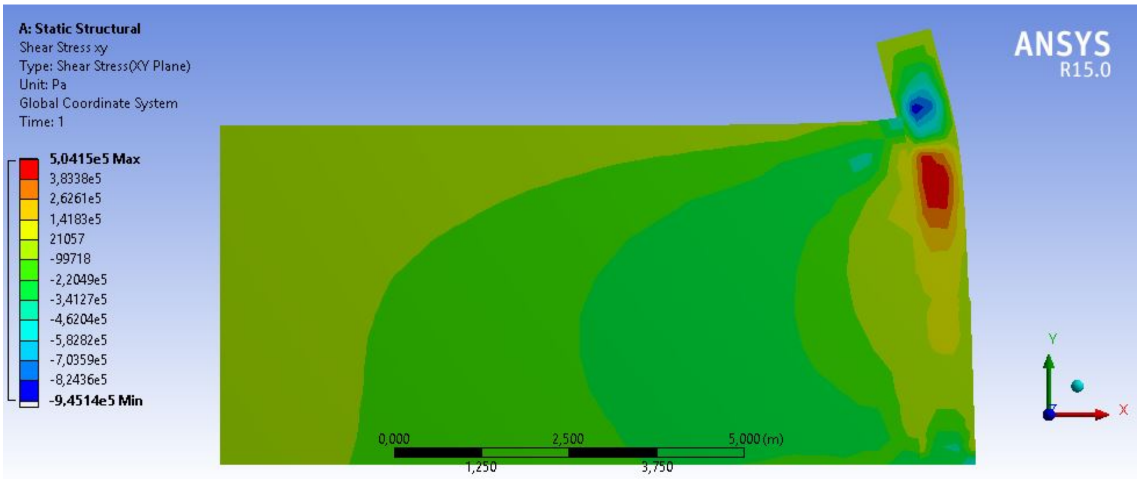

Figure L.7: Shear stresses for the raised repairing alternative for the ULS condition

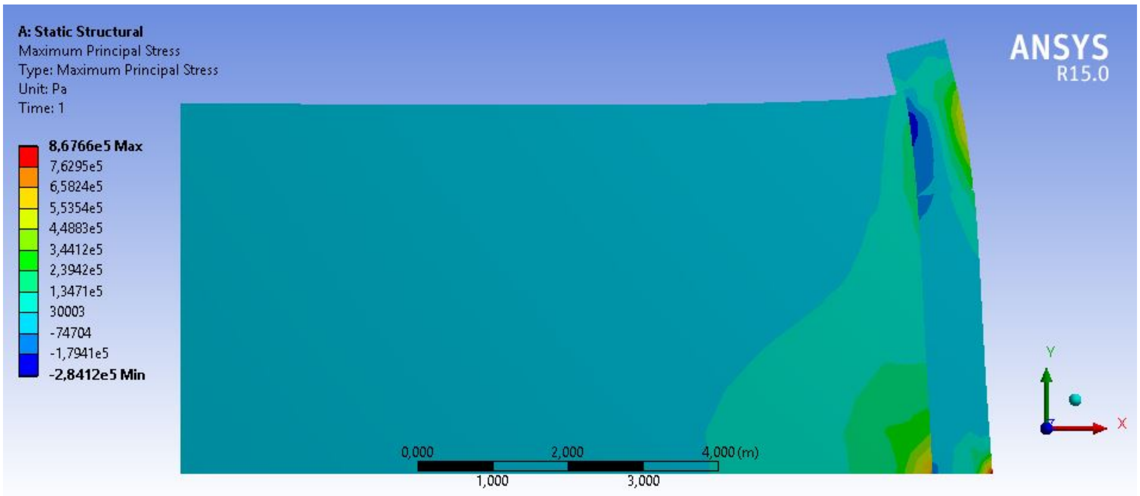

Figure L.8: Maximum principal stresses for the unraised repairing alternative for the ULS condition

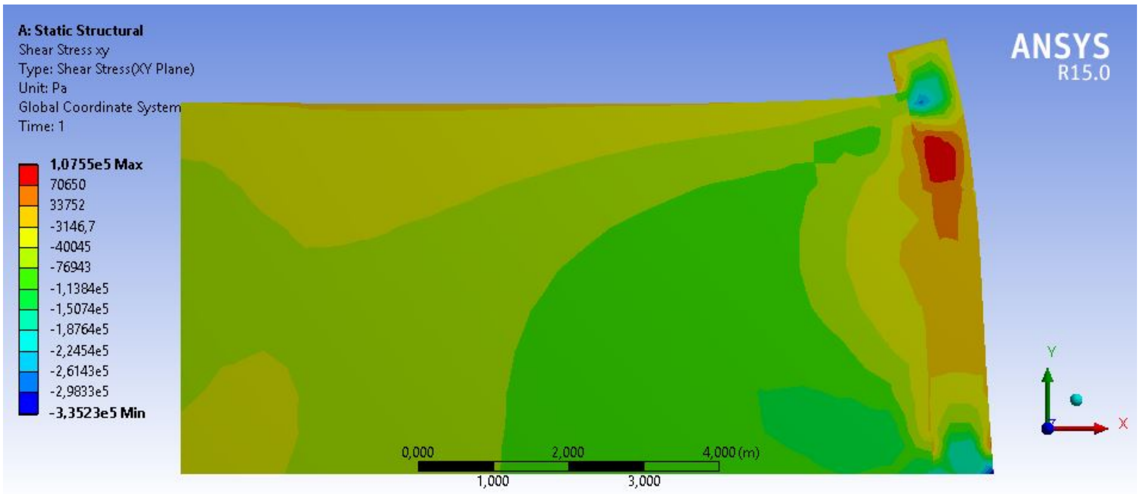

Figure L.9: Shear stresses for the unraised repairing alternative for the ULS condition

L.2 Addition alternative

L.2.1 Stresses for a storm with a return period of 50 years

Figure L.10 shows the maximum principal stresses and figure L.11 shows the shear stresses in the structure.

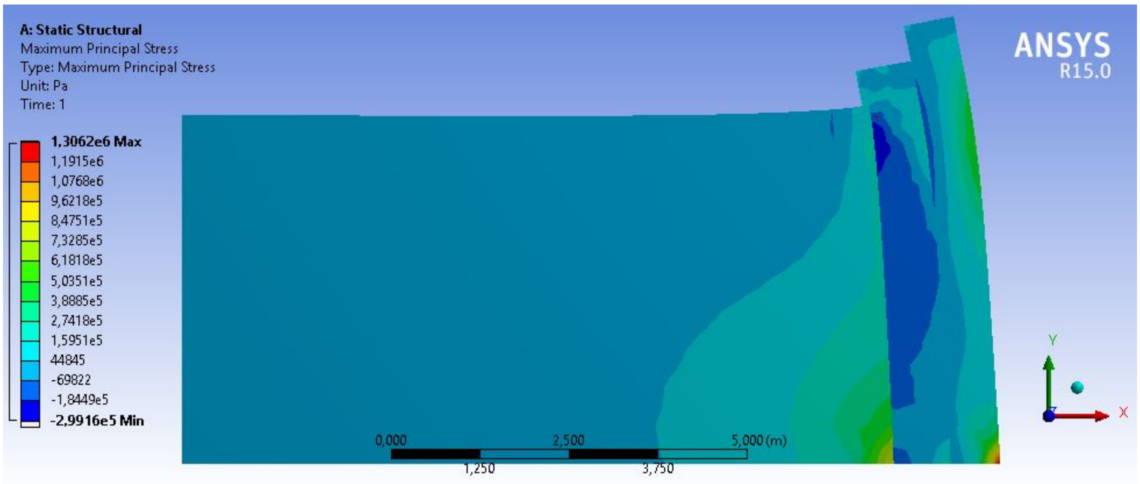

Figure L.10: Maximum principal stresses for the addition alternative for $T_{return} = 50y$

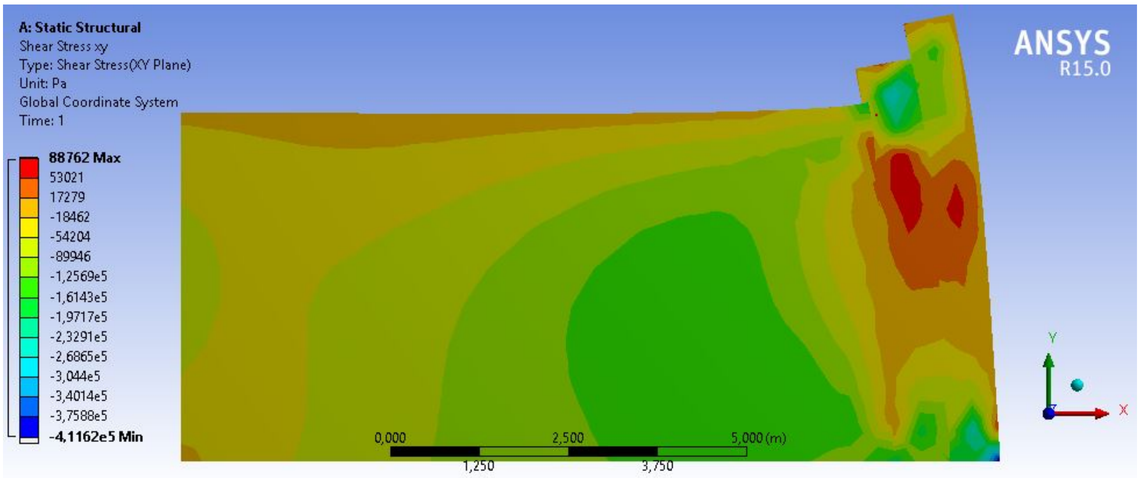

Figure L.11: Shear stresses for the addition alternative for $T_{return} = 50y$

L.2.2 Stresses for the ULS condition

Figure L.12 shows the maximum principal stresses and figure L.13 shows the shear stresses in the structure during the ULS condition.

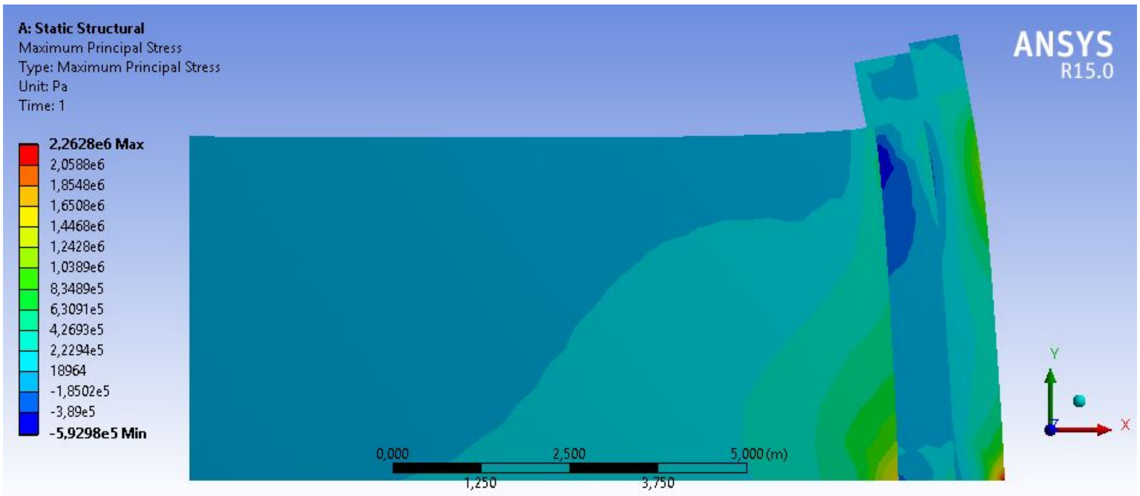

Figure L.12: Maximum principal stresses for the addition alternative for the ULS condition

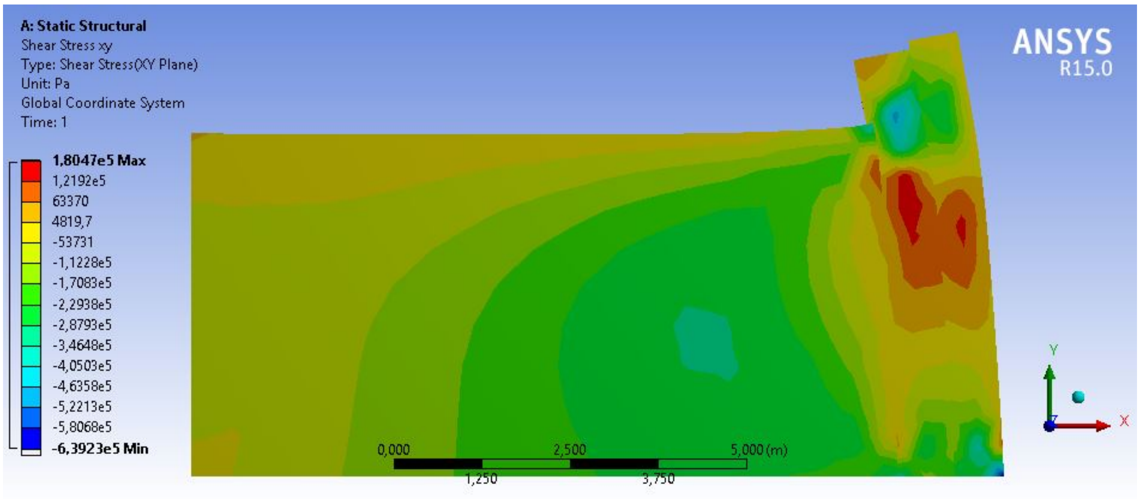

Figure L.13: Shear stresses for the addition alternative for for the ULS condition

L.3 Partial replacement alternative

L.3.1 Stresses at the interface between the old and new part of the wall

The stresses at the interface location are presented in figure L.14 and figure L.15.

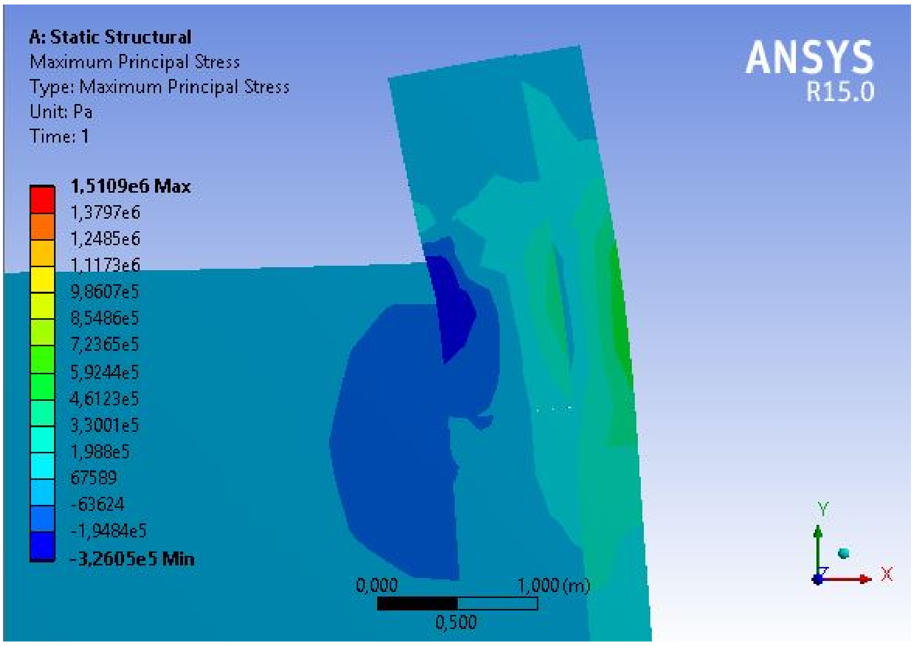

Figure L.14: Maximum principal stresses at the interface between the old and new part of the wall for the partial replacement alternative

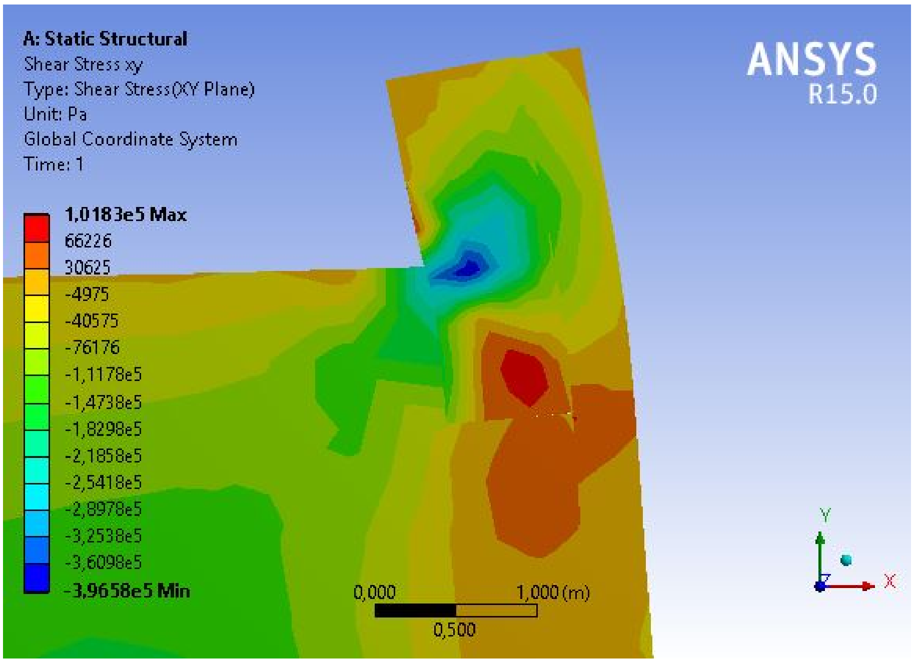

Figure L.15: Shear stresses at the interface between the old and new part of the wall for the partial replacement alternative

L.3.2 Crack width evaluation

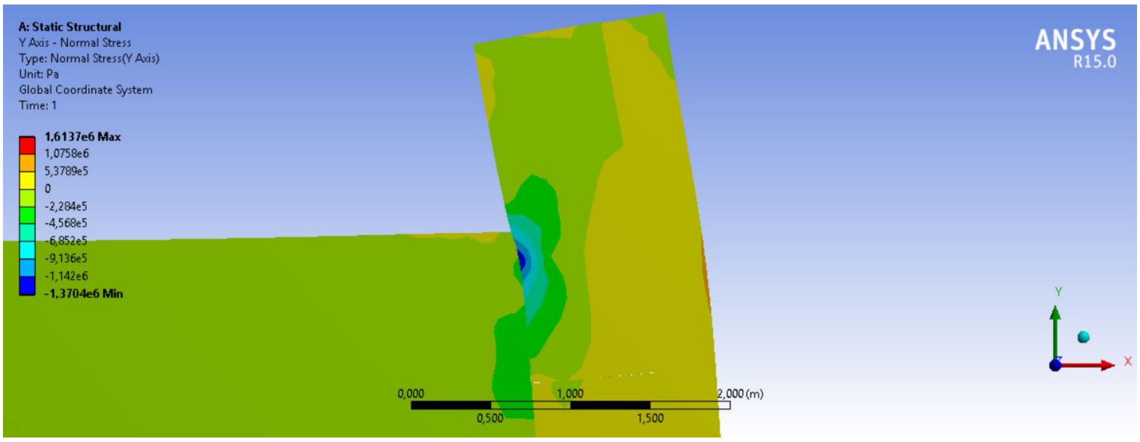

Figure L.16: Normal stress in y direction for a $\frac{1}{50}$ year storm

L.3.3 Stresses for a storm with a return period of 50 years

Figure L.17 shows the maximum principal stresses and figure L.18 shows the shear stresses in the structure.

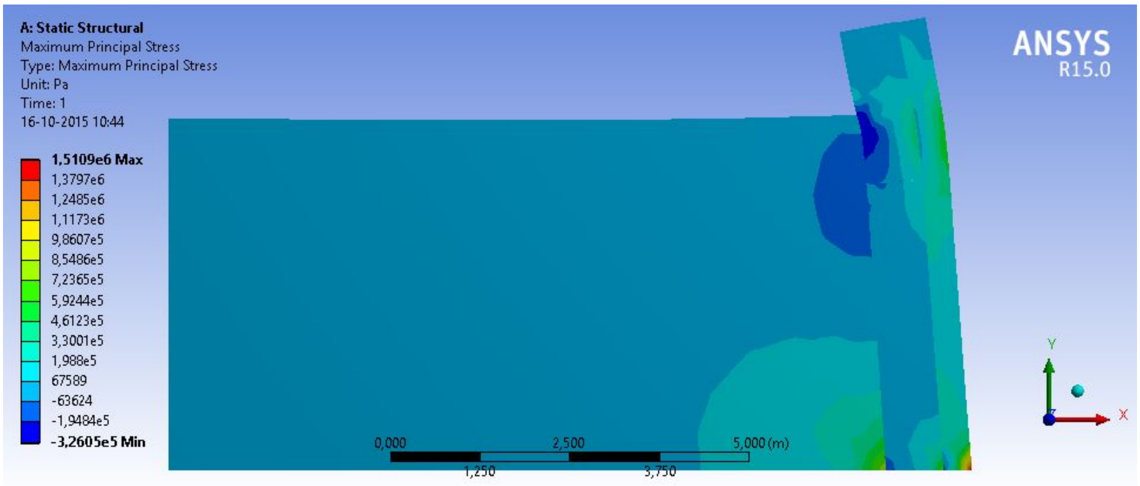

Figure L.17: Maximum principal stresses at the interface between the old and new part of the wall for the partial replacement alternative

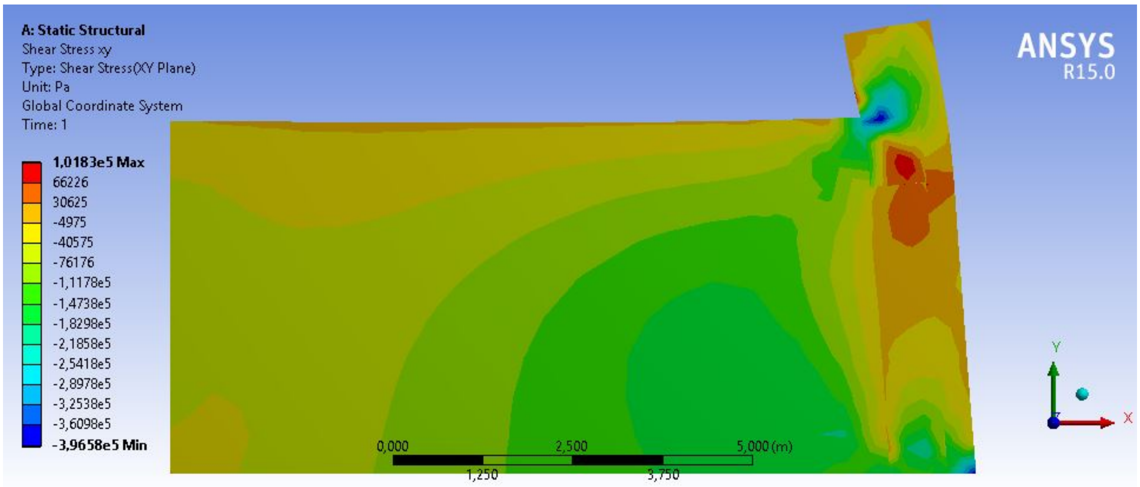

Figure L.18: Shear stresses at the interface between the old and new part of the wall for the partial replacement alternative

L.3.4 Stresses for the ULS condition

Figure L.19 shows the maximum principal stresses and figure L.20 shows the shear stresses in the structure during the ULS condition.

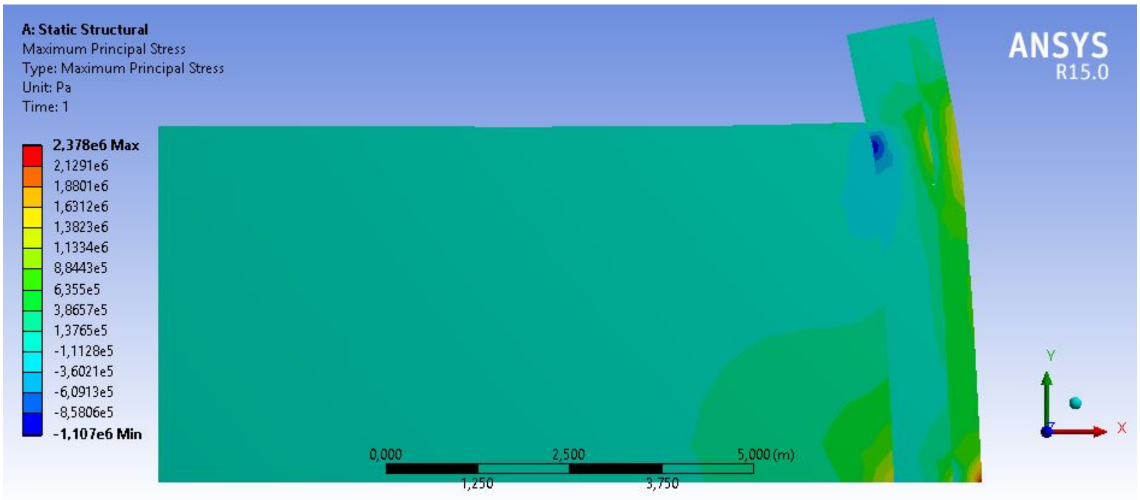

Figure L.19: Maximum principal stresses for the partial replacement alternative for the ULS condition

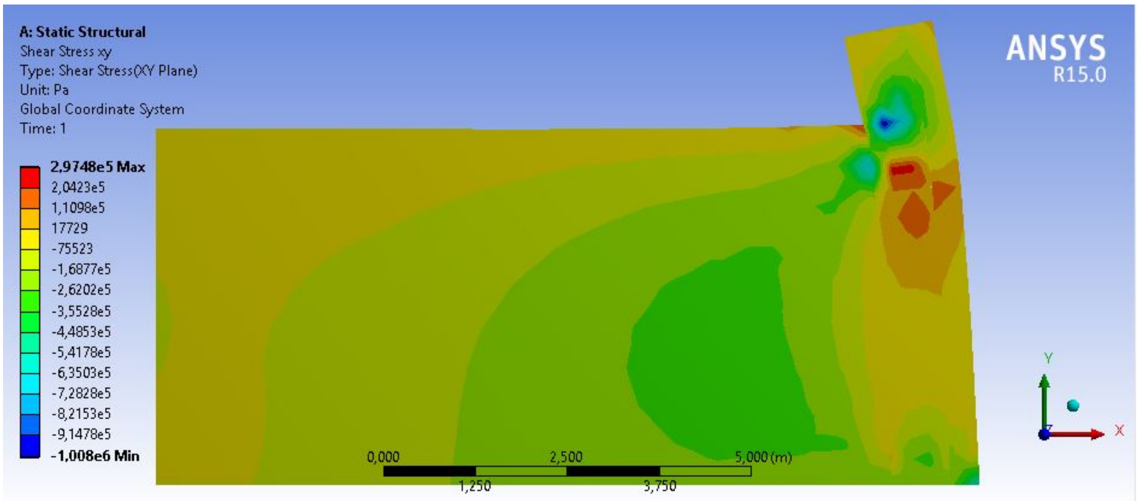

Figure L.20: Shear stresses for the partial replacement alternative for for the ULS condition

L.4 Final design partial replacement alternative

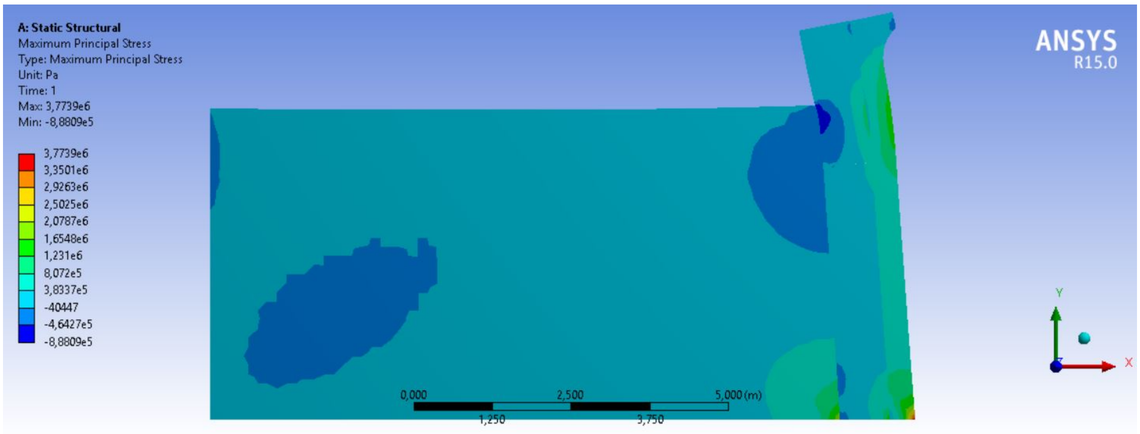

Figure L.21: Maximum principal stresses for a curved wall for section 3

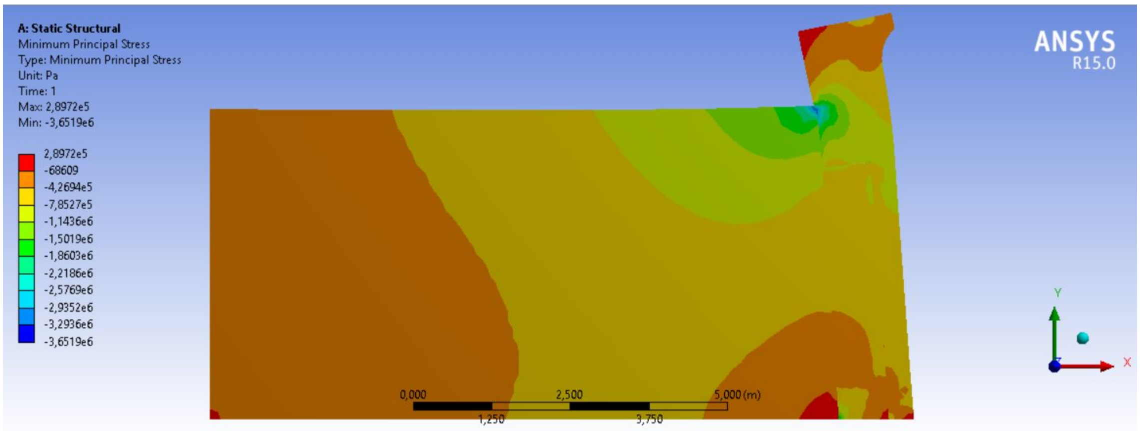

Figure L.22: Minimum principal stresses for a curved wall for section 3

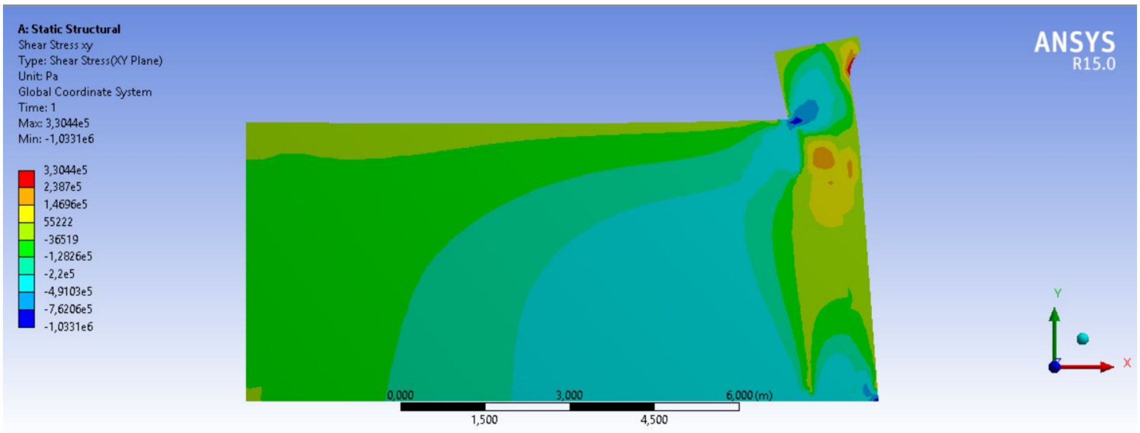

Figure L.23: Shear stresses for a curved wall for section 3

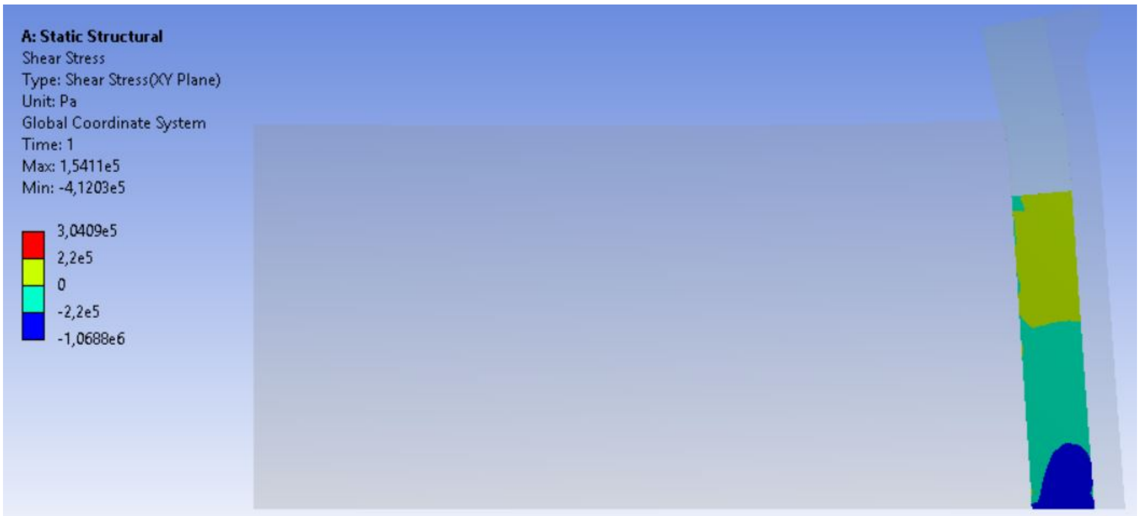

Figure L.24: Shear stresses in the old wall for a curved wall design for section 3

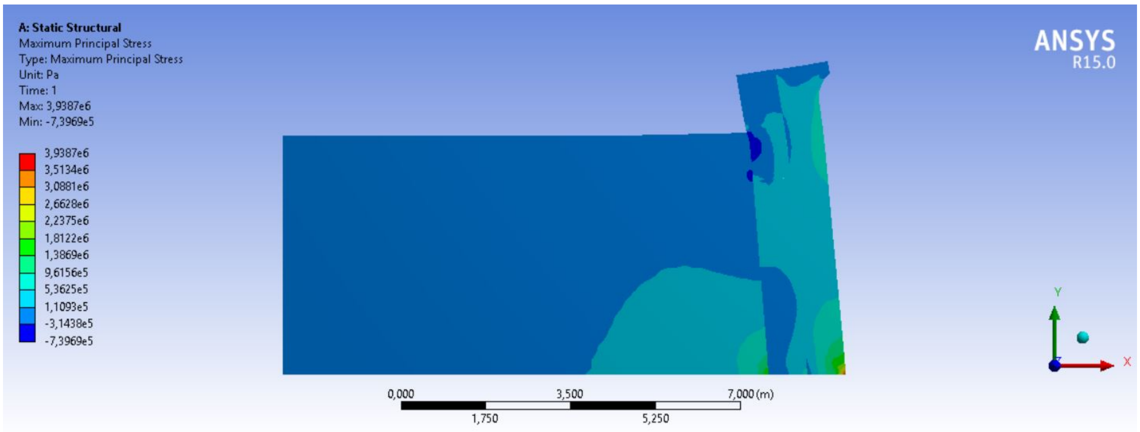

Figure L.25: Maximum principal stresses for a curved wall for section 3 with a bottom thickness of 0.80 m

Appendix M

SWASH model

This appendix gives background information of the SWASH model, a model used in this project to simulate wave propagation behind a breakwater.

M.1 Model description

In this section the SWASH model is described, which is based on the SWASH User Manual [SWASH team, 2015]. SWASH (Simulating WAves till SHore) is a phase-resolving wave propagation model, which makes it suitable for rapidly changing conditions within the scale of the wave length. It is based on the non-linear shallow water equations with a non-hydrostatic pressure model.

M.1.1 Numerical approach

The model is based on an explicit, second order finite difference method for staggered grids. Time integration of the continuity and momentum equations is done with the second order leapfrog scheme. Time discretisation can be done explicitly (while meeting the CFL condition) or semi-implicitly (using the θ-method). The physical domain is divided by a grid. In the horizontal direction the grid can be rectilinear or orthogonal curvilinear boundary-fitted. Also the choice between Cartesian and spherical coordinates can be made. In the vertical direction the computational domain is divided into a fixed, chosen number of layers.

M.1.2 Boundary conditions

Conditions of various types can be imposed on the boundaries. This can be a velocity, water level, discharge, Riemann, Sommerfeld or a zero water level gradient condition. The velocity, water level, and discharge can be given as constants or can vary over time. The Riemann and Sommerfeld conditions are weakly reflective boundary conditions.

M.2 Model input

In figure M.1 the input file for the numerical SWASH model is shown.

```
!*********************      HEADING    ******************************************
PROJECT 'Malecon' '1'

!*********************    MODEL INPUT ******************************************
MODE NONSTationary ONED
CGRID REG 0. 0. 0. 450. 0. 225 0
VERTICAL 2

!~~~~~~~~ Input grids and data ~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~
INPgrid BOTTOM REG 0. 0. 0. 225 0 2. 0.
READINP BOTTOM 1. 'modelinput\bottom\sec3_188_mx2.bot' 1 0 FREE

INPGRID POROSITY   REG   0.  0.  0.  225  0  2.  0.
INPGRID PSIZE      REG   0.  0.  0.  225  0  2.  0.
INPGRID HSTRUCTURE REG   0.  0.  0.  225  0  2.  0.
READINP POROSITY   1.    'modelinput\structure\brwhigh_and_wall.por'    1 0 FREE
READINP PSIZE      1.    'modelinput\structure\brwhigh_and_wall.psiz'   1 0 FREE
READINP HSTRUCTURE 1.    'modelinput\structure\brwhigh_and_wall.hstruc' 1 0 FREE

!~~~~~~~~ Physics and numerics ~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~
FRICTION
BREAKING

NONHYDR
DISCRET UPWIND MOMENTUM
TIMEI 0.1 0.5
BOTCEL MIN

!~~~~~~~~ Initial and boundary conditions ~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~
BOUND SHAPESPEC JONSWAP
BOUNDCOND SIDE EAST CCW BTYPE WEAK ADDB CON SPECTRUM 6. 12. 0. 0. 50. MIN
BOUNDCOND SIDE WEST CCW BTYPE SOMM

!*******************    OUTPUT REQUESTS ****************************************
QUANTITY HSIG SETUP dur 50 MIN
CURVE 'bott' 0. 0. 225 450. 0.
TABLE 'bott' NOHEAD 'output\test_brwhigh_wall_gridbott.mat' XP BOTLEV HSIG SETUP

COMPUTE 20160101.000000 0.1 sec 20160101.011000
STOP
```

Figure M.1: SWASH input file

Appendix N

Cross-sections

In this appendix all cross-sections of the different sections along the Malecón are given. The following notes apply to all the cross-section:

- All dimensions are in mm.
- Levels are in meters relative to Mean Sea Level.
- Storm Still Water Level is indicated at MSL +1.88 m.

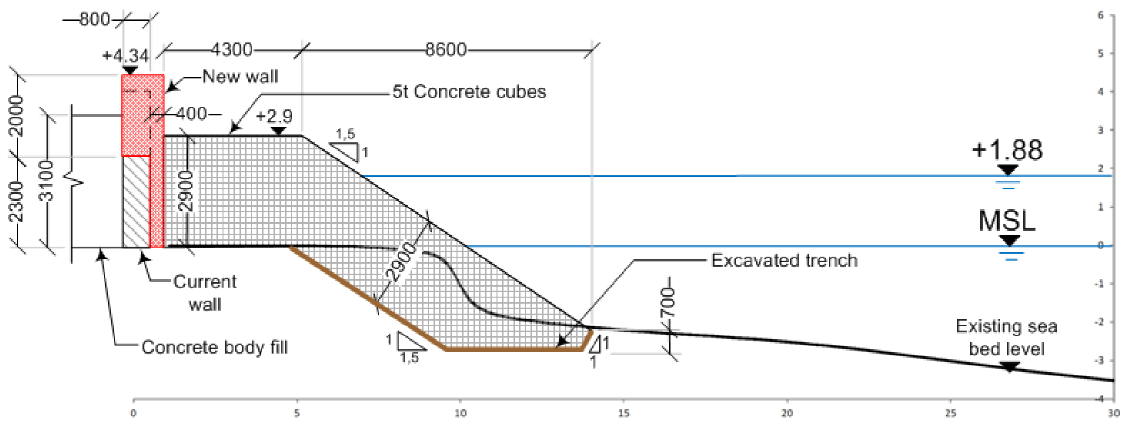

Figure N.1: Cross-section section 1

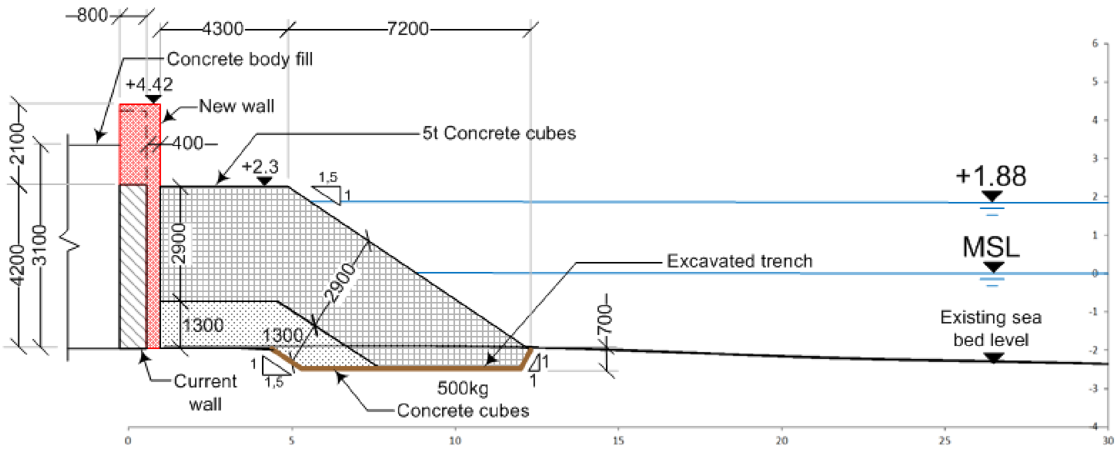

Figure N.2: Cross-section section 2

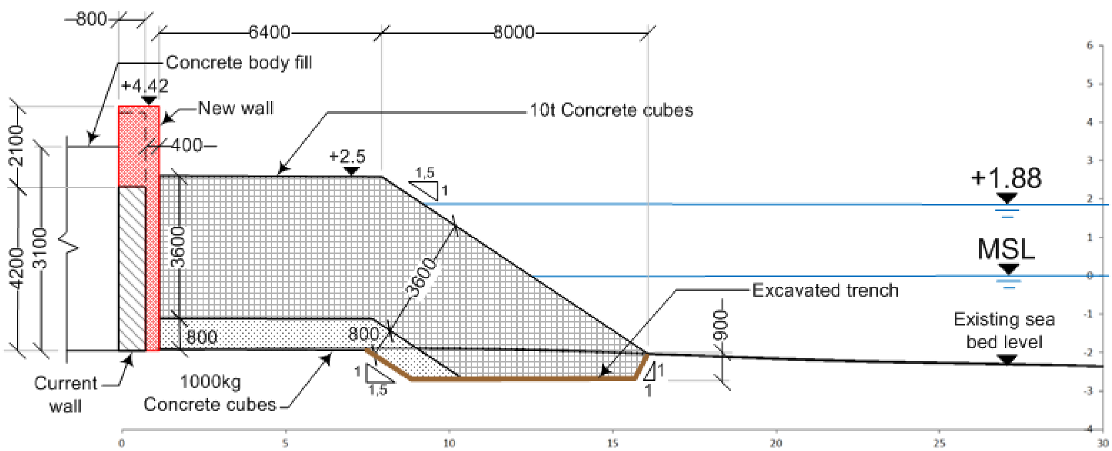

Figure N.3: Cross-section section 2*

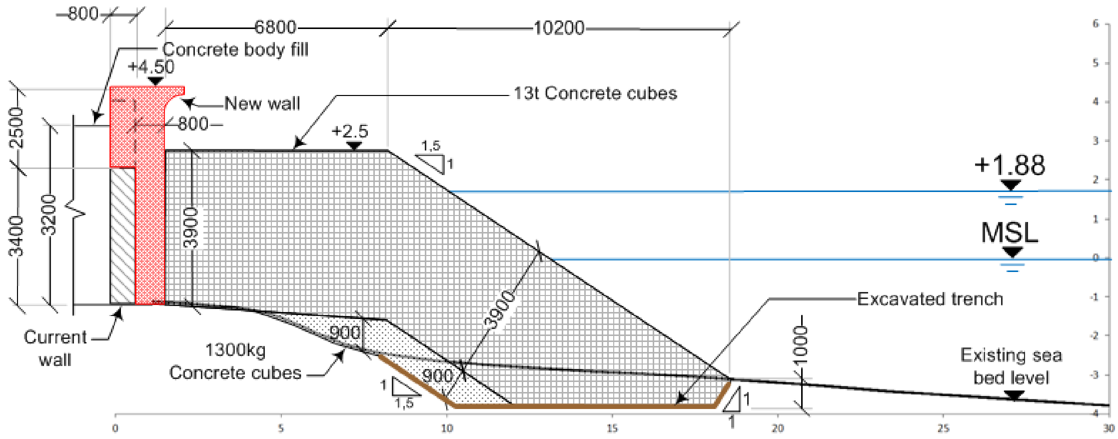

Figure N.4: Cross-section section 3

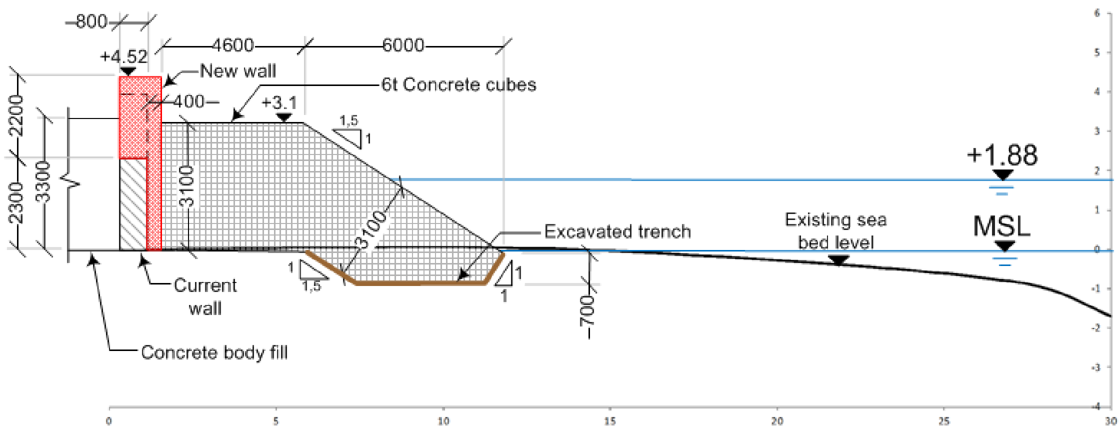

Figure N.5: Cross-section section 4

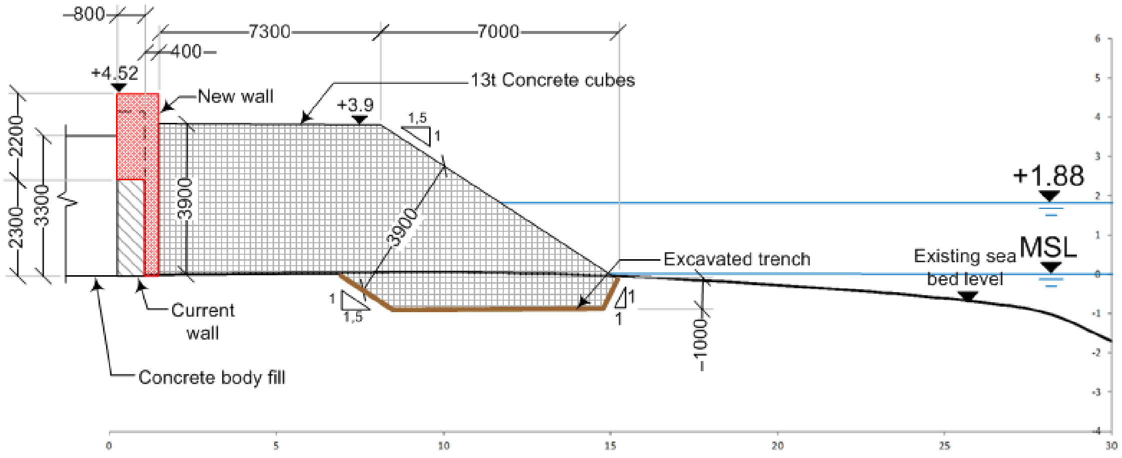

Figure N.6: Cross-section section 4*

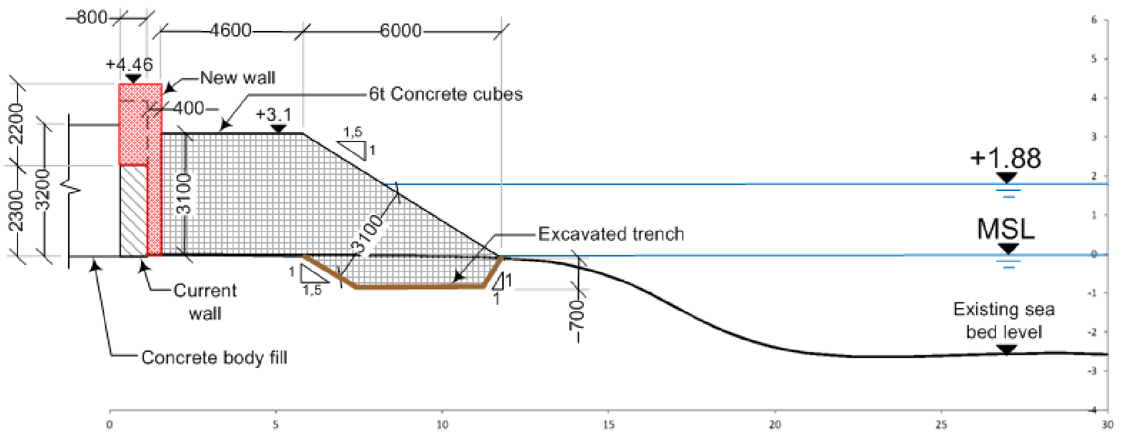

Figure N.7: Cross-section section 5

Appendix O

Breakwater design

This appendix specifies details of the design of a breakwater as solution to the overtopping problems. The detailed design of the breakwater continues on the initial design of the low-crested emerged breakwater described in section 7.5. This is done because the low-crested emerged breakwater is the most ideal option of the three breakwaters described. It has a relatively low crest and needs relatively less material. There are however some changes in the method of designing. In the initial design equation 7.2 is used, while the detailed design is based on equation O.1. The latter is meant to be used for emerged breakwaters, while the first is only valid for rubble mound breakwaters, so both are not ideal. The sizes of the armour units in found with the method used in the initial design are very large, therefore the detailed design is made with equation O.1, which also has the advantage that it can be used to determine the crest and the rear side armour.

The breakwater has an armour layer that consists of two layers of concrete cubes, an 1:1.5 slope and a crest width based on necessary transmission. The breakwater design in section O.1 is based on information from section 5.2 of the Rock Manual [CIRIA, CUR, CETMEF, 2007]. Section O.2 describes the construction method of the breakwater.

O.1 Dimensions

For low-crested structures part of the wave energy can pass over the breakwater due to wave overtopping. This increases stability on the seaward slope, but requires extra stability for the rear side. Therefore the required armour size on the front slope is smaller and on the rear slope larger units should be used, than for structures with only minor overtopping. Moreover the structure is not allowed to reshape (as would be the case for berm breakwaters). Therefore the stability of the rear slope is not directly affected by the stability of the front slope. With equation O.1 the stability of the concrete armour layer can be determined as a function of the relative crest height based on the ratio $\frac{R_c}{D_n}$. As the breakwater crest is designed at still water level, this ratio will equal zero and the right hand side of equation O.1 reduces to the coefficient A, that has different values depending on the segment of the breakwater.

$$\frac{H_s}{\Delta D_n} = A + B\frac{R_c}{D_n} + C\left(\frac{R_c}{D_n}\right)^2 \tag{O.1}$$

Front side armour

For the front slope, the coefficient A equals 1.831. For section 2*, this leads to front slope armour units that are 2.63 m in size, a mass of 4 t and a layer thickness of 4.71 m. For section 3, this leads to front slope armour units that are 2.18 m in size, a mass of 24.5 t and a layer thickness of 4.82 m. The cubes will be randomly placed, as this configuration gives the best reduction of overtopping and reflection.

Crest

For the crest, the coefficient A equals 1.652. In section 2* this leads to a crest armour size of 2.63 m, a mass of 42.9 t per unit and layer thickness of 5.81 m. In section 3 this leads to a crest armour size 2.57 m, a mass of 40.0 t per unit and a layer thickness of 5.68 m. The crest width is usually determined by the construction method or functional requirements. The minimum crest width is three to four times the armour size. In order to achieve the transmission as computed in section 7.5.3. The minimum crest width in order to obtain the allowed transmission is 39 m in section 2* and 31 m in section 3. Considering the size of the armour units, the crest widths of section 2* and 3 become 40.8 m and 31.4 m respectively.

Rear side armour

For the rear slope, the coefficient A equals 2.575. In section 2* this leads to a rear slope armour size of 1.87 m, a mass of 15.4 t per unit and a layer thickness of 4.13 m. In section 3 this leads to a rear slope armour size of 1.83 m, a mass of 14.4 t per unit and a layer thickness of 4.04 m.

Toe

The toe stability is essential for the whole armour layer stability. The sloping rock foreshore (with a smooth surface) provides limited sliding resistance for the toe. Therefore the toe support is achieved by excavating a trench. Hence, there is no offset. The same armour units will be used as for the front slope, no reduction in armour size is possible because of the limited depth.

Underlayer

The ratio between the mass of underlayer units and armour units should be between $\frac{1}{15}$ and $\frac{1}{10}$, as recommended by the Shore protection manual [, CERC]. A ratio of $\frac{1}{10}$ gives an underlayer unit mass of 4.3 t in section 2* and 4.0 t in section 3. For section 2* this corresponds to a concrete element size of 1.22 m for the underlayer and a layer thickness of 2.70 m For section 3 this corresponds to a concrete element size of 1.19 m for the underlayer and a layer thickness of 2.64 m.

Core

The core material should have a mass ratio of $\frac{1}{25}$ to $\frac{1}{10}$ with respect to the underlayer.

The core may consist of quarried rock. The mass of core material necessary is 286 kg for section 2* and 267 kg for section 3, so a stone grading of 60-300 kg is used for both sections.

Breakwater roundheads Wave breaking over breakwater roundheads can cause large velocities and wave forces. For a specific wave direction a limited area (around still water level) of the roundhead is exposed to high wave attack. Two options are available to obtain the same stability as for the trunk section: increasing the mass of the armourstone or making a less steep side slope at the roundhead. No specific rules are available for the breakwater head. Nevertheless, in practice the armour units at the head section often have an increase in size by a factor 1 to 1.3 (or between 1 and 4 with respect to the mass). The Shore protection manual [, CERC] recommends K_D values to be applied in the Hudson formula. At a roundhead with randomly placed armour units on a $\frac{1}{1.5}$ slope and breaking waves, it gives a K_D value of 1.9. This results in an armour size of 3.40 m in section 2* and 3.32 m in section 3 for the roundheads, which is larger than the armour size on the front slope by a factor 1.3 for both sections.

O.2 Construction method

The material is stored in the marina of Havana, very close to the project area. The casting of cubes is also done here. From here, the material is transported to the project area using ships.

The breakwater is detached, so using land based equipment would require a temporary construction towards shore. Therefore marine equipment is used, since the water depths are sufficiently large for this type of equipment. Since specialised equipment is not readily available, flat-top barges are used with an excavator to place the core material of quarry run and the underlayer of concrete cubes. A floating crane is used to place the armour layer of concrete cubes.

Appendix P

Cost estimation in Cuba

General information about construction costs in Cuba is achieved from the Cuban ministry of Construction [Dirección de Presupuestos y Precios del Ministerio de la Construcción, 2005]. The costs are classified into two categories: primary costs and secondary costs. The primary costs are specified in section P.0.1. These costs are in direct relation to the construction. The secondary costs have an indirect relation and are specified in section P.0.2.

P.0.1 Primary costs

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.

Direct costs of work by hand (C2)

- Design.
- Technical preparations (office, calculations, communication).
- Wages.
- Water (not used for concrete).

Direct costs of equipment (C3)

- Fuel, lubricant, oil, electricity.
- Wages for the permanent operators of the material.
- Reparation and maintenance of the material.
- Security for the material.
- Tires.
- Interest of the use of capital.
- Taxes.

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)

- 29% of C5

Total costs (C7)

- 3% of C1+C2+C3

Profit (C8)

- 20% of the preparation costs

Total primary costs (C9)

- C7+C8

P.0.2 Secondary costs

Temporary facilities (P1)

- Toilets, material warehouses, etc.

Transport (P2) Other additional costs (P3)

- Proof of good quality of the used materials for the client.
- Transport of not used materials at the finish of the work.

Banking (P4)

- Risk of cost estimation.
- Risk of time estimation.
- Risk of price-changes during the project.

Security (P5)
Unpredictable costs (P6)
Total secondary costs (P7)

- P1+P2+P3+P4+P5+P6

P.0.3 Construction costs Malecón seawall

Figure P.1 shows a rough calculation that is made for the costs of the new seawall. For the material costs the value of $ 450 was used. This consists of $ 230 material costs for the concrete material only and the rest of the unit price is determined for reinforcement, dowels and formwork. The total volume of reinforced concrete needed has been evaluated using the average cross section needed per section.

P.0.4 Construction costs revetment

Figure P.2 shows a rough calculation that is made for the costs of the revetment. Where possible unit prices from PRECONS were used in the calculation. For the rest of the costs estimations were made, based on a unit price of approximately 10,000 CUC/m[1] for constructing a revetment (including materials, placement and transport costs).

		quantity	unit price	subtotal	total
	Primary costs				
C1 - Direct costs of material					
	constructing reinforced concrete seawall [m3]	8156	450	3670171	
					$ 3.670.171
C2 - Direct costs of work by hand					
	Design (5 people; 8 weeks) [hrs]	1600	40	64000	
	Technical preparations [hrs]	1000	23	23000	
	Salaries (20 people; 6 months) [hrs]	41600	23	956800	
					$ 1.043.800
C3 - Direct costs of equipment					
	Fuel, lubricant, oil [gallons]	10000	0,64	6400	
	Security for the material and equipment			5000	
	Moveable crane 10-20 ton [hrs]	1000	24,28	24280	
					$ 35.680
C4 - Direct costs of means of support and small material				$ 142.490	
					$ 142.490
C5 - Total direct costs					$ 4.892.141
C6 - Indirect costs					
					$ 1.418.721
C7 - Total costs					$ 6.310.862
C8 - Profit					$ 526.706
C9 - Total primary costs					**$ 6.837.568**
	Secondary costs				
		quantity	unit price	subtotal	total
P1 - Temporary facilities					
	Toilets, material warehouses, etc.			3000	
					$ 3.000
P2 - Transport					
	Trucks 4,6 - 6,0 (32 trucks) [m3]	24 months		40000	
	barge for storing material (6 months)			24000	
	Concrete truck (5trucks) [hrs]	5505,9595	24,28	133684,6973	
					$ 197.685
P3 - Other additional costs					
	Proof of good quality			2000	
	Transport of unused materials (5% of P2)			$ 9.884,23	
	Closing the Malecón road				
					$ 11.884
P4 - Banking					
	Risk of cost estimation			200000	
	Risk of time estimation			100000	
	Risk of price-changes during the project			100000	
					$ 400.000
P5 - Security				100000	
					$ 100.000
P6 - Unpredictable costs				$ 75.501,37	
					$ 75.501
P7 - Total secondary costs					**$ 788.070**
Total costs of the construction					**$ 7.625.638**

Figure P.1: An estimation of the costs for the new seawall

Primary costs	quantity	unit price	subtotal	total
C1 - Direct costs of material				
concrete prefab [m3]	20472,08	200	4094415,195	
concrete in situ [m3]	204,7208	230	47085,77474	
additional material costs			3058499,03	
				$ 7.200.000,00
C2 - Direct costs of work by hand				
Design (5 people; 8 weeks) [hrs]	3200	40	128000	
Technical preparations [hrs]	2000	25	50000	
Salaries (40 people; 6 months) [hrs]	41600	25	1040000	
additional labour costs			582000	
				$ 1.800.000,00
C3 - Direct costs of equipment				
Moveable crane 10-20 ton [hrs]	1000	24,28	24280	
Excavator [hrs]	2400	34,33	82392	
specialist rock breaker to excavate toe in bedrock [?]			1500000	
Additional equipment costs			193328	
				$ 1.800.000,00
C4 - Direct costs of means of support and small material			324000	
				$ 324.000,00
C5 - Total direct costs				$ 11.124.000,00
C6 - Indirect costs				$ 3.225.960,00
C7 - Total costs				$ 14.349.960,00
C8 - Profit				$ 1.254.996,00
C9 - Total primary costs				**$ 15.604.956,00**

Secondary costs	quantity	unit price	subtotal	total
P1 - Temporary facilities				
Toilets, material warehouses, etc.			3000	
				$ 3.000,00
P2 - Transport				
Dumptruck[hrs]	30000	23,56	706800	
Concrete truck [hrs]	150	24,28	3642	
				$ 720.000,00
P3 - Other additional costs				
Proof of good quality			2000	
Transport of unused materials (5% of P2)			21811,848	
				$ 23.811,85
P4 - Banking				
Risk of cost estimation			200000	
Risk of time estimation			100000	
Risk of price-changes during the project			100000	
Risk of damage to the seawall			100000	
				$ 500.000,00
P5 - Security			100000	
				$ 100.000,00
P6 - Unpredictable costs			$ 169.517,68	
				$ 169.517,68
P7 - Total secondary costs				**$ 1.516.329,53**

Total costs of the construction	**$ 17.121.285,53**

Figure P.2: An estimation of the costs for the revetment