Blau Arenal Hotel Project

Protection against flooding to prevent mangrove forest contamination

S. Bom
R.F. Vorderegger
P.J.P. Koot
E.A.F. Wendt
G.P. van Rinsum

October 2016

Delft University of Technology

TU Delft Delft University of Technology

cujae

Van Oord Marine ingenuity

Challenge the future

Cover image

Aerial picture of Blau Arenal Hotel. [1]

Blau Arenal Hotel Project

Protection against flooding to prevent mangrove forest contamination

Date:	October 25, 2016	
Place:	Havana, Cuba	
Supervisors:	Ir. H.J. Verhagen (TU Delft)	
	Dr. ir. L.F. Córdova López (CUJAE)	
Students:	Sam Bom	4227808
	Paul Koot	4120727
	Guido van Rinsum	4213785
	Ruben Vorderegger	4114469
	Emiel Wendt	4231260

PREFACE

This report is written by five students from the University of Technology Delft, the Netherlands, who have worked on a multidisciplinary project in Havana, Cuba for two months. This project has the aim to analyse the flooding problems in the vulnerable coastal ecosystem located around the Itabo River and the Blau Arenal Hotel. The causes of flooding, the environmental impact for the surroundings, the problems for the hotel and several possible solutions and recommendations are presented in this report. XBeach was used to model the area, the flooding and to test the solutions. Besides that, we broadened our personal experience by working abroad in a country that does not function like the Netherlands. Different regulations, different standards and a different culture made this project both challenging and a great experience.

The study is performed with cooperation of the Hydraulic Investigation Centre (CIH) at the Instituto Superior Politécnico José Antonio Echeverría (CUJAE) in Havana. We want to thank two professors from the CUJAE for their personal support. First of all Dr. ir. L.F. Córdova López, our tutor, who was nearly every day available for our questions and supported us throughout the whole project. Thanks to Prof. dr. N. Marrero de León who was available for questions on the hydrological part of the project. We would also like to thank our supervisor from TU Delft, Ir. H.J. Verhagen for his contribution to the project.

Last but not least, we would especially like to thank our sponsor Van Oord for their financial support to make the project feasible.

Project group 199
S. Bom
P.J.P. Koot
G.P. van Rinsum
R.F. Vorderegger
E.A.F. Wendt

Havana, October 2016

Sponsor

Van Oord
Marine ingenuity

Partner

BMT
ARGOSS

SUMMARY

The main objective of this study is to protect the mangrove forest, which surrounds the Blau Arenal Hotel, from being contaminated due to flooding of the waste water treatment plant (WWTP) and secondly to prevent flooding of the Blau Arenal Hotel. The Blau Arenal Hotel is situated at the Itabo River mouth and Boca Ciega beach. This location makes the hotel vulnerable for both flooding due to high waves and high river discharge. The Blau Arenal Hotel is completely encompassed by the mangrove forest, which was declared as protected area after the hotel was built in 1985. The Ministry of Environment (CITMA) demands the Ministry of Tourism (MoT) to protect the mangrove forest against contamination. If the waste water treatment plant gets flooded, hazardous substances will flow into the protected area. The Ministry of Tourism, with its state enterprise Cubanacan, which owns the Blau Arenal Hotel, can decide whether they want to protect the hotel itself against flooding as well.

Two safety levels are defined for the two flood risk scenarios. The solution is designed for a lifetime of 50 years. The return period of flood protection of the waste water treatment plant is set to one in 225 year, using a probability of failure of 20% during the lifetime. The safety level of the hotel is set at a lower level of protection, that is one in 100 year conditions, using a probability of failure of 40%. The reason for the different safety levels is because of the small probability of life loss given flooding of the hotel, due to evacuation and limited amount of people in the area.

The offshore wave conditions are determined with the peak over threshold method, based on 25 year of storm data. SWAN is used to determine the nearshore wave conditions. The reduction of wave height is large due to an offshore shoal, which causes breaking of high waves. The storm surge level is set to 0.40 meter. The sea level rise during the lifetime of the structure is 0.25 meter. The tidal amplitude is set to 0.25 meter. The river discharge conditions are calculated with the rational method and triangular unit hydrograph method. The precipitation input data for this method is based on the Intensity Duration Frequency curves for a location close to the study area. No correlation is assumed between storms at sea and high precipitation, which results in no simultaneous occurrence of the two events.

Relevant flooding scenarios are determined based on the analysis of the hydraulic conditions. One of the scenarios is for example a river discharge with a return period of one in 225 year combined with normal wave conditions. The study area consisting of the hotel area, river and beach is modelled with XBeach, which is normally used for modelling nearshore processes. Modelling of all the scenarios showed that both the hotel and waste water treatment plant are flooded at the predefined protection level due to high river discharge. The maximum flood levels are shown in table 1. Waves do not result in flooding, according to the model, due to the relatively mild nearshore wave conditions.

Table 1: Flooding levels

	Flood level +MSL [m]	Flood level +lowest ground level [m]
Hotel 1/100 condition	2.75	0.63
WWTP 1/225 condition	2.90	0.49

Three categories of solutions are proposed to solve the flooding problems. The first one is a flood protection wall around the waste water treatment plant and hotel or around the waste water treatment plant only. The second solution is a flood retention basin that stores a part of the river discharge temporarily. The third proposed solution is a dike structure, situated south of the hotel area, that prevents flooding at the main river bank and leads the water around the hotel area. Modelling of all solutions with the governing hydraulic conditions results in the final solution.

A solution that can be chosen if the predefined safety levels are preferred, is a gravity wall around the waste water treatment plant that retains one in 225 year flood conditions and a wall around the hotel that is able to withstand one in 100 year conditions. With this solution, the Ministry of Tourism satisfies CITMA by protecting the mangrove forest and invests in the protection against flooding of the Blau Arenal Hotel. Now, the hotel is also protected against flooding occuring at return periods lower than the one in 100 year condition. However, a wall around both the waste water treatment plant and the hotel is effective as a flood retention, it will have a negative influence on the functionality and aesthetic value of the hotel. Furthermore, the Multi Criteria Analysis (MCA) and the costs of the measures give no high credits to this solution. Based on these considerations another solution is advised.

The final solution follows from the MCA that shows that the most optimal solution is to build a wall around the waste water treatment plant only, with a safety level of one in 225 year. The dimensions of the wall around the waste water treatment plant can be seen in table 2. This solution fulfils the requirements of CITMA with respect to contamination of the mangrove forest. However, in this case, no measures are taken to prevent flooding of the Blau Arenal Hotel. Further research on the Blau Arenal Hotel area is advised to prevent flooding occurring at return periods lower than the one in 100 year condition.

Table 2: Wall solution

	Top of structure +MSL [m]	Max. height of structure [m]
WWTP 1/225 wall	3.70	1.70

The reason for deviating from the predefined safety level of one in 100 year protection of the hotel is due to the flood characteristics. The flood characteristics determine the consequences of flooding. The probability of life loss is low due to the relatively low inundation depth of 68 cm, with respect to the lowest ground level at the hotel, during one in 100 year conditions. Furthermore, the damage due to flooding at the hotel is a function of inundation depth and due to this low inundation depth the amount of damage is limited.

Contents

1

INTRODUCTION

1.1. GENERAL INFORMATION OF CUBA

1.1.1. ANTHROPOLOGY

Cuba, the 'Pearl of the Antilles', was first reached by humans from South America around 2000 BC, which was proved by carbon dating. These Indians still lived on the island when, on October 27 in 1492, Christopher Colombus, also known as Cristóbal Colón in Spanish, arrived in Cuba. Columbus described Cuba as 'the most beautiful land human eyes have ever seen'. For the next four centuries the Spanish dominated the island and used it as a gateway to Spain's vast American empire, where beautiful cities such as Havana, Trinidad and Santiago de Cuba are a few examples of this period of wealth.

From October 10, 1868, Cuba was frequently struggling for independence. Famous freedom fighters and visionaries have originated from this long-lasting fight against oppression, such as Fidel Castro and Ernesto 'Che' Guevara. One of the most famous philosophers and writers was Jose Marti, who already warned of the threat of sporadic US imperialism.

Yo soy un hombre sincero	I'm a sincere man
de donde crece la palma,	from the land of the palm tree,
y antes de morirme	before I die
quiero echar mis versos del alma	I wish to sing these heartfelt verses.
con los pobres de la tierra	with the poor of the land
cuiero yo mi suerte echar,	I want to share a fate,
y el arroyo de la sierra me complace	and the mountain stream pleases me
mas que el mar	more than the sea.

Lines from Versos Sencillos (1891), by Jose Marti. Later incorporated in the most popular Cuban song, Guajira Guantanamera.

The last independence war began on July 26, 1953, led by Fidel Castro and resulted in the establishment of the current republic. The present-day communist republic was founded on the first of January 1959. As a communist country it had a strong bond with the former Soviet Union. With the fall of the Soviet Union in 1989, Cuba fell into an economic depression. Recently the island is slowly climbing out of its poverty, due to the recent improved relations with the United States of America, who created an embargo in the last century.

1.1.2. GEOGRAPHY

Caught like a cigar between the fingers of Florida and Yucatan, Cuba lies in the heart of the American Mediterranean, just south of the Tropic of Cancer, at the mouth of the Gulf of Mexico (see figure 1.1). The Atlantic Ocean is located north of the island and the Caribbean south of the island. With 110,922 km^2, Cuba has almost exactly the same area as Bulgaria and is the largest island in the Caribbean. In total Cuba has 289 natural beaches [2]. Beaches of the northern side tend to be longer and whiter with rolling surf waves, while the southern coast is rockier and swampier with darker beaches and sea urchins.

Figure 1.1: Map of the American Mediterranean, with Cuba highlighted [3]

The 7200 meter deep Cayman Trench between Cuba and Jamaica, south of Cuba, forms the boundary of the North American and Caribbean plates. The collision of these plates occasionally generates earthquakes affecting Cuba. These same tectonic movements have tilted the island, leading to uplifted limestone cliffs along parts of the north coast and low mangrove swamps in the South. While largely flat, the island has four major mountain ranges. The highest peak is named Pico Turquino, with an altitude of 1,974 meters.

Cuba does not have large lakes or rivers, but the Cubans have built a lot of dams that house large freshwater reservoirs for irrigation and water supply. Cuba's largest river is the 343 kilometer long Cauto, located north of Bayamo. This river is only navigable by small boats. Submerged old river valleys have created excellent harbours, such as those of Cienfuegos, Santiago de Cuba and Guantanamo Bay.

1.1.3. CLIMATE

Cuba has a subtropical climate and two clearly defined seasons: the wet season from May till October and the dry season between November and April. The average temperature is approximately 25.1°C, with the lowest temperatures in January (21.9°C) and the highest in August (27.7°C). The average annual rainfall is 1207 mm, of which 65% falls in the wet season. The relative humidity in Cuba has an annual average value of 73%. All values are from an article by the Meteorological Institute of Cuba [4].

Due to its geographical situation and influences from the marine climate, Cuba suffers from extreme weather conditions. Heavy rainfall, strong winds, inundations at the coast, destructive storms and tropical cyclones happen every year. During the hurricane season, from September until November, multiple hurricanes passed Cuba in the past. Cold fronts mostly appear between November and April, also known as the storm season. When the cold fronts come from the Gulf of Mexico, high waves can be created.

1.2. STUDY AREA

The area of interest of the research project is located in the province of Havana, which lies in the north-western part of Cuba. The capital of Cuba, Havana, is only 30 km away from the project area (the red circle), as can be seen in figure 1.2. The beaches, which are located in this region, are called Playas del Este. This literally means 'Eastern Beaches', which refers to Havana as these beaches are located east of the capital. Unlike the famous beaches of Varadero that are about 120 km east of Havana, the Playas del Este are mostly visited by the local Cubans. The area of interest of the research project is located around a large resort called the Blau Arenal Hotel.

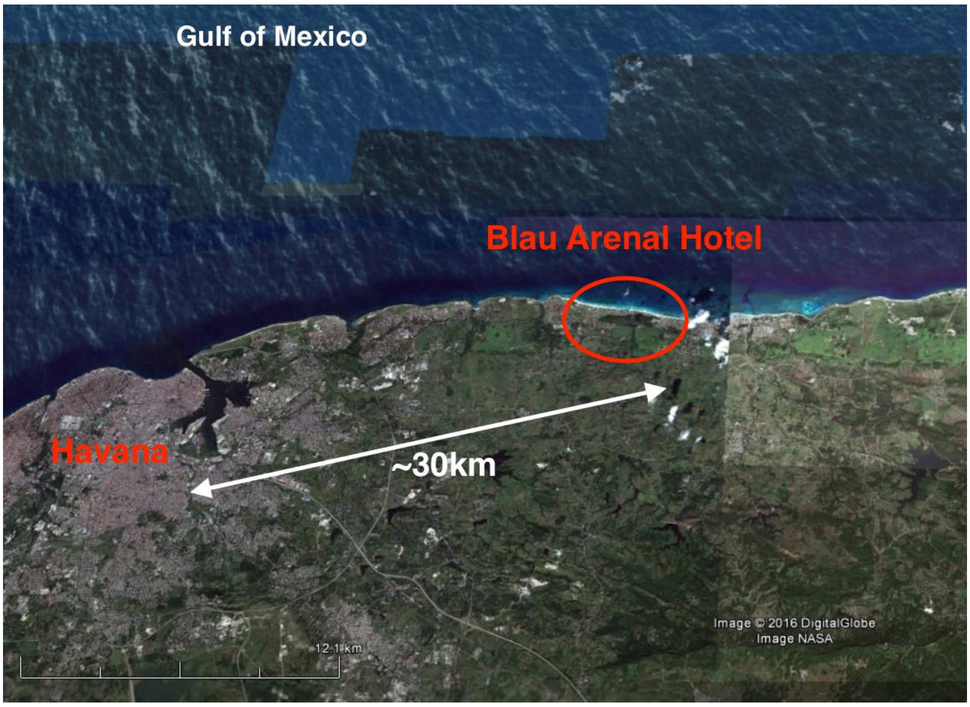


Figure 1.2: Location of the study area near Havana, satellite image from 16-11-2011 [5]

1.3. PROBLEM DESCRIPTION

1.3.1. PROBLEM DEFINITION

The study area consists of Boca Ciega beach and the lagoon behind the dunes, where the Blau Arenal Hotel is located. This area has had flooding issues, especially during a subtropical storm in 1982 that flooded the hotel and surrounding area [6]. For this specific storm the waves were not the main reason for flooding, but the rain coming with this depression caused the hotel to be flooded. Waves play an important role in the development of this river mouth, but they could initiate flooding on their own too. Especially since the coast of Boca Ciega beach is eroding nowadays [7].

Since 2008, the whole study area has been declared to be a protected environmental region [8]. This is because of the presence of the unique ecosystem of mangrove forest, which completely encompasses the Blau Arenal Hotel. The protected state of the region provides another issue due to the fact that the hotel has its own waste water treatment plant. If this plant gets flooded, hazardous substances will flow into the protected area. For this reason, the Ministry of Environment (CITMA) demanded the Ministry of Tourism to prevent flooding that can cause environmental contamination. As no previous solutions have been developed to protect the Blau Arenal Hotel against flooding and waste water contamination, research needs to be done to find out if and how this problem can be solved for several scenarios. Important factors are the wave conditions, the river discharge and the state of the river mouth.

1.3.2. OBJECTIVE

The objective of this project is to find out under which circumstances flooding and contamination by the waste water occur in the study area, consisting of Boca Ciega beach, the Blau Arenal Hotel and the protected ecosystem of the mangrove forest, and to come up with a suitable solution to prevent contamination of the mangrove forest. Furthermore, the consequences of flooding at the Blau Arenal Hotel are analysed.

1.3.3. STRUCTURE OF THE REPORT

This report begins with an extensive analysis in chapter 2. The area and the stakeholders are analysed. Then the boundary conditions are elaborated. From this analysis several scenarios are deduced and a statistical approach is applied to deal with the uncertainty in these scenarios. In chapter 3 these scenarios are put into SWAN and XBeach. The model input and setup are explained, and the results of the models are shown. In chapter 4 the results of the model runs are translated to possible solutions that can withstand the conditions. The solutions are elaborated in this chapter. Consequently these solutions are put into XBeach and simulated, to see if the solution works. This is done in chapter 5. As a result of the simulation process, combinations of solutions are found. These are also modelled. Chapter 6 elaborates on the solutions that succeeded to prevent flooding during the simulation. A Multi Criteria Analysis and an inventarisation of the costs are used to choose the final solution. This final solution is elaborated in chapter 7. Several problems that come with this solution are treated and solved. Also a construction plan is included. Next, a discussion on several choices made throughout the report is included in chapter 8. Recommendations are given on the most relevant parts of the project that determine the outcome of the calculations in chapter 9.

2

Analysis

2.1. Area

As already treated in the introduction, the area of interest is the Blau Arenal Hotel, located at the Playas del Este, 30 km east of Havana. A more zoomed and detailed overview of the exact location of the study area (the red circle in figure 1.2) is shown in figure 2.1.

Figure 2.1: Overview of the study area, satellite image from 19-12-2014 [5]

The red lines in figure 2.1 are the land boundaries of the project. Each corner of this rectangular shaped domain has a number associated with it. These numbers correspond to the geographical locations, starting at the upper left corner with number 1 (see table 2.1).

Table 2.1: Latitudes and longitudes of each corner of the study area

Corner	Latitude	Longitude
1	23° 10' 32,51" N	82° 10' 32,40" E
2	23° 10' 18,83" N	82° 9' 20,54" E
3	23° 9' 55,72" N	82° 9' 21,57" E
4	23° 10' 1,83" N	82° 10' 33,95" E

A large road in the South bounds the area. This is the main road along the north coast of Cuba connecting Havana in the West and Matanzas in the East. The road is called Via Blanca, which translates to 'White Road' and refers to the white beaches parallel to its direction. In the North, the area is bounded by a beach called Boca Ciega, which means 'Blind Mouth'. The name originates from the fact that the river mouth is covered with vegetation. Boca Ciega beach is part of the previously named and larger Playas del Este. The boundaries in the West and East are chosen in such a way that all locations of interest, like the lagoon, a large section of the beach and the river are inside the study area.

2.1.1. GEOGRAPHY

The geography of the study area can be divided into three sections, which are all further elaborated here. The sections are visualised in figure 2.2.

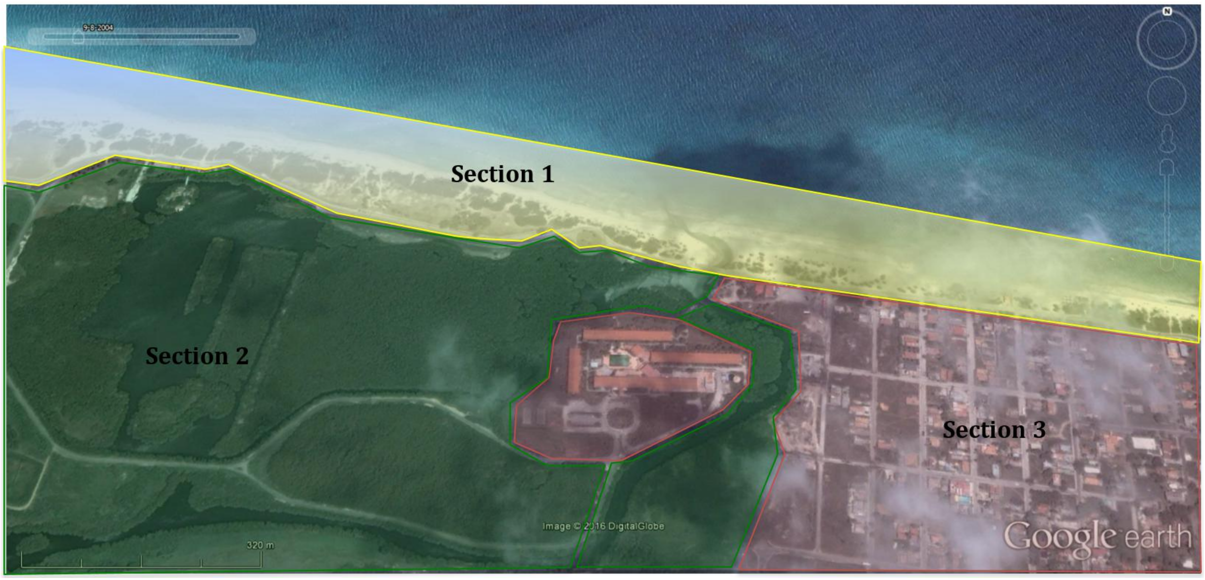


Figure 2.2: Overview of the three major sections of landscape, satellite image from 09-08-2004 [5]

SECTION 1

Section 1 is the coastal zone, where the beach and dunes are located. The sand characteristics can be found in section 2.4.1. The beach, which is about 1200 m long, gets wider in the summer (wet season). The exact opposite happens in the winter (dry season). Both these effects happen due to the different wave actions in summer and winter. In response to the milder wave conditions during summer, the offshore sandbars of the 'winter profile' move onshore and finally attach to the shore and rebuild the wider berm associated with the 'summer profile'. As already mentioned, the opposite happens in the winter, due to the more severe wave conditions [9]. The dunes, adjacent to the beach, are only around 15 m wide. As can be seen in appendix A, at the seaside a large scarp is visible. This scarp is formed, because the sea is slowly eroding the beach and dunes. In appendix B the movement of the dune can be seen, as it slowly moves land inward and at other sections of the beach erosion is clearly visible. The Itabo River cuts right through the middle of the dunes and beach. At this location a bridge used to cross the mouth of the river, but nowadays only the foundation is visible. This change can be seen in appendix B. The river mouth is unstable and closes during some weather conditions, which are further elaborated in section 2.5.

SECTION 2

Section 2 consists of the mangrove forest and lagoon. The mangrove forest completely circumferences the Blau Arenal Hotel. In appendix A it can be seen that the mangrove forest is very dense. A mangrove forest thrives in an intertidal region, where salt water can intrude the soil. Although it cannot be seen in the aerial pictures, the soil of this entire section, which consists of clay, is mostly submerged. In the western part of the area a lagoon is located. This lagoon receives its water from the river Itabo. Both the lagoon and mangrove forest give home to a unique ecosystem, which is very vulnerable to changes.

SECTION 3

Section 3 can be described as a rural zone. The Blau Arenal Hotel and a few houses in the East are part of this section. The surface exists of a combination of buildings, grassland and roads, which are constructed with asphalt.

The most important feature of the landscape of the study area is the Itabo River. This river starts 10 km land inward and flows through the middle of the area of interest exiting into the Gulf of Mexico. The river is the source for the lagoon and the mangrove forest. A detailed analysis of the Itabo River can be found in section 2.6.

2.1.2. BLAU ARENAL HOTEL

The hotel is located in the centre of the project area and is the main cause of the problems in the study area. Before the hotel was opened, the area was completely covered with mangrove forest with small lagoons in between. The Itabo River did not really have one path, like nowadays, but spread its water over the entire region. For the construction of the hotel, a large part of the mangrove forest had to be removed and the land was reclaimed. The reclamation was necessary because, as already mentioned, mangrove forests are submerged most of the time. After the reclamation and construction of the buildings, the Blau Arenal Hotel opened its doors in 1985 (see figure 2.3). In appendix A a detailed description of the hotel and its surroundings is elaborated.

Figure 2.3: Aerial overview of the Blau Arenal Hotel, satellite image from 10-10-2002 [5]

The hotel can accommodate around 300 tourists. This will only be reached during high season. Winter is the high season for the hotel, because it is the dry season. As the temperature does not drop below 20°C, the season is perfect for beach tourists.

In 2008 the hotel was closed due to problems with the waste water treatment plant. This lasted until 2012, and is further described in appendix C. Since then the hotel has been open. The renovation of the hotel in that period costed around 380.000 US dollars. In appendix B a change in the hotel can be seen, especially in the construction of the new waste water treatment plant.

2.1.3. WASTE WATER TREATMENT PLANT

In order to ensure that contaminants do not adversely impact the surface water by direct discharge of waste water, typical requirements for waste water discharge are made. Waste water should be treated prior to discharge and there are limits for the amount of contaminants that are allowed in the effluent after treatment [10]. Because of its isolation and relative high utilisation, the hotel has its own waste water treatment plant. This waste water treatment plant has to treat the waste water to meet the effluent requirements. The surrounding nature is part of the reason why tourists will visit the area, so the hotel has its own stake in treating the waste water well. In Cuba, the effluent is primarily measured for BOD (Biological Oxygen Demand) and the E. coli bacteria. Either one has to have a value below a certain threshold before the water can be discharged in the surrounding area. In the this section, research is done to the way the hotel has treated its waste water before and how it still treats its waste water.

HISTORY

Before 2008 the Blau Arenal Hotel used a different waste water treatment plant. From personal accounts during the site visit, a global overview of how the plant functioned in the past is described below.

The waste water was collected from the entire hotel in a large basin. In this basin, the water naturally degraded its organic material. This is done by the use of oxygen from the air. With enough retention time inside the basin, the organic material will diminish naturally below its limit [11]. After this basin the treated water was thought of as being clean enough to be returned to nature. However, when the head of the water department of the province of Havana visited the old waste water treatment plant, he thought otherwise. Due to bad maintenance, the pipes entering and leaving the basin were broken, resulting in the discharge of untreated waste water to the surrounding area. This led to the immediate closure of the hotel. As soon as the problem was solved, the hotel was allowed to reopen.

PRESENT-DAY

In 2014 the Blau Arenal Hotel had built a new waste water treatment plant. This plant is more modern and robust than the previous one. As can be seen in figure 2.4, the location of the plant (inside the blue circle) is in the eastern side of the site and next to the Itabo River.

Figure 2.4: Aerial overview of the Blau Arenal Hotel and location of the new waste water treatment plant (blue circle), satellite image from 29-06-2015 [5]

The treated water is cleaner and meets the effluent requirements to discharge in the surrounding protected region. Figure 2.5 shows an overview of the complete water system including the waste water treatment plant and the freshwater treatment plant, working with reverse osmosis. In appendix C a detailed explanation of the plants is given. Although these plants have been constructed well, a high contamination risk still remains when flooding occurs. In appendix C, the exact sources of these risks have been analysed and described.

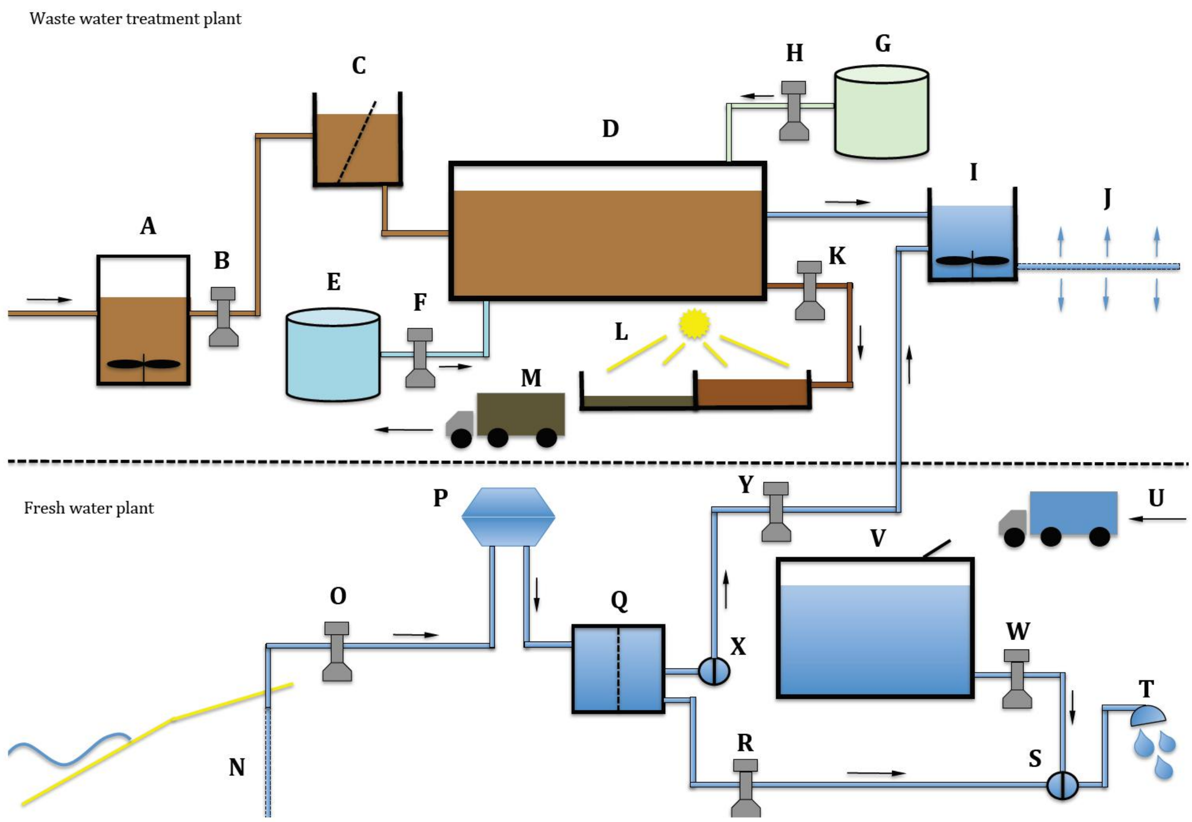


Figure 2.5: Visualisation of the waste water and freshwater treatment plant

A – Receiving waste water and mixing
B – Pump
C – Sieve (filter)
D – Active sludge tank
E – Oxygen tank
F – Pump
G – Chlorine tank
H – Pump
I – Mixing tank and treated waste water reservoir
J – French drainage for filtration of the treated waste water into the soil and river
K – Pump
L – Basin for sludge to dry in the sun
M – Dried sludge is transported away for use as fertilizer
N – Pipes close to the sea to pump up seawater
O – Pump
P – Water tower (salt water reservoir)
Q – Desalination tank, which uses reverse osmosis
R – Pump
S – Valve to select the source of freshwater supply
T – Use of freshwater (Blau Arenal Hotel)
U – Supply of freshwater by trucks
V – Reservoir for freshwater
W – Pump
X – Valve to empty the desalination tank after cleaning
Y – Pump

2.2. STAKEHOLDERS

This project is relevant for several stakeholders. In this section the most relevant stakeholders that show interest in this project are elaborated. The aim of this stakeholder analysis is to give a description of each stakeholder and to explain their involvement in the project.

2.2.1. ACTORS

The stakeholders are described here shortly. For the entire elaboration on each of the stakeholders is referred to appendix D.

GOVERNMENT OF CUBA

The government of Cuba is divided into separate ministries. The ministries relevant for this project are the Ministry of Science, Technology and Environment (CITMA), the Ministry of Tourism (MoT), the Ministry of Higher Eductation (MoHE) and the Ministry of Construction (MoC). The provincial representative offices of the ministries in Havana are responsible for the project of Blau Arenal Hotel, located in Playas del Este. They work on behalf of the ministry but on a regional level.

Also, the Cuban enterprises, that will execute the projects, are part of the government as they are part of the MoC. Besides that, Cubanacan is a state enterprise that runs several hotels, like Blau Arenal Hotel, and is part of the MoT. The Instituto Superior Politécnico José Antonio Echeverría (CUJAE) is the university of technology of Havana and is part of the MoHE. The hydraulic research center (CIH) of the CUJAE performs research on the project.

OTHERS

Other stakeholders that are involved include tourists, local residents and international contractors.

2.2.2. GOVERNMENTAL RELATIONS

To give an overview of the structure of the government of Cuba with its ministries and enterprises that are involved in the Blau Arenal Hotel project, a relationship diagram is created (see figure 2.6).

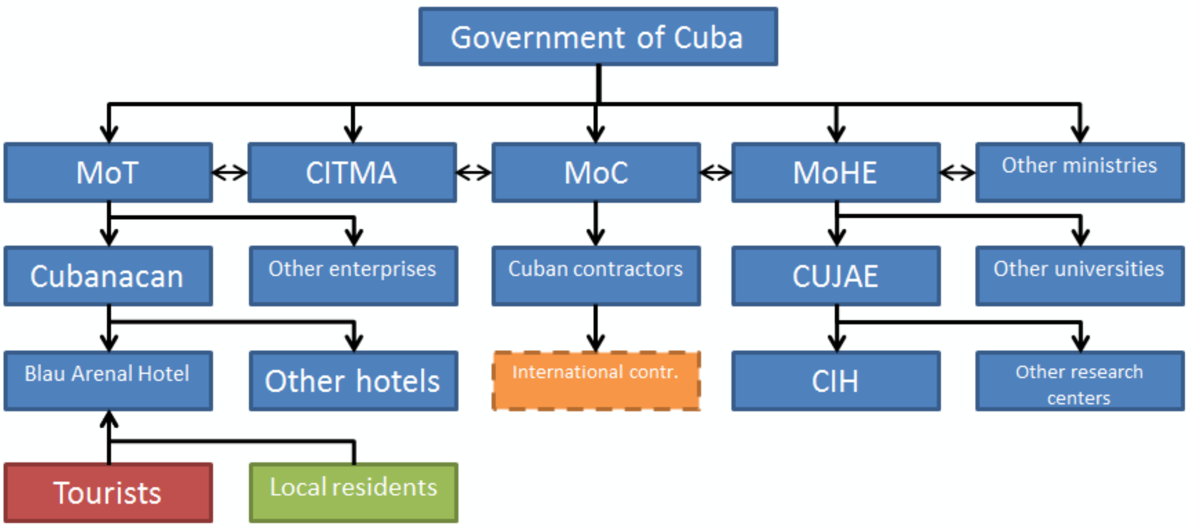


Figure 2.6: Overview structure of government of Cuba involved in Blau Arenal Hotel project

As can be seen in figure 2.6, the boxes representing the government are blue. It can be concluded that the government of Cuba has a lot of power in Cuba with its organisations in every sector.

The ministries are interconnected with arrows on both sides, because they need each other to realize what they want. Take for example this project; CITMA wants the MoT to do something about the flooding and waste water contamination in an area with tourism. The MoT needs the MoC for their contractors and the MoHE that provides them with information obtained by the CIH of CUJAE.

The three other stakeholders are also included in figure 2.6. The tourists (red box) and the local residents (green box) are connected to the Blau Arenal Hotel by a reversed arrow, because they depend on the hotel. For them, the hotel should remain attractive as a holiday location. The international contractors are in the orange box. The box is dashed, because it is not yet known if the expertise of these contractors is needed.

2.2.3. STAKEHOLDER INVOLVEMENT

Each of the stakeholders has its own interest in the project, a problem perception and a certain goal that they strive for. They use their own resources, like money, power and knowledge to reach their goal. The different interests, problem perceptions, goals and resources of the stakeholders are elaborated in figure 2.7.

The main goal of the stakeholders is to improve the safety of the area against flooding and the waste water contamination. Their interests, however, are different. For the tourist dependent stakeholders, like the MoT, Cubanacan and Blau Arenal Hotel, their interest is to earn money with the hotel. They are also the ones who are investing money into the project to keep CITMA from closing the hotel. CITMA wants to preserve the ecosystem and to do so it can use its (political) influential power. This protection of the unique mangrove forest is also favourable for the tourism and its stakeholders, because Blau Arenal Hotel, surrounded by mangrove forest, is an attractive holiday location for tourists. In figure 2.7, it can be seen that the ministries have more (political) influence than the other stakeholders. Furthermore, the knowledge and expertise of the CUJAE, Cuban- and international contractors can be used, both with different interests and goals.

Stakeholder	Interest	Problem perception	Goal	Resources
CITMA	Protection and maintenance of environment.	The mangrove forest ecosystem is not protected against flooding and contamination.	To protect the unique ecosystem of mangrove forest.	Political and influential power that can close the hotel.
Ministry of Tourism	Development of tourism at Playas del Este. Earn money with tourists.	Without a safe area against flooding and contamination CITMA will close Blau Arenal Hotel, which means no tourists and no income.	To realize a safe and attractive solution that attracts many tourists and satisfies CITMA.	Political and influential power and money to invest.
Cubacan	Earn money with their hotels.	CITMA can close the hotel if the flooding and waste water issues are not solved, which means no tourists and no income.	To get a safe hotel that is attractive for tourists due to its unique location.	Some influential power and money to invest.
Blau Arenal Hotel	Earn money with the tourists that come to Blau Arenal Hotel.	CITMA can close the hotel if the flooding and waste water issues are not solved, which means no tourists and no income.	To get a safe hotel that is attractive for tourists due to its unique location.	Little influential power.
Ministry of Construction	Earn money with projects.	-	To construct and execute projects.	Political and expertise in construction.
Cuban contractors	Earn money with projects.	Find a solution against the flooding and waste water problem.	To work on the project and come up with the best solution.	Expertise in construction of hydraulic structures.
Ministry of Higher Education	Get funds by doing research. Development of quality of higher education.	-	To provide data about the project to the MoT.	Political and funds.
CUJAE including CIH	Funds for research and education. Gain knowledge by doing research.	Find a solution against the flooding and waste water problem.	To gain more data about the Cuban coast (this project area in particular). To come up with new/best solution(s) for coastal protection.	Knowledge, expertise and manpower in hydraulic research.
Tourists	Have a pleasant stay in Playas del Este (and Cuba).	If the hotel is not safe and attractive, the tourists will not spend their holidays here.	To spend their holiday on a unique location in Cuba.	Money to spend.
Local residents	Live in the area. Earn money with tourist business.	The locals need a safe living area. They will lose income when tourists will not visit the hotel.	To live in a safe area with tourists from the Blau Arenal Hotel to do business.	Little influential power.
International contractors	Earn money with projects (in Cuba).	Find a solution against the flooding and waste water problem in case Cuban contractors need external expertise. Want to do more projects in Cuba.	To work on the project and improve the relation with the Cuban government for future projects.	Expertise in construction of hydraulic structures.

Figure 2.7: Interests, problem perceptions, goals and resources of the different stakeholders

2.2.4. POWER AND INTEREST

To see how the different stakeholders act regarding the project, a power-interest graph is made (see figure 2.8). The influence of each stakeholder is expressed in the power they have to make decisions with respect to the project, shown on the horizontal axis. Their interest is valued as well and is shown on the vertical axis.

As can be seen in figure 2.8, a lot of stakeholders have a high interest in the project. Besides that, it can be seen that the higher in the governmental hierarchy, the more power the stakeholders have. The ministries are on the right side of the graph and their state enterprises and hotels more on the left. CITMA has the most power as they can close the hotel with their political influence. Their interest is high as they want to preserve the mangrove forest ecosystem. The MoT has the most power, when looking at influence and money, of the tourist dependent stakeholders, but their interest is lower than the more locally involved stakeholders, like Cubanacan and the Blau Arenal Hotel.

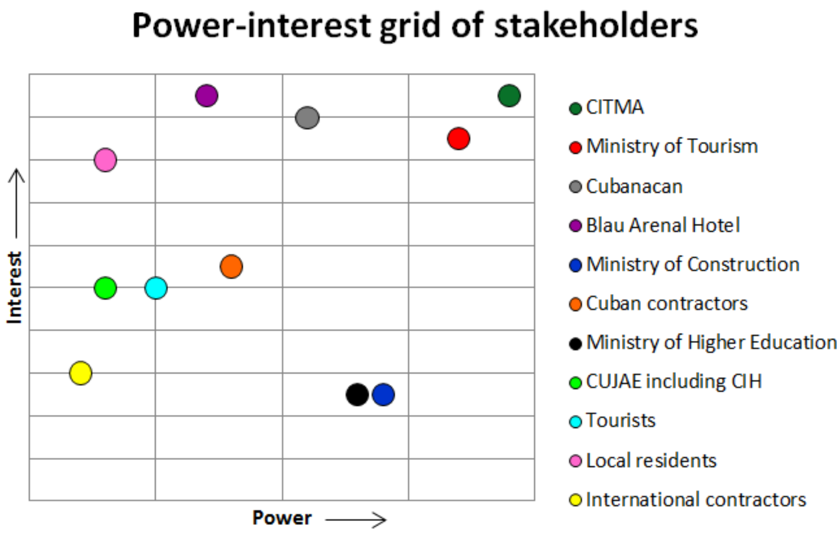


Figure 2.8: Power-interest grid of the stakeholders

The Ministry of Construction and the Ministry of Higher Education have political power, but their interest in this project is low. The CUJAE and CIH have interest in coastal projects, but not particularly in this project. They have limited power, but their knowledge is certainly important for the project. The tourists have some power and interest as they are deciding whether they spend their money at Blau Arenal Hotel or somewhere else. The local residents are dependent on the tourist business, so they have high interest. Unfortunately, they have fewer resources, so their power is lower. The international contractors have some interest to execute projects in Cuba, but can earn their money with projects elsewhere in the world.

2.2.5. CONCLUSION

It can be concluded from the entire stakeholder analysis that CITMA and MoT (and their provincial representatives) are the main stakeholders, see also the power-interest graph in figure 2.8. Besides that, the government of Cuba has a lot of power with all its ministries and state enterprises in this project. This confirms the fact that Cuba is ruled by the government only.

The local residents are another important stakeholder as they are the only non-governmental stakeholder with high interest in the project due to their dependence of the tourist business to earn their living. The tourists need to be satisfied by the tourist dependent stakeholders, because their holidays at the hotel are an important source of income for the Blau Arenal Hotel and the surrounding local residents. Their decision to visit Blau Arenal Hotel (or not) gives them an indirect power where the MoT, Cubanacan and the Blau Arenal Hotel have to cope with.

Furthermore, the main interests of the different stakeholders are preservation of the environment and improving tourism. The environmental goal of the project is to prevent the area from flooding and protect the mangrove forest against waste water contamination. The touristic goal is to create a hotel that is safe for flooding and has an attractive surrounding. Both goals can be reached by the stakeholders if they can come to an integrated solution taken each other's interests and goals into account.

2.3. OCEANOGRAPHY

2.3.1. STORMS

Storms are an important phenomenon in the northern regions of Cuba. Yearly multiple storms and cold fronts pass Cuba and bring strong winds, heavy rains and high waves with them. The Meteorological Institute [4] provides an article which contains data from measurement station Casablanca, located close to the Malecón boulevard in Havana. Information about 45 cold fronts and hurricanes that resulted in inundations is shown in appendix E.

COLD FRONTS

Cold fronts occur at a location where cold air from the poles meets warmer air from lower latitudes. The colder air moves under the layer of warm air, which causes strong mixing of the warm and cold air. This results in heavy rainfall, thunderstorms, a temperature drop and high wind speeds at the location of the front.

In Cuba, cold fronts mostly occur in the storm season (November to April, as described in section 1.1.3). They develop in the northern parts of Cuba, when cold and warm air meet at that location. As described above, they can cause a lot of problems, such as inundation of the land. This mostly happens if the strong winds result in high waves at sea.

HURRICANES

Hurricanes are more severe storms with a characteristic rotating movement of the system. They are formed at a location where an initial disturbance, such as a cold front, occurs above warm seawater. At middle latitudes, the disturbance will cause clouds to rotate around a centre with minimal pressure due to the Coriolis effect. Hurricanes occur in Cuba with a return period of 5.9 years. The extraordinary event of a year with two hurricanes has a return period of 66.2 years [4].

In the tables in appendix E the wave height and period of all cold fronts and hurricanes during a period of 35 years are given. The maximum wave height that occurred during a storm in that period is 5.8 m.

2.3.2. TIDE

The astronomical tide can have a large influence on the water level in the study area. To analyse its behaviour, data from the annual NOAA (National Oceanic and Atmospheric Administration, located in the USA) tide predictions has been taken [12]. NOAA uses over 3000 stations in key areas in each state. NOAA has two different categories of stations:

- Harmonic, which means that the predicted water level values are conducted by combining the harmonic constituents into a single curve.
- Subordinate, which obtains the high and low water values by means and differences, and ratios applied to the full harmonic constant predictions at a specific harmonic station (a reference station).

To receive the correct information for the study area, a reference station is used. Key West is the reference station best suited for the extrapolation of the northern coast of Cuba. Important to know is that these values are predictions. They are published annually on October 1st, are not from the exact location of interest and only contain one year. For these reasons the values may differ slightly in reality. However, they do give a good enough estimation for this study, because it contains the characteristics of the expected tidal signal and wave.

The data from NOAA consists of the time and height of each high- and low water every day. For this specific area it means that for almost every day four data points are available. These values have been used to create tidal signals, like the one in figure 2.9.

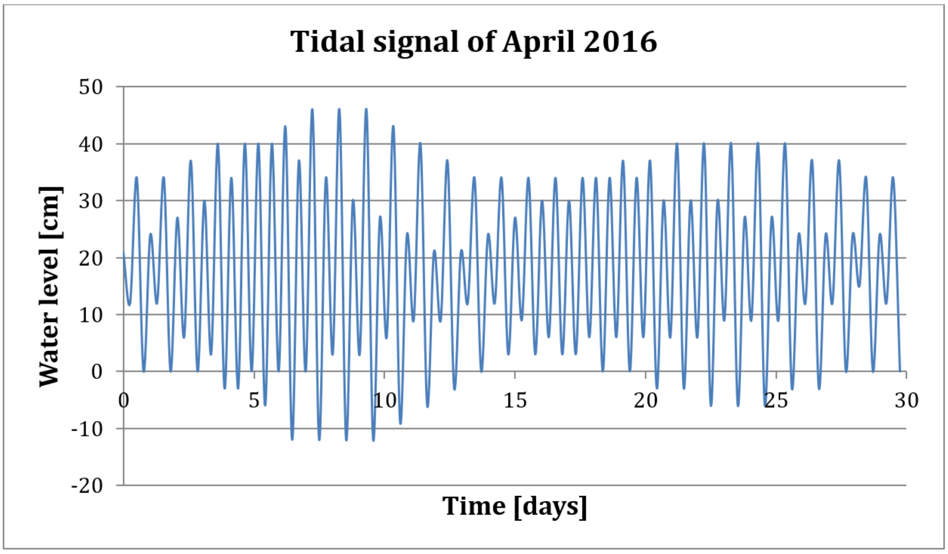


Figure 2.9: Prediction of tidal signal of April 2016 [12]

A tidal signal can be characterized by the form factor. This form factor F is determined as the ratio of the amplitudes of the sum of the two main diurnal components K1 and O1 and the sum of the two main semi-diurnal components M2 and S2 [9]. As the specific components can not be analysed, the tidal curve can be categorized by its shape, which originates from the tidal components. When analysing the tidal signal in figure 2.9, the tidal character of the study area is a mixed type, but predominantly semi-diurnal. This means that the form factor of the tidal character is between 0.25 and 1.5 [9].

Additionally, an analysis of the tidal range and maximum water level has been done. The same data from NOAA has been used to determine the average and maximum tidal range and maximum water level for each month. The water level is determined with the mean sea level as reference. In table 2.2, these values are shown. According to *Servicio Hidrográfico y Geodésico de la República de Cuba*, the tidal range of the area has a value between 0.25 m and 0.5 m [13]. The analysis of NOAA is in agreement with these values. Therefore, a tidal range of 0.5 m is used in the simulations. However, to calculate the tidal prism the total average tidal range is used, which can be obtained from table 2.2. In figure 2.10 the tidal signal of the month December is shown, with the largest differences.

Table 2.2: Different tidal ranges and maximum water level of the study area for each month of 2016 [12]

2016	**Tidal Range**		**Water level**
Month	*Average [cm]*	*Maximum [cm]*	*Maximum [cm]*
January	30.3	58	46
February	31	55	43
March	30.8	55	43
April	31.4	58	46
May	31	64	49
June	31.6	64	49
July	31.4	61	49
August	31.5	55	49
September	31.6	49	52
October	31.6	61	58
November	31.4	64	58
December	31.1	67	55
Total	31.2	67	58

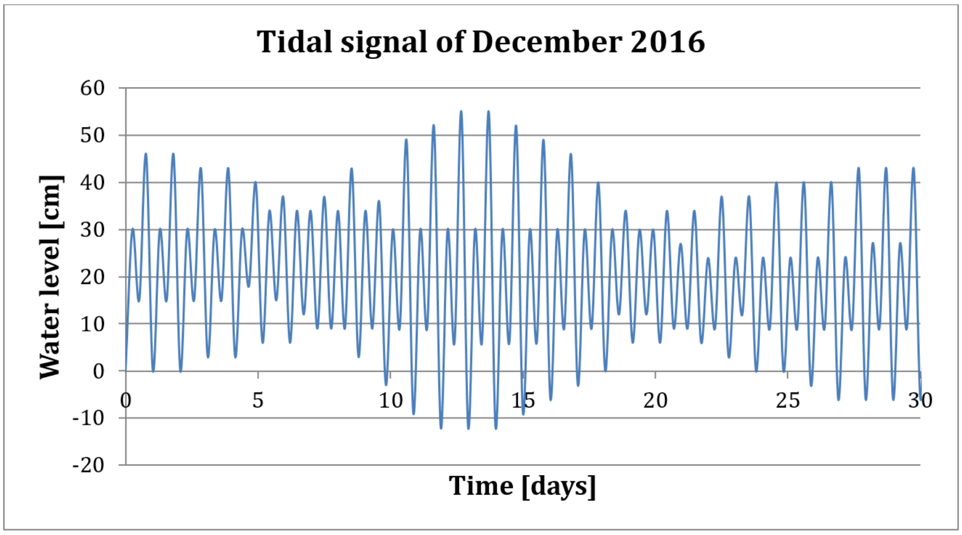


Figure 2.10: Prediction of tidal signal of December 2016 [12]

2.3.3. CURRENTS

The tidal currents are the main type of alongshore currents that are present near the coast of Cuba. At the northern coast of Cuba these tidal currents have low flow velocities due to the small tidal range of 0.31 m (and maximum of 0.67 m) of the mixed semi-diurnal tide (see section 2.3.2). During flood, the main direction of the tidal current is east-northeast and, during ebb, its direction is mainly to the west-northwest. These tidal currents have a mean velocity of 0.1 m/s and a maximum of 0.37 m/s [14].

Other types of currents can be induced by the wind or caused by differences in temperature, density, salinity and pressure in the water mass distribution. When these currents are directed in the opposite direction of the tidal currents, they can counteract the tidal currents [14].

The velocities of these currents, including tidal currents, are low, so their effect will not be of importance for this project. Therefore, they will not be taken into account.

2.3.4. WAVES

The wave conditions that are present at Boca Ciega beach are an important part of the flooding problem. In order to determine a reasonable significant wave height that occurs during different conditions, a statistical analysis of wave data is done in this chapter. Multiple sources of wave data are available for the analysis: data from BMT Argoss [15] and the Cuban Meteorological Institute [4]. Besides that, in appendix E data on waves during storms is already presented.

BMT ARGOSS

BMT Argoss provided wave data measured by satellites in an area of 100 x 100 km^2, north of Boca Siega beach. The data contains a value of the average wave characteristics every three hours for a period of 23 years. In order to determine the significant wave height of a design storm, a Peak over Threshold method is used. This method is extensively described in appendix F. A short summary and the outcomes will be described in this section.

ASSUMPTIONS

The full dataset contains waves travelling in all directions with a large range of wave heights. Waves are assumed to reach Boca Ciega beach only if they originate from directions between 292.2° and 67.5°. Also, waves are assumed to be part of a storm if their wave height exceeds a threshold of 2.0 m. If multiple consecutive waves exceed the threshold, they belong to one storm.

RESULTS

The method is performed with an exponential-, Weibull- and Gumbel distribution. In figure 2.11a the results are plotted, with the return period on the x-axis and the significant wave height on the y-axis. As can be seen, the significant wave height for large return periods does differ, but lies within 0.6 m for all distributions.

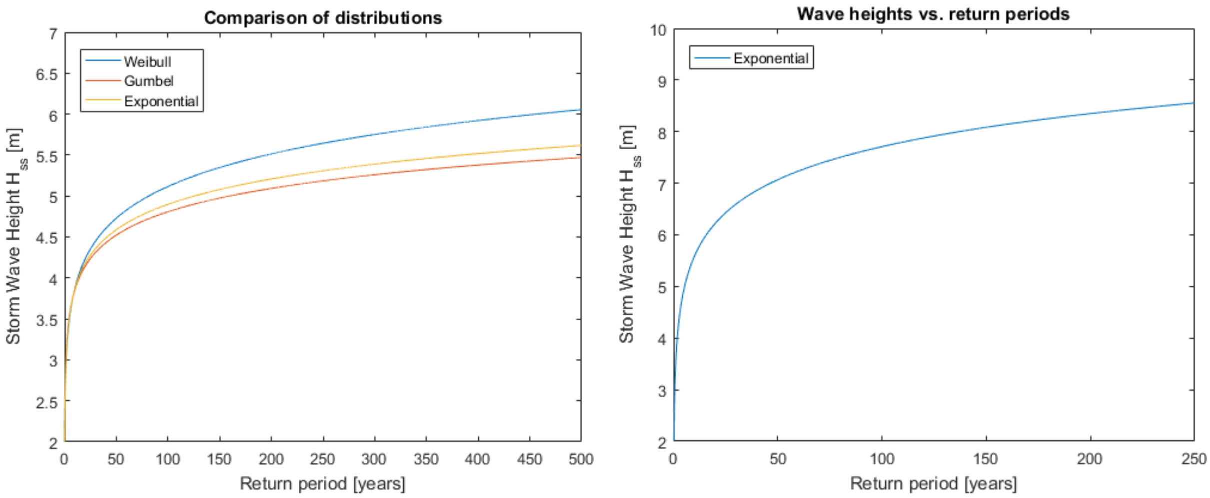
METEOROLOGICAL INSTITUTE

The data provided by the Meteorological Institute [4] is shown in table 2.3. It is not clear what data is used to produce the table. When compared to the BMT Argoss data, table 2.3 shows higher wave heights (with a factor 2).

Table 2.3: Return periods and significant wave heights provided by the Meteorological Institute [4]

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

The data from the tables in appendix E can be used to create a similar table. The method to create the table is the same as for the BMT Argoss data, a peak over threshold method with 2.0 m as threshold. Figure 2.11b shows the calculated values. These values coincide better with the results from the BMT Argoss dataset.



(a) Comparison of the different distributions, calculated with data from BMT Argoss

(b) Return periods and significant wave heights calculated with the data in appendix E

Figure 2.11: Results of the statistical wave analysis

CONCLUSION

Since it is not known where the data from the Meteorological Institute originate from, it can not be stated which of the data sources provides the best data. The data from the Meteorological Institute are presented in the article about tropical storms and hurricanes, so it is plausible that the data is based on storms only.

The BMT Argoss wave data are based on a full dataset of measurements in 23 years. However, the presented data are average values from three hours of waves. Because of this, extreme values are smeared out over three hours and therefore disappear in the dataset. In that way, tropical storms and hurricanes are not represented correctly by the BMT Argoss data.

In table 2.4 the values from the BMT Argoss data, the Meteorological Institute data and the calculated data in figure 2.11b are compared. As described above, both the BMT Argoss dataset and the data from the Meteorological Institute have their uncertainties and downsides. Therefore it seems reasonable to take the calculated values (third column in table 2.4) as governing.

Table 2.4: Comparing return periods and significant wave heights from different data sources

Return period [y]	BMT Argoss	Meteo	Calculated
5	3.5	6.0	4.9
10	3.9	6.9	5.6
20	4.2	7.8	6.2
50	4.7	9.2	7.1
100	5.1	10.1	7.7

The peak wave period of the design storm is calculated using the average steepness of the waves and the design wave height. Equation 2.1 shows the calculation of T_p.

$$T_p = \sqrt{\frac{2\pi \cdot H_{ss}}{s \cdot g}} \tag{2.1}$$

2.3.5. SURGE

Surge is the temporary rise of the water level at a coast. Surge is often referred to as storm surge, because the extreme weather conditions of a storm are causing the water level to increase. The three components that induce the storm surge are wind set-up, wave set-up and atmospheric pressure. Wind set-up can be explained as the onshore directed wind causing an increase in water level due to the shear force of the wind on the water surface.

The water level rise due to the breaking of waves in the surf zone is the wave set-up. The maximum wave set-up can be obtained from the balance between the water level slope and the change in radiation stress. The wave set-up is dependent on the wave direction, the breaker index and the wave height. Normally, the atmospheric pressure is 1,013 mbar. The storm surge is caused by a pressure decrease. With each 1 mbar pressure drop, the water level rises with 0.01 m.

From the article of the Meteorological Institute, data about the storm surge and its contributions caused by the hurricanes Kate, Rita and Wilma is obtained (see table 2.5) [4]. These measurements are done at a station close to Havana.

Table 2.5: Storm surge and its contributions of the hurricanes Kate, Rita and Wilma [4]

Hurricane	Date	Wind set-up (m)	Wave set-up (m)	Atmospheric pressure (m)	Total storm surge (m)
Kate	11-11-1985	0.4	0.41	0.31	1.12
Rita	21-09-2005	0.9	0.31	0.4	1.61
Wilma	24-10-2005	-	-	-	1.53

To have a save margin for the storm surge used in this project, a storm surge of 1.56 m is estimated, consisting of a wind set-up of 0.65 m (the average of hurricanes Kate and Rita), a wave set-up of 0.41 m and an atmospheric pressure of 0.4 m (the maximum values of the two hurricanes in table 2.3). The average value of the wind set-up is taken, because this contribution is very variable.

2.3.6. SEA LEVEL RISE

According to the Fourth Assessment Report (AR4) of the Intergovernmental Panel on Climate Change (IPCC), the global mean sea level (GMSL) rise is expected to be 0.5 m for the next century [16]. A more extensive elaboration about the global- and regional sea level rise is performed in appendix G.

For this project in Cuba, taking into account a GMSL rise of 0.5 m for the next century a rate of the mean sea level rise of 5 mm/year is estimated. This is somewhat higher than the expected regional value to deal with the uncertainty in the predictions.

2.4. MORPHOLOGY

2.4.1. SEDIMENTOLOGY

In this section, the sediment is analysed. The sediment characteristics are elaborated as it is an important factor influencing the sediment transport and beach profiles, including erosion and accumulation, in the project area. The composition and lightness of the sediment is presented in appendix H

CHARACTERISTICS

From the article *Evolution of the texture and composition of beach sand in playas del Este, Havana, Cuba* [17], the median grain size, standard deviation and texture classification are obtained. The beach of Boca Ciega is also researched. As can be seen in table 2.6, the median grain size varies between 0.28 mm (max. erosion) and 0.54 mm (max. accumulation) [17]. The large difference in grain size even causes a change in texture classification; the sand becomes coarser when the beach accumulates more and more.

Table 2.6: Sediment characteristics at Boca Ciega Beachs[17]

	Median grain size (mm)	**Standard deviation (ϕ)**	**Texture classification**
Maximum erosion	0.28	0.72	Moderate sand
Maximum accumulation	0.54	1.26	Coarse sand

2.4.2. EROSION

The Institute of Oceanology is the Cuban organisation doing research on the erosion of the Varadero beaches. They estimated an erosion rate of 50,000 m^3 sand per year by comparing the measured beach profiles over the years. Besides that, the regression of the beach was expected to be in the order of 1.2 m per year [18]. These values are obtained from the Varadero beaches, because according to *Dr. Ing. Luis Fermín Córdova López* from *CUJAE*, the erosion rates at the Varadero coastline are comparable to the erosion rates at Playas del Este. This is in accordance with the short distance between the Varadero beaches and Playas del Este (100 km), resulting in the same wave field at both coasts. In appendix H, the causes of erosion are described. During the site visit, the erosion was clearly visible in the form of large scarps and dune regression (see appendix A).

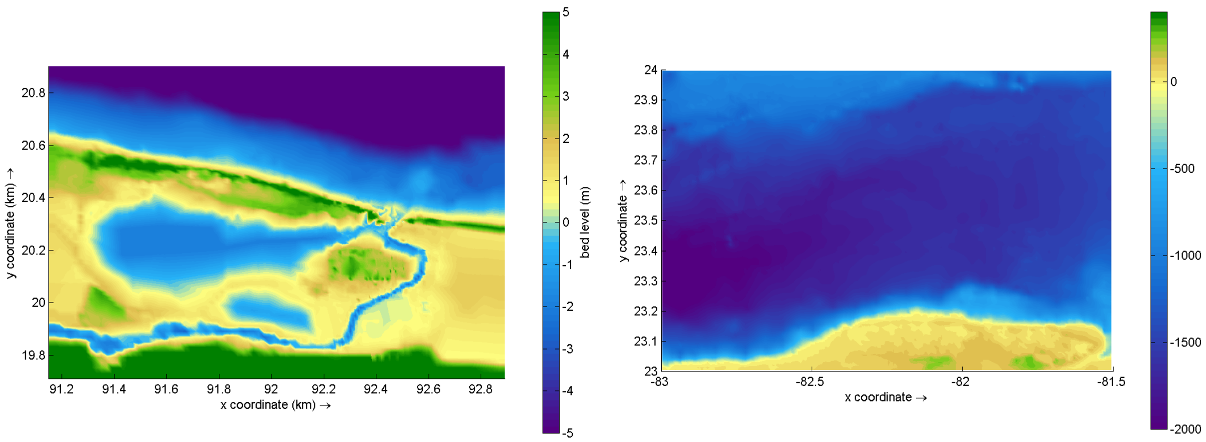
2.4.3. BATHYMETRY

Data on the bathymetry that is used, consists of three parts. All parts are set to the WGS84 coordinate system:

- Bathymetry and topography of the Blau Arenal Hotel, the area around it and the nearshore coast are provided by GeoCuba [19]. This data is measured with laser equipment. Therefore, the locations with mangrove forest are not included in the data. A visualisation is shown in figure 2.12a. The hotel is located between x-coordinates 92.2 and 92.5 and y-coordinates 20.1 and 20.3. The river and the lagoon are clearly visible. It can be seen that most of the land is only 1 to 2 meters above mean sea level. The Blau Arenal Hotel lays a little higher: 2 to 4 meters above MSL.

- The bathymetry of the deep ocean is obtained from GEBCO [20]. The data from this source is measured using an echo-sounding technique. Every 925 meters a point is measured. This dataset is coarse, but for calculations in the deep ocean it is useful. A plot of the data can be seen in figure 2.12b.

- Data on the bathymetry of the coast in front of Playas del Este is also obtained from GeoCuba [19]. From MSL -40 m to MSL 0 m, the resolution is very high. More into the deep ocean, the points get more coarse.



(a) Topography of the Blau Arenal Hotel coordinates are converted for the XBeach model (see appendix O)

(b) Bathymetry of the Gulf of Mexico, coordinates are in WGS84.

Figure 2.12: Plots of the bed level data

2.5. TIDAL INLET

The Itabo River flows into the Gulf of Mexico at the location of the Blau Arenal Hotel. It is important to know the stability of the river mouth, because it can have an effect on the possibility of flooding of the hotel and waste water treatment plant. Closure of the river mouth leads to blocking of the river discharge, so the area will possibly flood faster. Therefore, due to the unstable river mouth, extra scenarios will play a role in the simulation of the designs.

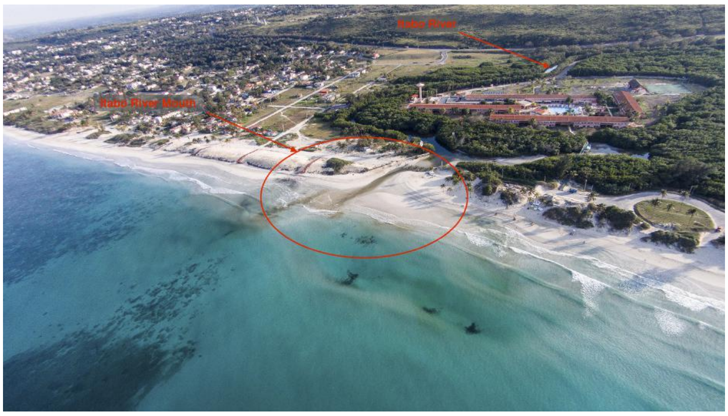


Figure 2.13: Aerial view of the location of the Itabo River and river mouth [1]

In appendix I, the tidal inlet has been analysed using Bruun and Escoffier. The result is that the inlet is a type 3 inlet, which means that it closes at irregular or seasonal intervals, but is stable at its location, which means the location of the mouth does not change. The most likely situation is that the inlet closes at the end of summer and fall, and remains open the winter and spring. The tidal prism, which is the tidal range times the area of the lagoon, alongshore sediment transport, wave energy and river discharge, causes the closure of the river mouth. The results of the Itabo River mouth reach the conclusion that indeed scenarios, where the river mouth is closed, have to be taken into account.

2.6. HYDROLOGY

In this section the hydrology of the Itabo catchment area is described. The main objective is to obtain river discharges given a certain return period.

2.6.1. METHODOLOGY

The rational- and the triangular unit hydrograph method are used to calculate the river discharge of the Itabo catchment area. Both are briefly discussed here, for a detailed explanation see appendix J.

RATIONAL METHOD

Little data is available about the small Itabo catchment area, in particular the discharge data is missing. In order to come to a relation between rainfall and river discharge, the rational method is used. The rational method is a quick and rough estimation of the discharge with as input parameters: the catchment area, constant and uniform rain intensity and the runoff coefficient. The advantage of this method is that little information is needed to estimate the river discharge. However, a big disadvantage is the considerably uncertainty that hides in the determination of the runoff coefficient [21].

UNIT HYDROGRAPH METHOD

The triangular unit hydrograph method is used to determine the river discharge based on precipitation histogram of a rain shower. The result of this method is a hydrograph: the river discharge over time. The precipitation input for this method is the synthetic histogram: a histogram of the rain based on Intensity Duration Frequency curves (IDF-curves). As with the rational method, not all precipitation will contribute to the direct runoff. The method to calculate the 'losses' is complicated and detailed information about the soil and terrain is required. Furthermore during heavy rainfall the soil can be highly saturated, which reduces the infiltration. In this case the precipitation can become fully effective, which means no 'losses' [22]. Therefore, no 'losses' due to for example evaporation and infiltration, are assumed. This assumption is conservative since the real river discharge is lower.

2.6.2. ITABO CATCHMENT AREA

The Itabo catchment area is a relatively small, rectangular shaped area with a total of 35.6 km^2. The main type of vegetation is grasslands and cultivated terrain. Due to the small area, the concentration time ("the time required for the farthest point of the catchment to contribute to runoff" [21]) of the catchment is also relatively short: 2.8 hours.

Figure 2.14 shows the catchment area of the Itabo River. The main characteristics of the river and catchment area are listed in table 2.7.

Figure 2.14: Catchment area Itabo River [23]

Table 2.7: General information Itabo river

Characteristic	Value	Dimension
Catchment area [A]	35.6	km^2
Maximum length catchment area [L]	10	km
Concentration time [t_c]	168.3	min
Runoff coefficient [C]	0.4 - 0.8	–

2.6.3. RAIN DATA

Three sources of rain data are available for this analysis:

- Monthly and yearly averaged rain data [23].
  The data can be found in appendix J, figure J.8
- Intensity Duration Frequency curves [24] for a location within the Itabo catchment area: La Habana del Este.
- Precipitation data of the 1982 tropical system, this precipitation is calculated as described in appendix J. This event caused severe flooding in the Itabo River area.

The monthly and yearly averaged rain data is used to calculate the mean river discharge. For the rational method the IDF-curves are used in combination with the area specific concentration time. The duration of the rain is taken equal to the concentration time t_c, which leads to a rain intensity for a certain return period. A longer duration, with the same return period, gives per definition a lower peak discharge due to the lower rain intensity. A shorter duration, higher intensity event can not be modelled with the rational method.

A rain histogram per rain shower is required for the triangle unit hydrograph method. For this purpose the synthetic histogram is calculated based on the IDF-curve, this method is described in appendix J. Furthermore, the rain data from the 1982 event is used as input for the triangular unit hydrograph method, to evaluate this tropical system.

2.6.4. RIVER DISCHARGE

PEAK RIVER DISCHARGE

The peak river discharge is calculated based on the rational method and input data. The upper and lower bound of the runoff coefficient C are used, because this coefficient is not known for the catchment area. The value of C is between 0.4 and 0.8. The triangular unit hydrograph method results in a hydrograph, the peak of this hydrograph is also presented in table 2.8.

Table 2.8: Peak discharge Itabo river

	Rational method		*Unit hydrograph method*
Return period [years]	**Q [m^3/s] for C = 0.4**	**Q [m^3/s] for C = 0.8**	**Q [m^3/s]**
2	113.4	226.7	172.4
5	141.2	282.3	219.5
10	157.3	314.5	246.7
25	178.3	356.5	282.2
50	199.9	399.8	319.0
100	214.4	428.7	342.8
1000	269.9	539.9	436.9

MEAN RIVER DISCHARGE

The mean river discharge is low during the whole year, as can be seen in table 2.9. The highest discharges are found during the wet season, May till October. The graphs of the discharges and the year mean discharge from 1970 - 2006 can be found in appendix J, figure J.17 and J.18.

Table 2.9: Mean river discharge (m^3/s) per month and per year

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Q [C = 0.4]	0.32	0.36	0.29	0.31	0.64	1.16	0.71	0.71	0.95	0.78	0.57	0.40	0.59
Q [C = 0.8]	0.64	0.71	0.58	0.63	1.29	2.33	1.42	1.43	1.90	1.56	1.14	0.79	1.18

HYDROGRAPH PER RETURN PERIOD

The hydrograph, of the Itabo River, based on the triangular unit hydrograph method is calculated for the different return periods. The precipitation input data, per return period, is the synthetic histogram based on the Intensity Duration Frequency curve from station CH-345, located inside the Itabo catchment area. This result can be seen in appendix J, figure J.19.

HYDROGRAPH 1982 EVENT

The precipitation data from the 1982 event is used as input for the triangular unit hydrograph method. As with the previous calculation for the synthetic hydrograph, no reduction factor for 'losses' is used. This is an overestimation of reality, but will lead to a conservative result. The precipitation will become fully effective, so no losses, only when the soil is sufficient saturated or when the soil is impermeable [22]. This precipitation data is interpolated in order to get a rain distribution with equal time steps, which makes the computation easier. This rain distributions can be seen in appendix J, figure J.14 and J.15. The resulting hydrograph can be seen in the same appendix, figure J.21. The peak discharge is equal to 731.5 m^3/s.

2.6.5. DISCUSSION

The method used to calculate the peak discharge has a number of limitations:

- The result is highly sensitive to the runoff coefficient C, as can be seen in appendix J, figure J.22. Furthermore, the determination of this coefficient is difficult without detailed information about the catchment area.
- The rational method can be used in case of a uniform rain intensity with a duration equal to the concentration time of the catchment area. However, a high intensity, short duration gives a higher peak river discharge compared to the long duration, lower intensity rain [25]. This can not be calculated using this method.
- The rational method results in the peak discharge at the time equal to the concentration time t_c. It does not give any information about the shape of the hydrograph. However, the peak discharge calculated with the triangular unit hydrograph method is in the range of peak discharges calculated by with the rational method.
- The hydrograph calculated with the triangular unit hydrograph method will be used in the XBeach models. The return period used in the model will depend on the different scenario's.

2.7. RIVER DISCHARGE REDUCTION

2.7.1. GENERAL

The river discharge, peak and hydrograph, determined in 2.6, is the river discharge at the river mouth. This is because of the fact that the whole catchment area contributes to the runoff. An important item, not accounted for in the rational method and triangular unit hydrograph method, is the reduction of discharge due to outflow points. The river consists of a main branch. At certain points the discharge of the river reduces due to outflow through culverts (see figure 2.15), secondary branches and local inundation.

Figure 2.15: Locations of the culverts [26]

The amount of outflow is a function of the discharge. Low discharge might result in a higher percentage of water through the main branch compared to high discharge events. Below some factors that affect the amount of outflow, over the whole river length are listed below:

- **Weak spots in river bank**
  A weak spot in the river bank might cause inundation of local land. This inundation is a local storage of water, which causes a reduction of the (peak) discharge.

- **Culverts**
  At certain locations culverts are constructed in the river bank. In case of a water head difference, water will flow through the culvert to the other side.

- **Secondary branches**
  Secondary branches are branches that bifurcate from the main branch. A certain part of the water will go through the secondary branches.

- **Bifurcation layout**
  The layout of a bifurcation determines the division of water through the different branches.

In summary, there are a lot of factors determining the reduction in river discharge. A detailed study of the Itabo River is needed to quantify the reduction. A conservative assumption is made in section 2.7.3.

2.7.2. BLAU ARENAL HOTEL ZONE

The neighbourhood of the Blau Arenal Hotel zone is studied in order to qualitatively determine the reduction of the discharge. In figure 2.16 the Blau Arenal Hotel area is shown. The XBeach model boundary is shown with the orange dotted line. The discharge input in the XBeach model is at this point. Because of the short distance to the mouth of the river, it is allowed to use the hydrograph from the triangular unit hydrograph method, see section 2.6. Because the catchment area in front of the model area is very small, so it can be neglected.

There are two outflow points in the river shown in figure 2.16. One outflow point is a small pond on the western side of the main branch. This pond can not store much water, but in case of high river discharges and flow velocities, the water flow will partly inundate the land north of the pond.
The other outflow point is a culvert. Water will flow through this pipe when there is a water head difference between the lagoon behind the road and the main branch of the Itabo River. The diameter of the culvert is not known, so neither is the capacity.

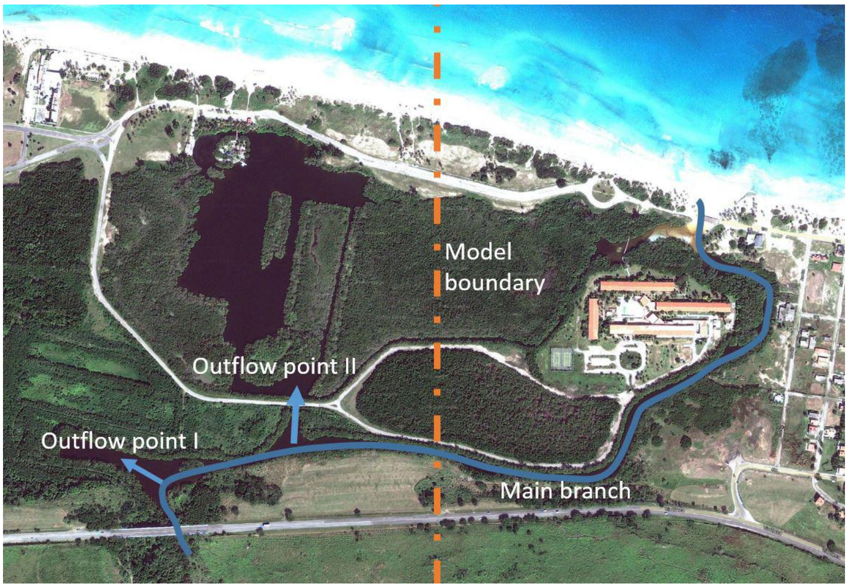


Figure 2.16: Overview Itabo river Blau Arenal Hotel area

A detailed analysis of the reduction of the discharge is recommended for a more reliable estimate of the river discharge that is put in the model at the model boundary. For this study an engineering estimation has been made to get a more representative approximation of the real discharge.

2.7.3. ESTIMATION REDUCTION RIVER DISCHARGE

At the inflow point of the river, in figure 2.16, a bridge is shown. The road, that overpasses the river, is at a high elevation. The bridge with its piers is assumed to have no reduction on the river discharge. Two outflow points are shown in figure 2.16, the first and second outflow point are assumed to result in 10% losses. The reduction might be higher in reality, because of the sharp bend in the river. However, a conservative estimation is preferred because of the high uncertainty. The hydrographs for the governing situations, 100 and 225 years return period, are reduced by 10%. The result of this can be seen in section 2.11.

2.8. CONSEQUENCES

The Hotel Blau Arenal zone can be divided in the following sub-systems:

- Environment
- Hotel
- Waste water treatment plant
- Tourists

Flooding of the Blau Arenal Hotel area can lead to multiple consequences. The following division can be made: tangible versus intangible consequences and direct versus indirect costs. The consequences are summarised in table 2.10. Further elaboration of the different consequences is given in appendix K. Point six is of major importance, therefore it is worked out below.

Table 2.10: Summary of the consequences

	Tangible	**Intangible**
Direct	1. Damage waste water treatment plant 2. Damage hotel facilities 3. Evacuation and rescue operation 4. Clean up and reconstruction costs	6. Fatalities and injuries 7. Environmental losses
Indirect	5. Decrease tourists revenues	8. Damage to reputation Ministry of Tourism

6. Fatalities and injuries

The maximum capacity of the hotel is approximately 300 tourists and the total staff consists of 200 persons at most [8]. So, at most there are 500 persons at risk. The likelihood of the maximum occupancy during an extreme event is not very high, but it is the upper boundary for the total amount of people in the area. The total amount of people in the area is not equal to the total estimation of loss of life during a flood. The total estimation of loss of life is expressed by equation 2.2 [27].

$$N_{loss} = F_d \cdot (1 - F_E) \cdot N_{par} \tag{2.2}$$

Where:

Where:	N_{loss}	=	Loss of life estimate
	F_d	=	Mortality fraction
	F_E	=	Evacuation fraction
	N_{par}	=	Number of people at risk

The total number of fatalities is expressed by the total number of people in the area, reduced by the amount of people that are evacuated. This total amount of people at risk is multiplied by the mortality fraction. This mortality fraction is a function of the flood characteristics. A flood with a large inundation depth and a flood that rises quick will have a large mortality fraction. For the Blau Arenal Hotel the evacuation fraction is assumed to be high, because the total amount of people to be evacuated is relatively small. The mortality fraction can be assumed based on the model results. The important parameters are the total inundation depth of the hotel area and the rising time.

2.9. RISK

In this analysis the risk of flooding in the study area has been quantified using a fault tree to identify the failure mechanisms. The result of this analysis is a return period of failure. This return period will function as a threshold or boundary, where the system should not fail when a flooding event occurs.

In figure L.2 and L.3 in appendix L, two systems that can fail due to flooding have been established. These are the flooding of the Blau Arenal Hotel and the flooding of the waste water treatment plant. These fault trees have been created to analyse the situations that can lead to the failure of both systems. The situations will be used in section 2.11 to model and simulate scenarios that could cause the failure.

To know the boundary of flooding, an altitude level is needed. In figure L.4 in appendix L, the altitudes are given of certain locations. From this map the height of the surface can be obtained and used as a boundary water level during flood.

The values on the map are altitudes of the ground level. This is correct for the hotel, as openings such as doors go all the way to the ground. However, for the waste water treatment plant, the ground level is not exact. As can be seen in appendix C, the first reservoirs of the waste water treatment plant are about 20 cm above ground. This value has to be added to the value of the map to receive the correct boundary for the waste water treatment plant. In table 2.11, the threshold altitudes are shown. If these values are reached during a flood, failure occurs. To know how often this can happen, simulations will be made of different scenarios.

Table 2.11: Altitudes for the threshold water level during a flood

Location	Altitude (m +MSL)
Blau Arenal Hotel	2.12
Wate water treatment plant	2.41

The safety levels for both the waste water treatment plant and hotel are determined independently of each other.

- **Waste water treatment plant flooding**
  Flooding of the waste water treatment plant can cause contamination of the mangrove forest, which is disastrous for the local environment. As a consequence, fewer tourists will visit Blau Arenal Hotel due to the loss of its unique surroundings.

- **Hotel flooding**
  Flooding of the hotel can cause loss of life. However, as described in section 2.8 the probability of loss of life given a flood is assumed to be low. Secondly, flooding can cause a lot of economical damage, depending on the inundation depth. The larger the inundation depth the larger the damage to the hotel.

To calculate the return period of an event that failure could occur to either one of these structures, a design lifetime and chance of failure has to be implemented. This calculation is done with equation 2.3. This equation can be rewritten into equation 2.4 to determine the return period.

$$p = 1 - e^{-f \cdot T_L} \tag{2.3}$$

$$f = -\frac{1}{T_L} \cdot \ln(1-p) \tag{2.4}$$

Where:	f	=	frequency	[Hz]
	T_L	=	design lifetime	[years]
	p	=	probability of failure	[-]

In appendix L, the area has been analysed and classified for the vulnerability to flooding by waves or high river discharge. The result of this analysis is visible in table 2.12.

Table 2.12: Results of the vulnerability analysis of the area

	Flooding by waves	**Flooding by high river discharge**
Vulnerability	33,5 out of 100	38,5 out of 100
Vulnerability level	Low	Medium

These results value the area and help to converge to an acceptable probability of failure. This probability gives a return period in which the system should not fail. The results also show that flooding caused by high river discharge is more dangerous than by high waves. This has to be taken into account with the design and simulation of the problem.

Together with the use of section 2.8, table 2.12 and the information received during the site visit, the return period of both systems has been determined (see table 2.13). The probability of failure of the Blau Arenal Hotel and the waste water treatment plant in a lifetime of 50 years is set to respectively 40% and 20%. The return periods that are used in further calculations are 100 years and 225 years for respectively the Blau Arenal Hotel and the waste water treatment plant.

Table 2.13: Return period of both the Blau Arenal Hotel and the waste water treatment plant

	Blau Arenal Hotel	**Waste water treatment plant**
T_L (years)	50	50
p(-)	0.40	0.20
f(Hz)	0.01022	0.00446
Return period (years)	98	224

2.10. STATISTICAL APPROACH

2.10.1. POSSIBLE RELATIONSHIPS WAVES AND RAIN

In order to set up the different scenarios that will be simulated, the correlation between the rainfall and the storm wave height has to be determined. Precipitation, that results in river discharge, and waves can be fully dependent, independent, mutually exclusive or partly correlated [28]. These four different options are described in appendix M.

2.10.2. RELATIONSHIP WAVES AND RAIN

High river discharge and sea waves are caused by two different phenomena. Wind in combination with a certain fetch produces waves. High river discharge is caused by precipitation in the catchment area. There are no studies available that proof a correlation between high waves and high precipitation. Two extreme events are an example of this:

- In 1982 extreme precipitation, approximately 700 mm in 24 hours [29], caused severe flooding in the Itabo catchment area. During that subtropical storm no high waves where noticed [6].
- Hurricane Wilma caused flooding in Havana in 2005. This hurricane did not produce heavy precipitation, according to *Dr. Ing. Luis Fermín Córdova López* from *CUJAE*.

Based on this consideration, an independence relation between high river discharge and sea waves will be used. This consideration is supported by *Prof. dr. Norberto Marrero de León* from *CUJAE* in an interview on *Thursday 22 September 2016*.

The total return period of one extreme event is 225 year, given a lifetime of 50 years and a probability of failure of 20%, as explained in section 2.9.

2.10.3. INDEPENDENCY OF EXTREME EVENTS

Given the independence relationship, the total probability of simultaneous occurrence of a certain river discharge and storm sea waves is equal to the the product of the two. The probability of occurrence of a certain event, during the lifetime of 50 years, can be calculated with equation 2.3 from chapter 2.9. This can be done for both the waves and the peak river discharge. However, this is not the probability of simultaneous occurrence. The duration of the events, discharge and waves, is assumed to be one day. The total probability, of for example a certain peak discharge, during a specific day is equal to the probability of occurrence during the lifetime divided by the total amount of days (365 multiplied by 50). This probability is very low. Hence, the return period of the second event, for example high waves, should be very short in order to arrive at a simultaneous occurrence. The return period should be equal to multiple times a year. This means in practice that a high river discharge will not be combined with a storm, based on the independence relationship between the two. The high river discharge is combined with normal sea conditions. Storm at sea is combined with mean river discharge conditions.

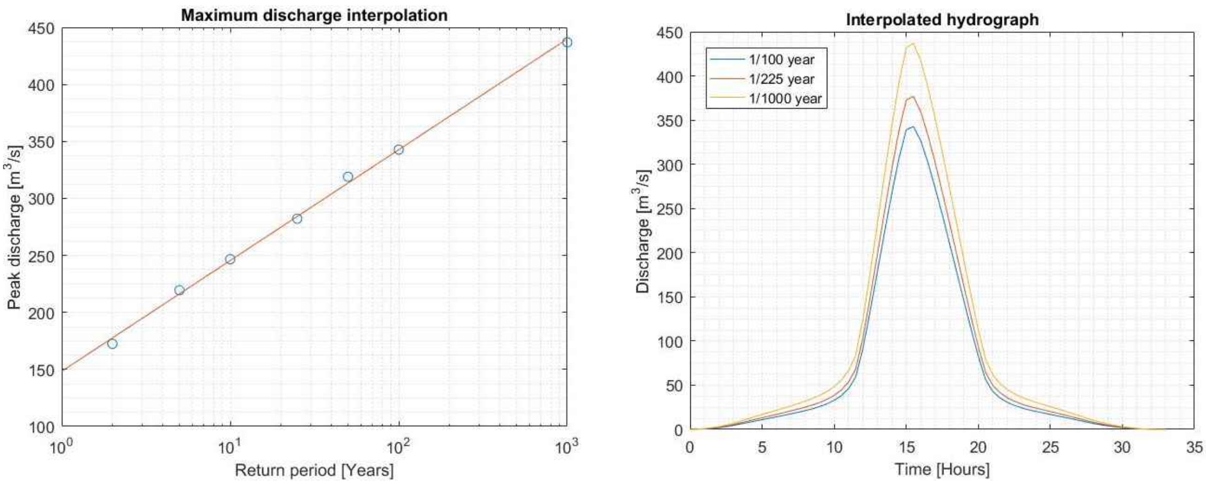
2.10.4. 225 YEAR RETURN PERIOD RIVER DISCHARGE

A return period for extreme conditions of 225 years is chosen, according to section 2.9. This value holds for both scenarios: high river discharge and high sea waves. The calculated relationship between significant wave height and return period is a continuous relationship. However, the return period versus the discharge is only known for discrete return periods (2, 5, 10, 25, 50, 100 and 1000 years). In order to determine the hydrograph for the 225 year condition, the data is interpolated. An exponential relation is assumed between the peak discharge, calculated with the triangular unit hydrograph method, see section 2.6. The relation between the peak discharge and return period can be seen in equation 2.5.

$$Q_p = a \cdot \ln(R_p) + b \tag{2.5}$$

Where:	Q_p	=	Peak river discharge	[m^3/s]
	R_p	=	Return period	[years]
	a	=	Interpolation coefficient: 42.198	
	b	=	Interpolation coefficient: 148.35	

The maximum discharge for the seven different return periods and the interpolation can be seen in figure 2.17a. With this relationship the peak discharge of the 225 year return period event is calculated. The peak discharge amounts 376.9 m^3/s. This interpolation is only done for the peak discharge, it is assumed that the same ratio holds for the whole hydrograph. The 225 year condition is at 36.2% of the difference between the 100 and 1000 year condition.



(a) Interpolation peak discharge (b) Hydrographs 100, 225 and 1000 year condition

Figure 2.17: Graphs showing results of the interpolation

The resulting hydrograph for the 225 year condition in combination with the 100 and 1000 year condition is shown in figure 2.17b.

2.11. SCENARIOS

The scenarios that are going to be simulated are elaborated in this section, see table 2.16 for the entire overview of scenarios.

2.11.1. PARAMETERS

The three parameters that have to be determined for each scenario are the wave conditions, the river discharge and the state of the river mouth.

WAVE CONDITIONS

The wave conditions have specific values for normal conditions, severe storms and extreme storms.

- The normal wave conditions are deduced from the BMT Argoss wave dataset after selecting the right directions (see appendix F). A separation is made between the winter and summer seasons. This is done because the wave conditions in these seasons differ a lot. As already described in chapter 1, the winter season coincides with the storm season. Therefore, the waves will be higher in winter. From the reduced dataset with only wave conditions from summer or winter, the average significant wave height (H_s) is taken. The average wind speed and directions of both waves and wind are also calculated. This results in the wave conditions shown in table 2.14.
- The severe storm conditions, with a return period of 100 years (derived in section 2.9), leading to a significant wave height (H_s) of 7.7 m.
- The extreme storm conditions, with a return period of 225 years (derived in section 2.9), leading to significant wave height of 8.5 m. The values for the severe storm and extreme storm are calculated with the wave distribution from section 2.3.4.

Table 2.14: Offshore wave and wind conditions for different scenario's

Scenario	H_s **(m)**	**Wave direction (°)**	T_p **(s)**	U_{10} **(m/s)**	**Wind direction (°)**
Normal (summer)	0.6	13.2	5.6	4.7	28.7
Normal (winter)	1.1	4.8	5.9	7	31.3
Severe	7.7	354	11.3	14.6	12
Extreme	8.5	354	11.9	14.6	12

RIVER DISCHARGE

For the river discharge three values can be chosen: mean, severe and extreme river discharge. The different values are discussed below and summarised in table 2.15.

- The mean river discharge conditions are based on the rational method calculations from section 2.6. The conservative runoff coefficient C of 0.8 is used. No distinction will be made between summer and winter mean discharge conditions. The mean river discharge is low over the whole year, as can be seen in section 2.6. The difference between the summer and winter conditions is assumed to have no influence on the model. The mean river discharge is 1.2 m^3/s.

- The severe river discharge conditions have a return period of 100 years. The peak discharge and hydrograph is determined based on the triangular unit hydrograph method, see section 2.6. The peak discharge is equal to 342.8 m^3/s. Not all the discharge will go through the main branch, see section 2.7. A reduction of 10% is chosen. This leads to a total peak river discharge of 308.6 m^3/s. The hydrograph, with reduction included, can be seen in figure 2.18.

- The extreme river discharge conditions have a return period of 225 years. The peak discharge and hydrograph is determined based on interpolation of the peak discharges from the available return periods. An exponential distribution is used, see section 2.10. The peak discharge is equal to 376.9 m^3/s. The same reduction factor is applied to the extreme river discharge condition. The final peak river discharge is equal to 339.2 m^3/s. The reduced hydrograph can be seen in figure 2.18.

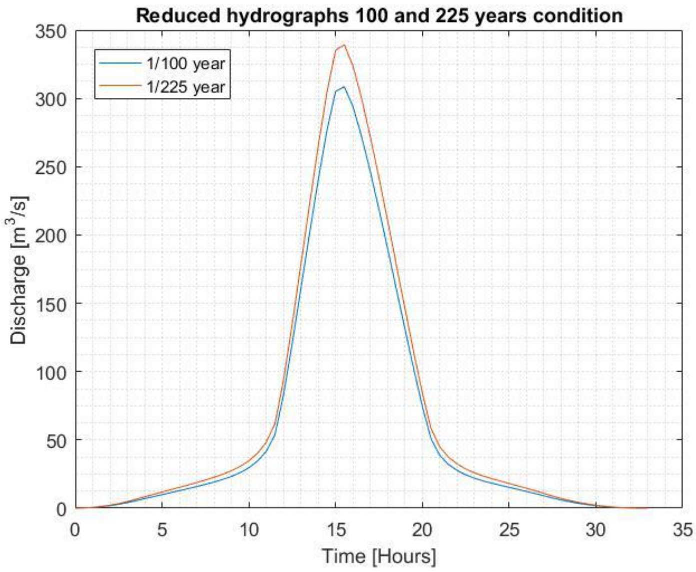


Figure 2.18: Reduced hydrograph 100 and 225 years return period

Table 2.15: Peak river discharge for different scenario's

Scenario	**Peak discharge (m^3/s)**
Normal	1.2
Severe	308.6
Extreme	339.2

STATE OF RIVER MOUTH

Finally, the third parameter is the initial state of the river mouth. The river mouth can be open or closed, as is explained earlier in section 2.5. It is assumed to be open in winter and closed in summer.

2.11.2. 1982 EVENT

The 1982 event, described in section 2.6 is not used as scenario in the analysis. This is mainly due to the fact that the data is insufficient and unreliable. The distribution of the rain is calculated based on the total rain during that day. Furthermore, the peak of the hydrograph is way more than the one in 1000 year condition. The event is therefore out of the study domain, since the solution is designed on one in 225 year conditions.

2.11.3. OVERVIEW

An overview of the scenarios that are going to be simulated can be seen in table 2.16. The first two scenarios that will be simulated are the reference scenarios with normal wave conditions in winter and summer, normal river discharge and an open or closed river mouth. The reference scenarios are needed to check what happens under normal conditions. In this way, a comparison of the results of the reference scenario with the changes to the area of the other scenarios can be done. After that, several scenarios are simulated with different values of the three parameters.

Table 2.16: Overview of the scenarios

Nr.	Scenario	Wave conditions	River discharge	River mouth
1.a	Reference scenario	Normal winter	Normal	Open
1.b	Reference scenario	Normal summer	Normal	Closed
2.a	Rain, open	Normal winter	1/100	Open
2.b	Rain, open	Normal winter	1/225	Open
3.a	Storm, open	1/100	Normal	Open
3.b	Storm, open	1/225	Normal	Open
4.a	Rain, closed	Normal summer	1/100	Closed
4.b	Rain, closed	Normal summer	1/225	Closed
5.a	Storm, closed	1/100	Normal	Closed
5.b	Storm, closed	1/225	Normal	Closed

3

Model setup and results

Two models are used to analyse the several scenarios, determined in section 2.11. To determine the wave transformation from deep water conditions to nearshore conditions, the model SWAN (Simulating WAves Nearshore) is used. Then, XBeach is used to examine the project area. In XBeach the effects of waves on morphology can be analysed and also the river discharge can be implemented. In this chapter the model setup of both of these models will be explained. After that the results of the models will be presented.

3.1. Setup

SWAN will be used to transform waves from deep water conditions to the shore. Because SWAN uses implicit schemes to calculate the parameters of a wave spectrum at the grid points, a coarse grid can be used in deep waters. A finer grid should be used when coming closer to the shore, because of the smaller scale of processes which are imposing influence on the wave field. The output of SWAN, which consists of wave spectra at several locations exactly at the boundaries of the grid of XBeach, will be used as an input for the XBeach model. This XBeach model starts outside of the breaker zone in order to adequately calculate what happens to the sea bed in the breaker zone. Sedimentation and erosion can be examined in XBeach, as well as the water levels during flooding, caused by a peak river discharge. First the setup of SWAN will be treated, and consequently the setup of the XBeach model.

3.1.1. SWAN

SWAN is used to compute the wave transformation from deep water conditions to shallow water conditions, based on a wave action balance. The model does not solve for individual waves, but it transforms an offshore wave spectrum to a wave spectrum nearshore. A 2D shore model is made for the coast of the Blau Arenal Hotel area. The model setup is discussed quickly here. For a more thorough discussion on the model input, see appendix N.

GRID AND BATHYMETRY

The bathymetry is created using the data from section 2.4.3. The coordinates of the data are translated from WGS84 to meters. The south-west corner of the data is set as (0,0) point. The domain of the study area is from 92.0 km to 92.6 km. Adding some more than 25 km on both sides results in a domain [58900; 133900] (in meters). This is done because of the influence of the shadow zones. This phenomenon is elaborated on in appendix N.

A nested approach was used to get a good resolution at the project area, without having to use a very fine grid in the deep sea and the shadow zones. This nested approach consists of three grids:

- Coarse grid, covering the whole area: x (58901.04, 133901.04) and y (11441.88, 44941.88) with a grid size of 500 x 500 m.
- Nest 1, covering 10 x 10 km around the project area: x (87300, 97300) and y (19740, 29740) with a grid size of 100 x 100 m.
- Nest 2, covering 4.62 x 4 km around the project area: x (90000, 94620) and y (19740, 23740) with a grid size of 20 x 20 m.

The bathymetry of the entire area with the most coarse grid can be seen in figure 3.1.

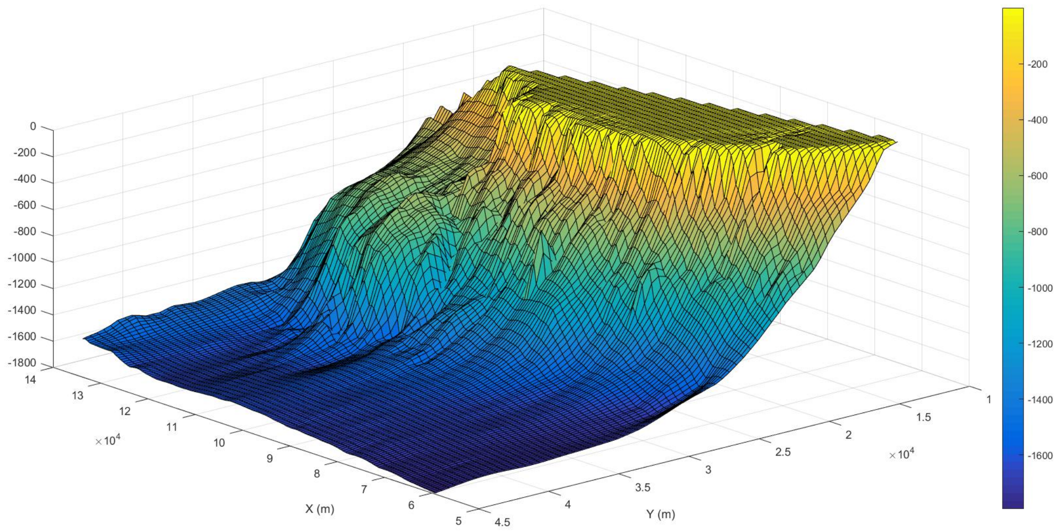


Figure 3.1: Bathymetry of the coarse grid

BOUNDARY CONDITIONS

For the shape of the wave spectrum the JONSWAP shape is chosen. This spectrum shape is useful for young sea states and for storms and hurricanes. Average values for the JONSWAP spectrum can be deduced, which gives a $\gamma = 3.3$ [30]. The wave spectrum should be imposed on the western, the northern and the eastern boundary. The H_s, T_p and direction of the waves are obtained from section 2.11, in table 2.16.

OTHER INPUT

The water level for summer and winter conditions is set to mean sea level, as no storm conditions are simulated during these conditions and waves are not expected to be the bottleneck for such conditions. For severe and extreme conditions, the tidal amplitude (0.25 m) is included (see section 2.3.2), as well as a storm surge of 0.4 m (see section 2.3.5). Also a sea level rise during 50 years is accounted for (0.25 m, see section 2.3.6), because this is the lifetime of the structure. This results in a total water level of MSL +0.9 m.

Wind parameters are obtained from section 2.11, in table 2.16. Quadruplet wave-wave interactions, whitecapping, triad wave-wave interactions, depth-induced wave breaking and setup are included with default parameters. The default value for the JONSWAP bottom friction is 0.067 m^2s^{-3}. However, for a smooth seafloor, like the Gulf of Mexico, a lower value of 0.019 m^2s^{-3} is advised [31].

OUTPUT

The output of the two largest grids consists of a grid of points with wave spectrum input for the following (nested) grid. The output of the smallest and finest grid consists of five output points exactly at the boundaries of the XBeach model. Three points on the northern boundary, and one point on the east and west boundary are given as output.

3.1.2. XBEACH

XBeach is a process-based model, used to simulate hydrodynamic and morphological processes. For this project, XBeach is used to simulate the possible flooding due to extreme rain- and storm events. The entire study area is put into the model, together with the river, dunes and mangroves. The model setup of the XBeach model is described extensively in appendix O.

GRID AND BATHYMETRY

The grid and the bathymetry that are used in the model can be seen in figure 3.2. In appendix O they are shown separately. The grid consists of 215 x 179 cells, with a maximum cell size of 20 x 20 m (in the top right corner of the grid) and a minimum cell size of 2 x 2 m (in the mouth of the river). The largest part of the grid consists of cells that are 4 x 4 m (in the lagoon, at the hotel and at the coastline). The total dimensions of the area that is put in the model are 620 m by 1100 m.

The bathymetry is created using the data from section 2.4.3. The deepest point is located at the offshore boundary and is approximately 10 meters deep. All the values of the bathymetry data points are with reference to mean sea level. The coordinates of the data are translated from WGS84 to meters. The south-west corner of the data is set as (0,0) point. The area of computation is located between 92 km and 92.6 km in y-direction (parallel to the coast) and between 19.74 and 20.84 in x-direction (perpendicular to the coast).

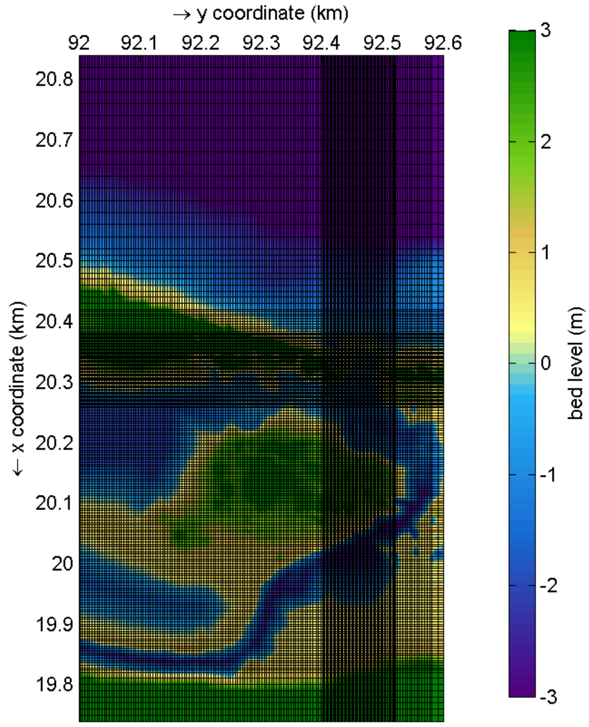


Figure 3.2: The computational grid and the bathymetry of the model

BOUNDARY CONDITIONS

As boundary condition at the offshore boundary of the domain, the wave spectra from the SWAN computations (described in appendix N) are imposed. Also, in the deeper part of the west and east boundary, a wave spectrum from SWAN is imposed. Together, these boundaries form the wave field. The west and east boundaries are defined as Neumann boundaries, which state that there is no change in surface elevation and velocity locally [32]. The north and south boundaries are defined as flow boundaries of the type absorbing-generating (weakly-reflective). This means that waves and flow are able to pass through the boundary.

OTHER INPUT

Besides the grid, bathymetry and boundary conditions, other phenomena are included in the model. Morphology is modelled with a morphological factor of 10. This means that when 6 minutes of hydrodynamic time is simulated, one hour of morphological evolution is simulated effectively. According to *Dr. Ing. Luis Fermín Córdova López* from *CUJAE*, not the full domain is erodible. The area that can be eroded is shown in figure O.2a in appendix O. The thickness of the erodible layer is set to 2 meters.

The river discharge of the Itabo River is put in the model. The river inflow in the domain is located near the lower left corner. A discharge point is added to simulate the river in the model.

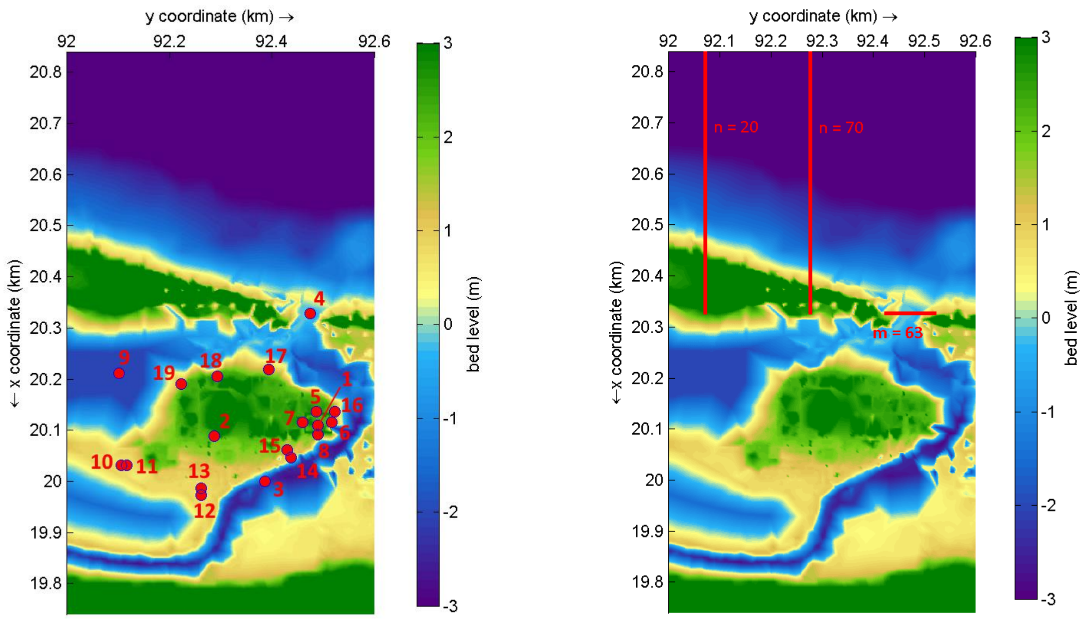
The tide, sea level rise and storm surge are derived in section 2.3. The values that follow from the derivation are put in the model:a tidal range of 0.5 m, sea level rise of 0.25 m and a storm surge of 1.56 m. The storm surge is larger in the XBeach model than it was in the SWAN model to account for wind and wave set up. These are generated itself in SWAN by using the wind input and by turning on the option wave induced setup (see section N.1 in appendix N).

The mangroves in the area are included in the model by giving different values for the bottom friction to locations where mangroves are present. The exact locations can be seen in appendix O. The Chézy value for the bottom friction of the mangroves is 15 $m^{1/2}/s$. The areas without mangroves have a value of 55 $m^{1/2}/s$ [22].

The simulation time of the model depends on the length of the hydrographs for the river discharge. The periods of high discharge last for 33 hours (see the hydrograph in section 2.11). Therefore, the model has to be run for at least 33 hours. In order to see the development of the domain after a period of high discharge, the total duration of one run is set to 45 hours. With the morphological factor of 10, a simulation time of 4.5 hours is used in the model. No extra spin-up time is used, as both the hydrograph and the wave field take time to fully develop. Therefore, the model results should always be analysed after a simulation time of 1 hour.

OUTPUT

The output given by the XBeach model can be visualised in three ways. Graphs can be made of a certain parameter at a certain location. The locations of the output points are shown in figure 3.3a. Multiple points directly around the Blau Arenal Hotel and the waste water treatment plant are chosen as output points, to compare the different locations. Another kind of output are cross-sections of a certain section of the domain. This can be used to look at the development of the dunes. Figure 3.3b shows the location of three cross-sections that are used in section 3.2.2. The third way is to show a top view of (a part of) the domain. The bed level image in the two figures in figure 3.3 is an example of this. Colourbars are used to visualise the value of a certain parameter at a certain location. Parameters that are selected to be outputted by the model are the bed level, water level, Eulerian velocity, significant wave height and sedimentation and erosion values.



(a) Locations of the points

(b) Locations of cross-sections

Figure 3.3: Locations of output generation

3.2. RESULTS

In this section the results of the model runs in both SWAN and XBeach are described.

3.2.1. SWAN

For the results of the four scenarios, see appendix N. In this section the interpretation of these results is discussed.

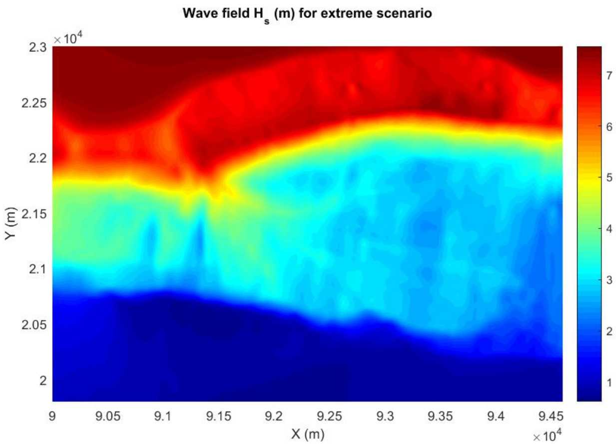


Figure 3.4: H_s field for the smallest nested grid during extreme conditions (scale is in meters)

In figure 3.4 the smallest nested grid can be seen for the extreme situation. The coastline lies in between $y = 2.05 \cdot 10^4$ m and $y = 2.10 \cdot 10^4$ m. The remarkable thing in this figure is that close to the shore, at 10 m depth, the significant wave height does not exceed three meter, which normally should be at least 5 meter. This can be explained by examining the bottom topography more seawards.

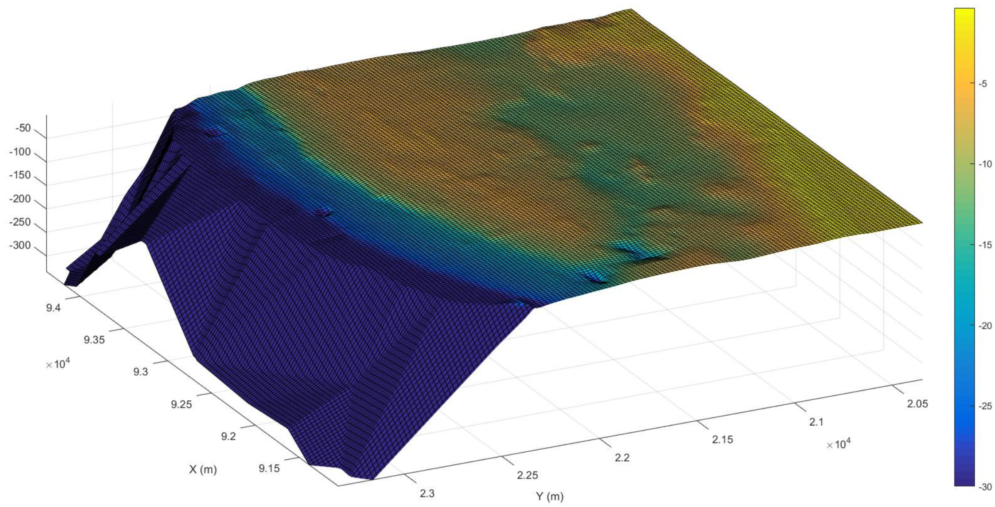
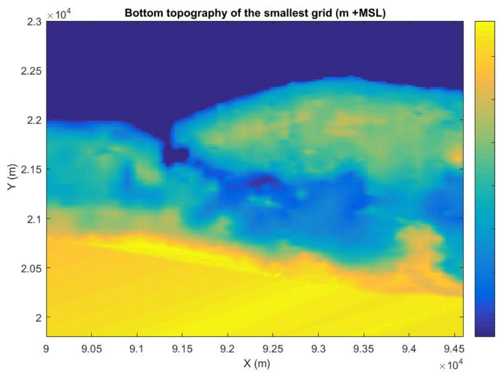
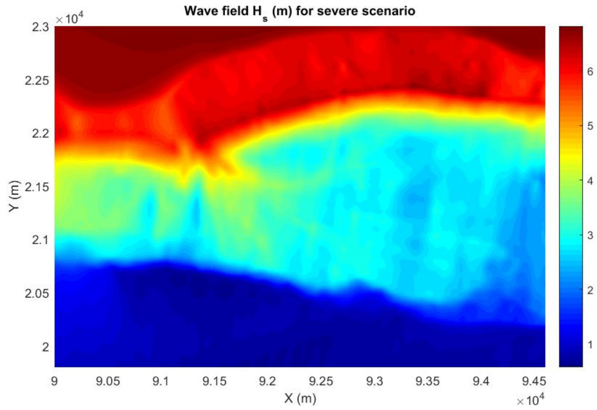


Figure 3.5: Bathymetry of the smallest nested grid, with the shoal offshore (scale is in meters + MSL)

In figure 3.5 a shallow part of the seabed can be observed (in between $y = 2.25 \cdot 10^4$ m and $y = 2.15 \cdot 10^4$ m), after which the depth grows again towards the shore. On this shoal the depth is only five meter. This explains the low wave heights nearshore during extreme wave conditions. When comparing figure 3.6a to figure 3.6b this is confirmed. At the location where the shoal begins, the wave height rapidly decreases. This decrease in wave height happens for severe conditions as well. In figure 3.7 this is shown in more detail by plotting the H_s (= H_{m0}) and the dissipation of wave energy with the bottom level for a cross-section in the middle of the domain. In this figure the large part of the decrease in wave height and of the dissipation of wave energy can be found at the shoal, 1.5 km offshore. The bathymetry used to make the bottom grid for these model runs was measured in 1988. However, the data obtained by GeoCuba in August 2016 shows the same shoal. This, together with the large distance offshore (1.5 km), pledges for a non-sandy material which is fixed at this location. For this reason, it is not very plausible that waves will get a large problem in the near future. However, further research on the shoal is recommended to exclude the possibility of disappearance of the shoal.



(a) Bottom topography (scale is in meters + MSL)



(b) Wave field (scale is in meters)

Figure 3.6: Comparison of the bathymetry and the wave field

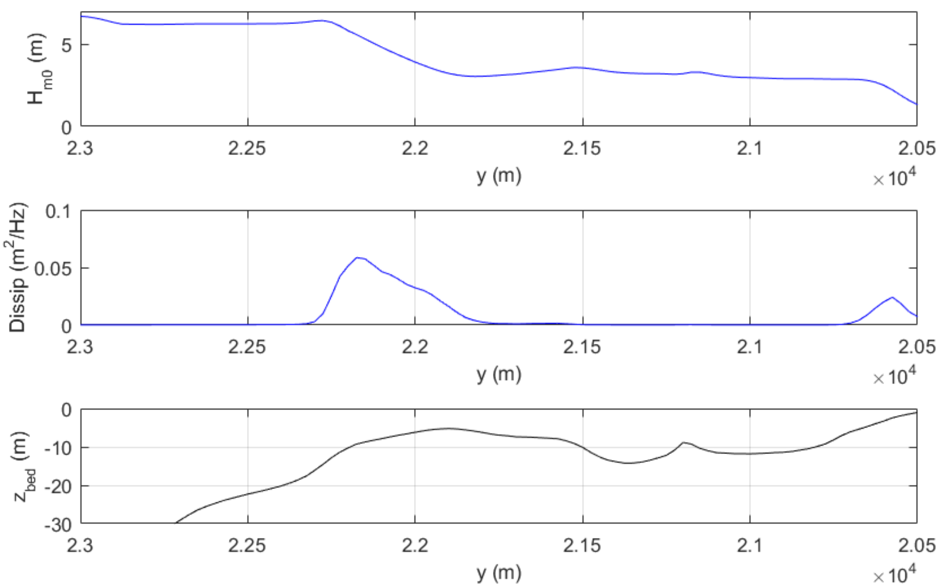


Figure 3.7: Effect of the shoal on the wave height and wave dissipation at cross-section x = 92,300

3.2.2. XBEACH

The results of the runs with XBeach are described in this section. All scenarios from table 2.16 are run in XBeach. The results are described below. Table 3.1 shows the results of the runs. It indicates the maximum water level in the river during the simulation and whether flooding occurred at the hotel and the waste water treatment plant (WWTP).

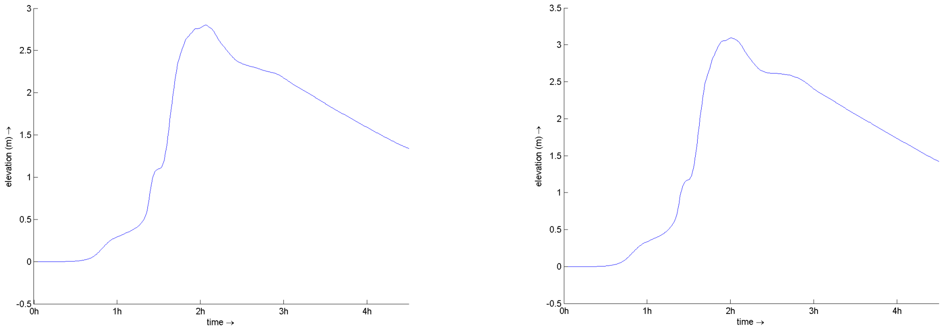
Table 3.1: Results of the scenarios

Nr.	**Wave conditions**	**River discharge**	**River mouth**	**Max water level m +MSL**	**Flooding hotel**	**Flooding WWTP**
1a	Winter	Normal	Open	0.25	No	No
1b	Summer	Normal	Closed	0.18	No	No
2a	Winter	1/100	Open	2.80	63 cm	30 cm
2b	Winter	1/225	Open	3.09	95 cm	49 cm
3a	1/100	Normal	Open	1.98	No	No
3b	1/225	Normal	Open	1.95	No	No
4a	Summer	1/100	Closed	2.80	63 cm	30 cm
4b	Summer	1/225	Closed	3.09	95 cm	49 cm
5a	1/100	Normal	Closed	1.98	No	No
5b	1/225	Normal	Closed	1.95	No	No

FLOODING

In table 3.1 is shown for each scenario whether flooding occurs or not. It can be seen that flooding occurs in four of the ten scenarios. In all these scenarios flooding is caused by high river discharges due to heavy rainfall. Discharges with a return period of 100 years cause flooding of both the hotel and the waste water treatment plant. From the results can be deduced that in scenarios with high waves and a normal river discharge no flooding occurs. Despite the significant rise of the water level in the river due to the storm surge and sea level rise, still no flooding is caused.

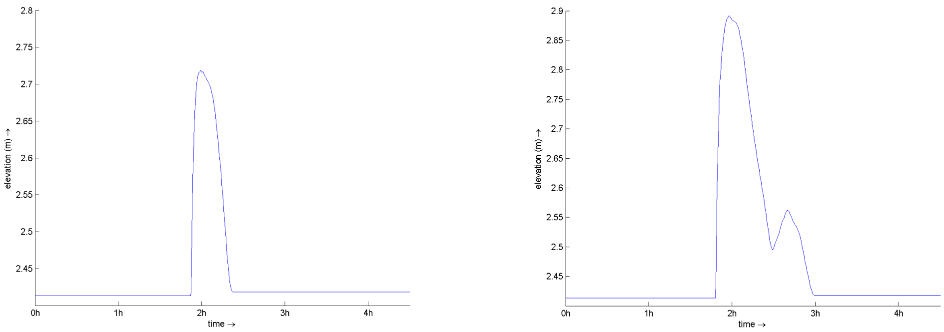
Figures 3.8, 3.9 and 3.10 show the water level elevation in m +MSL of the three output points (see appendix O) of scenario 2a and 2b. These scenarios have respectively a 100 year and a 225 year return period river discharge. In figure 3.8 it can be seen that the elevation in the river has the same shape for both scenarios. Of course, for the scenario with the highest return period, the water level during the peak discharge is higher than for the lower return period.



(a) Scenario 2a (b) Scenario 2b

Figure 3.8: Water level elevations in the river during a flooding period

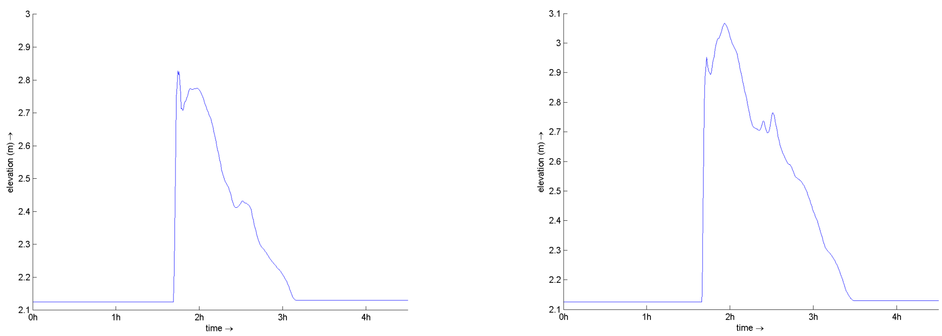
The results of the output point at the waste water treatment plant are shown in figure 3.9. The shape of the graphs look quite similar. The only difference is that the elevation has two peaks in scenario 2b, while it has only one peak in scenario 2a. This second peak is probably due to the way the bathymetry steers the higher river discharge. The level of flooding is 30 cm in scenario 2a and 49 cm in scenario 2b. The waste water treatment plant is inundated for respectively 5 hours (30 minutes simulation time) and 11.3 hours (68 minutes simulation time).



(a) Scenario 2a (b) Scenario 2b

Figure 3.9: Water level elevations at the waste water treatment plant during a flooding period

When looking at figure 3.10, the water level elevation at the hotel can be seen. The graphs look similar, but the peaks are different. Scenario 2a has a water level elevation of 63 cm and a duration of 14.2 hours (85 minutes in the model). Scenario 2b has a water level elevation of 95 cm and a duration of 17.5 hours (105 minutes in the model).



(a) Scenario 2a (b) Scenario 2b

Figure 3.10: Water level elevations at the Blau Arenal Hotel during a flooding period

Figure 3.11 shows a time series of the flooding of the hotel during scenario 4b. From the images can be deduced that the flooding happens with a flood wave from the river, because the model does not have time to flood the complete area. The area behind the Blau Arenal Hotel floods a little while later

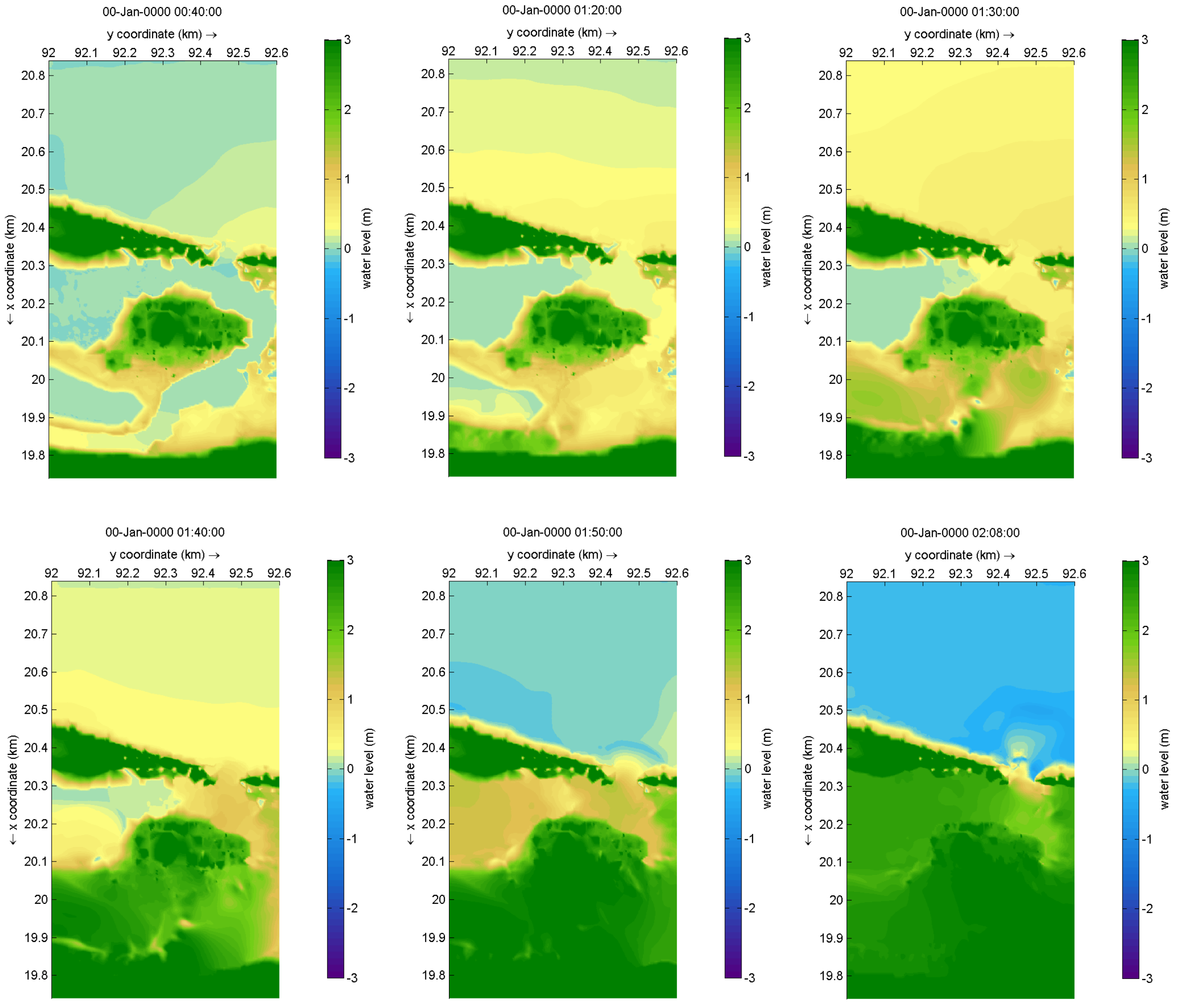


Figure 3.11: Time series of the flooding during scenario 4b

SEDIMENTATION AND EROSION

DEVELOPMENT OF THE COASTAL SHOREFACE PROFILE

When looking at sedimentation and erosion, it is important to see the behaviour of the shoreface during a storm. This can be analysed by looking at the results of scenario 3b, in which storm conditions with a return period of 1/225 years are modelled. In figure O.3b in appendix O the locations considered can be seen. The shoreface profiles before and after the storm with a duration of 45 hours, are shown in figure 3.12. This is the situation at location n = 20. The mean sea water level and the storm surge water level are included. As can be seen in figure 3.12, sand is moving offshore during the storm surge. The total amount of sand in the cross-section does not differ too much in both profiles. This dynamic equilibrium profile behaviour is in accordance with Bruuns rule [9]. The water depth relates to the mean waterline up to a certain depth.

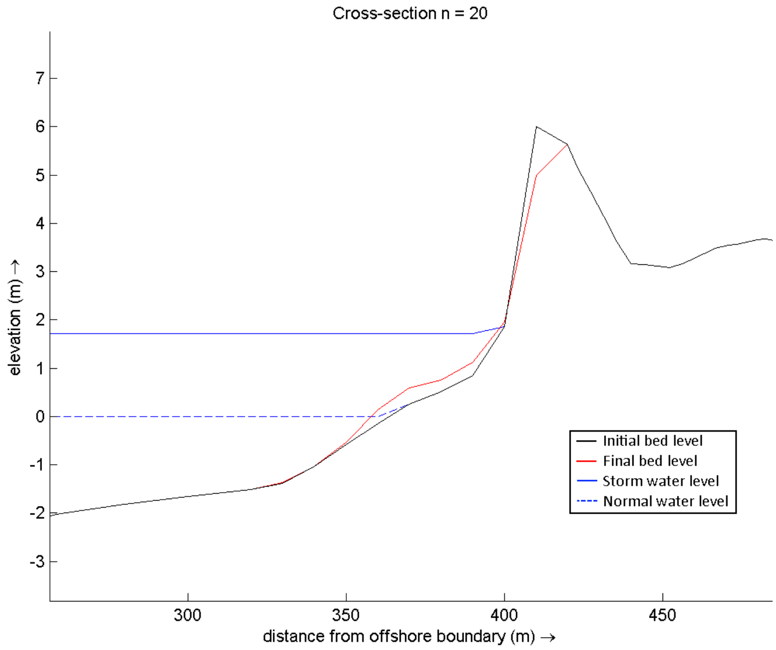


Figure 3.12: Cross-shore profile of cross section n = 20

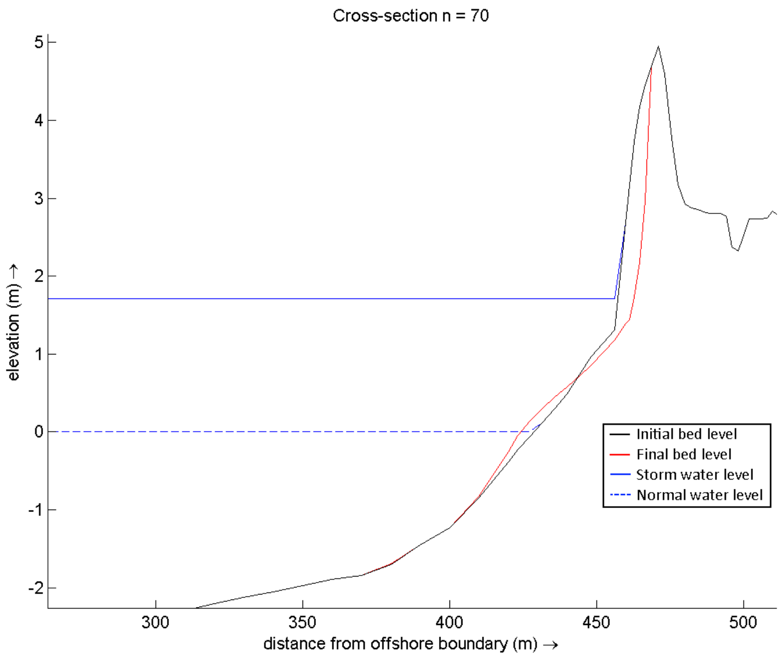
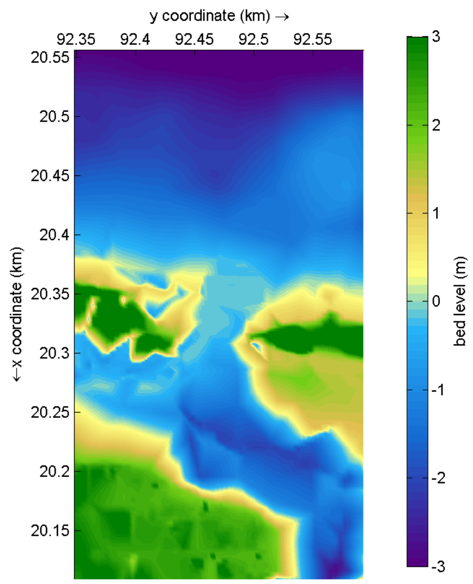


Figure 3.13: Cross-shore profile of cross section n = 70

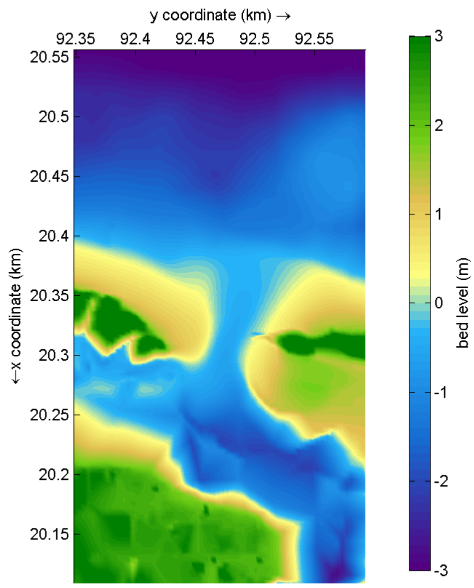
When looking at cross-section n = 70, a different behaviour can be observed, as can be seen in figure 3.13. First of all, the shoreface profile is not yet in its equilibrium state, because it has not fully adapted to the storm surge level. Also, the amount of sand in the shoreface profile is decreasing during the storm. Furthermore, it can be seen in figure 3.14 that a lot of sedimentation takes place around the river mouth. This could explain the losses of sediment in cross-section n = 70. However, the storm is coming from the NNE and the net currents are to the East, which would mean that the cross-sections west of the river mouth should not decrease. Besides, the west side of the river mouth is growing faster, which indicates a sediment transport to the East. An explanation of the losses in this cross-section could be the fact that the difference between the wave angle of incidence and the normal of the shore gets slightly bigger around cross-section n = 70, implying a larger outwards transport than inwards.

All these beach erosion observations are due to a storm event or seasonal effects, caused by a difference in water level. When regarding figure 3.13 it is hard to tell whether the loss in sediment in the cross-section is due to episodic erosion or due to structural erosion. Besides, this location is close to the river mouth, and effects induced by the model may also play a larger role over here.

However, one aspect that can be taken into account is the fact that the shoal, discussed in section 3.2.1, reduces the height of the waves coming to the shore significantly. This results in a low input of wave energy compared to the coastline west and east of the project area, resulting in a lower sediment transport and thus less erosion.



(a) Initial bed level in the river mouth area

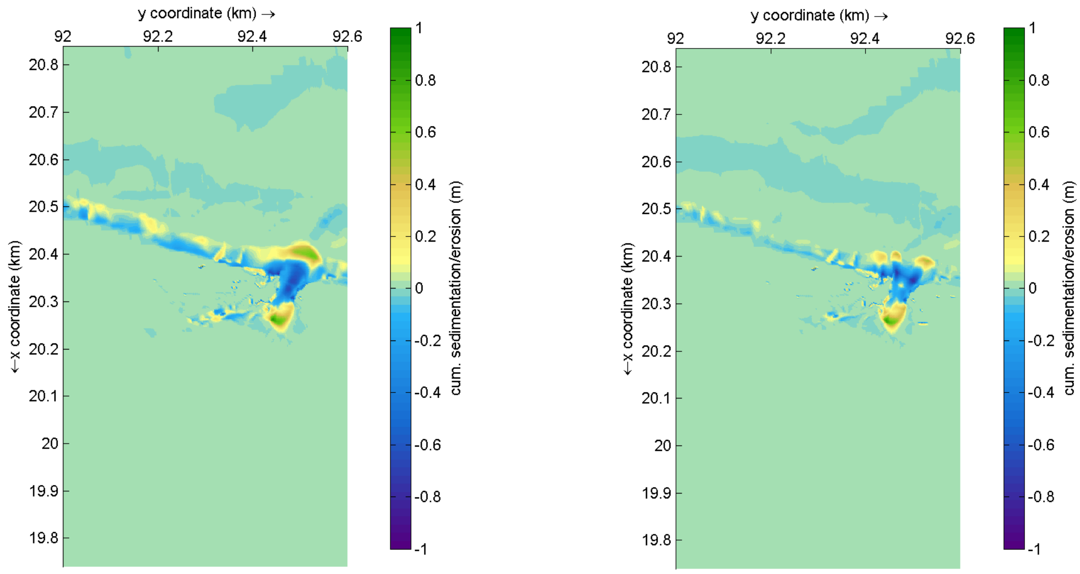


(b) Resulting bed level in the river mouth area

Figure 3.14: Sedimentation at the river mouth, scenario 3b

SEDIMENTATION AND EROSION OF THE RIVER MOUTH

As mentioned previously, sedimentation takes place in the river mouth. The closure of the river mouth is a typical phenomenon of wave forcing dominating the forcing caused by the river discharge. Therefore, this phenomenon can be seen for all scenarios with a normal river discharge, which is quite low compared to the wave forcing. However, the development of the ebb tidal delta differs, depending on the wave forcing. Next to wave- and river forcing, a third phenomenon, the tidal forcing, is playing a role [9]. The tide dominating delta is characterized by strong tidal currents creating tidal channels. When comparing scenario 1a (winter) to scenario 1b (summer), the effect of tides can be seen. During the winter conditions wave heights are larger, resulting in an offshore sediment transport in the cross-shore direction, as can be seen when comparing figure 3.15a to figure 3.15b.

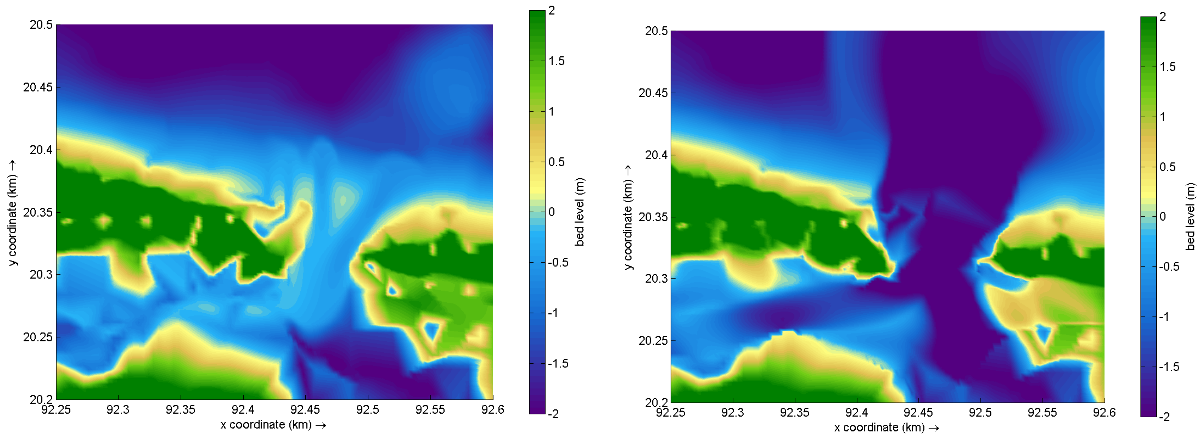


(a) Winter conditions (scenario 1a)

(b) Summer conditions (scenario 1b)

Figure 3.15: Sedimentation and erosion during winter and summer conditions

In figure 3.15a this larger sediment transport offshore can be clearly observed. Also, the formation of an ebb-tidal delta and a flood-tidal delta can be seen. The behaviour during summer conditions is different. In figure 3.15b it can be seen that tidal channels are formed. Sediment deposition happens at the end of the tidal channels. It should be pointed out that during this test with summer conditions the river mouth was closed at the start, while during the winter test it was not. However, when looking to the west of the main river mouth also a tidal channel is formed (see figure 3.16a). Therefore, this effect is not solely caused by the presence of the spit. Besides that, less sediment transport offshore at the straight coast can be observed in figure 3.15b.



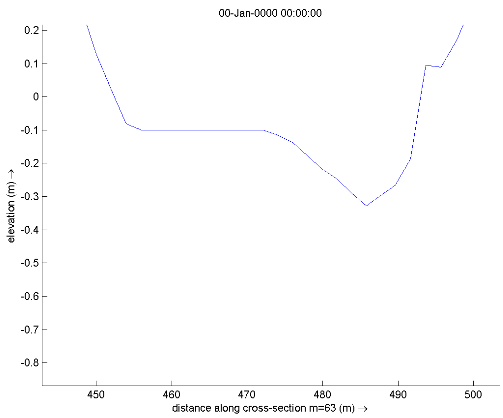
(a) Low river discharge (scenario 1b)

(b) High river discharge (scenario 4b)

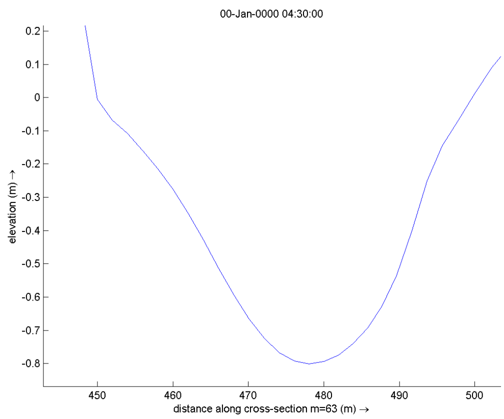
Figure 3.16: Bed level during summer conditions after the simulation

TIDAL INLET

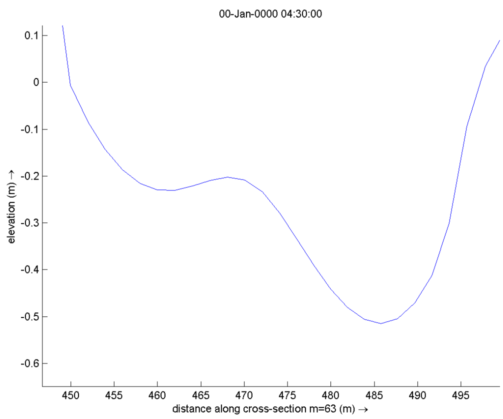
As explained in section 2.5 and appendix A, the mouth of the Itabo River is very dynamic and can close or open for different hydrodynamic conditions. In appendix I, an Escoffier curve is constructed for the Itabo River. With this curve the stability of the inlet can be determined. To analyse the morphological stability of the river mouth, a cross-section has been taken before and after scenarios that have been modelled. Figure 3.17 shows cross-sections at m = 63, which has the best depiction of the river mouth. In the graphs, an elevation of 0 m represents mean sea level, which is the water level when the simulation starts. With this elevation a cross-sectional area can be calculated and implemented in the Escoffier curve.



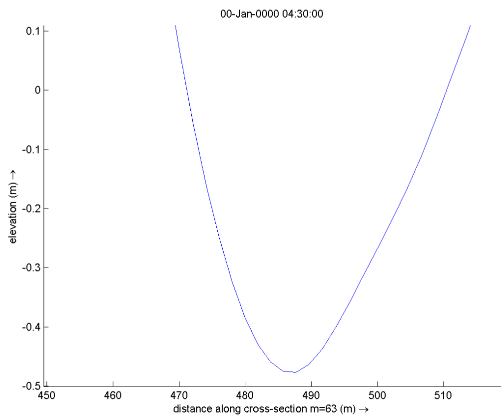
(a) Initial river mouth for scenarios 1a, 1b and 3a



(b) State of the river mouth after winter conditions (scenario 1a)



(c) State of the river mouth after summer conditions (scenario 1b)



(d) State of the river mouth after 100 year return period conditions (scenario 3a)

Figure 3.17: Bed level during summer conditions after the simulation

As can be seen in figure 3.18, the three situations are applied to Escoffier. As explained in appendix I, the curve has two important points. One of them is stable and the other is unstable. The right intersection point of the red and blue line is the stable point and inlet types will most likely converge if close to this point. This can be confirmed with results of the model.

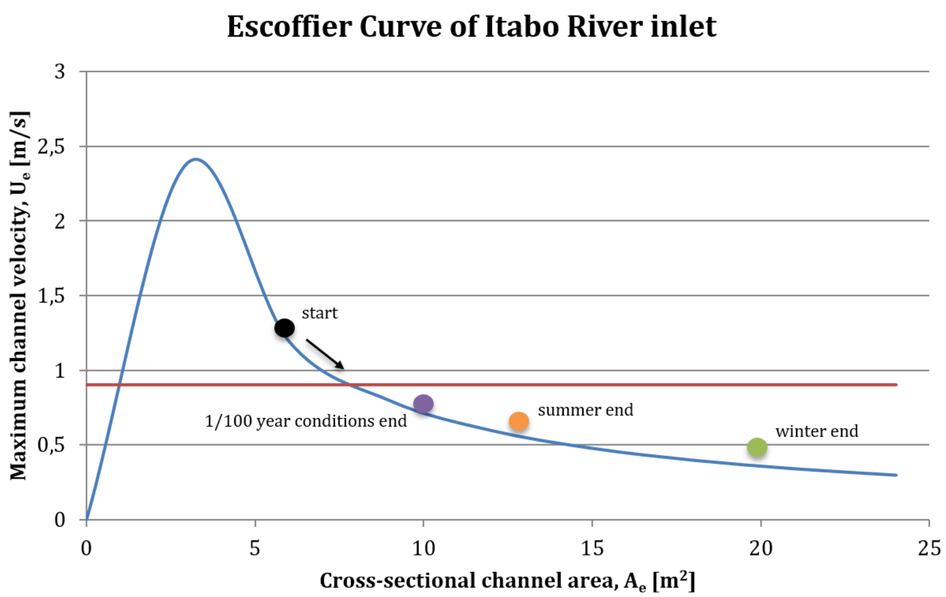


Figure 3.18: Escoffier curve of the river mouth, including the model results

The scenarios that are chosen for this analysis only contain relatively mild wave conditions and no extreme river discharge. This is done, because such extreme events cannot be used in the analysis of the stability of the river mouth, as it would not portray reality. During normal conditions the tidal wave has the largest forcing, which produces the best result for this analysis. The maximum channel velocities have also been analysed in the model. They exceed the velocity of the curve slightly in all scenarios by a maximum of 0.2 m/s. This is within the boundaries, as such an Escoffier curve is always a rough estimation. Also the curve is based on measured values. Influence of the model equilibrium, which is not exactly the same as the reality, will cause the modelled points to slightly deviate from the calculated curve. The overall trend is clearly visible. All scenarios have cross-sectional areas which are too large to maintain. Therefore, they will start to close up again. This will eventually result in a cross-sectional area close to the starting position, confirming the dynamic changes and stability of the Itabo River inlet.

CONCLUSION

An important result from XBeach is that both the waste water treatment plant and the hotel are flooded during a 1/100 year river discharge. The hotel is required not to flood for this return period, and the waste water treatment plant should not even suffer from flooding during a 1/225 year rainfall. This implies the urgency to come up with a solution that prevents flooding of this area.

Waves appeared not to be relevant as a flooding cause, as was already expected after the SWAN analysis (see appendix N). However, the absence of waves has a large influence on the development of the river mouth, and the processes induced by tide and river discharge could be clearly observed. These processes, together with the differences in summer and winter shoreface profiles, are aligned with the theory. However, the creation of the spit could not properly be modelled. This is mainly because the small size of the modelled area and with the absence of a sediment inflow at the boundaries. The absence of this inflow of sediment is due to the lack of information about the alongshore sediment transport. Besides, the simulation time is too short to model these processes.

4

SYNTHESIS

4.1. INTRODUCTION

On the *26th of September 2016* a brainstorm was done by the project group to come up with solutions to prevent flooding, caused by waves or high river discharge, of the Blau Arenal Hotel and the waste water treatment plant. The solutions can be divided into three groups: solutions that protect against flooding due to waves, solutions that protect against flooding due to river discharge and solutions that protect against both flooding causes. The results of the brainstorm are presented in appendix P.

The brainstorm was done before the model results were fully analysed. The model results showed that the flooding problem at the Blau Arenal Hotel area is mainly caused by high river discharge (see section 3.2.2). Storms also produce storm surge, which will lead to higher water levels in the area. However, this will not result in flooding. Furthermore, as described in section 2.10, a combination of high river discharge and storm is not considered, since the return period of simultaneous occurrence is out of the domain of interest. Based on the model results it is concluded that the solutions that prevent flooding due to waves, presented in appendix P, are not relevant to reduce flooding at the Blau Arenal Hotel area.

The measures to prevent flooding due to high river discharge are discussed in this paragraph. The solutions are described in more detail in appendix P. Not all the solutions are worked out in a detailed level. Three brainstorm solutions are directly considered not feasible nor relevant for solving the flooding problems. This solutions are listed below:

- **Upstream dam**
  The dam upstream is considered to be too expensive, especially because the mean discharge of the Itabo river is very low (see section 2.6). The additional benefit of electricity generation is therefore small.

- **Widening and channelling the river mouth**
  This measure is not used as solution, because channelling should take place over a larger distance to be effective. Flooding of the Blau Arenal Hotel area takes place further upstream, according to the model results, see section 3.2.2, figure 3.11. Furthermore, the environment will be damaged with this measure.

- **Relocation of the waste water treatment plant**
  Relocation of the waste water treatment plant is an effective and maybe cheap solution, provided that a good location is available. The waste water treatment plant should be in the neighbourhood of the hotel to work properly, otherwise the waste water is transported over a too large distance. The waste water treatment plant is already situated on the highest part of the Blau Arenal Hotel area. This is the best location to prevent flooding of the waste water treatment plant. Because of this reason, the relocation of the waste water treatment plant is not worked out as a final solution.

Two brainstorm solutions are analysed in more detail. After this analysis the conclusion was drawn that it is not the right way of solving the flooding problem. These solutions are:

- **Extra outflow point**
  The additional outflow point is investigated, see appendix T. This measure is not used as solution to prevent flooding. A weak spot in the flood defence is created by building an extra outflow point and the additional risk coming with this weak spot is a big drawback of this solution. Furthermore, strict maintenance is needed in order to guarantee the effectiveness of the measure, since the culverts can get clogged.

- **Extension of the mangrove forest**
  The extension of the mangrove forest is analysed, see appendix S. This solution does not solve the flooding problem at the waste water treatment plant or at the hotel. Flooding of the river bank already takes place further upstream from this new mangrove forest, which would function as a flood retention basin. The hotel and the waste water treatment plant would flood first of all, and then the water would flow into the new mangrove forest. Furthermore, mangrove forests need water to survive. This water layer will reduce the available effective storage capacity.

Three categories of feasible flood protection solutions are developed. The first solution is to construct a wall structure around the waste water treatment plant or around both the waste water treatment plant and hotel, that retains the water. The second measure is to construct a flood retention basin upstream from the mouth of the Itabo River. This retention basin will store a part of the water temporarily, which will reduce the peak river discharge. The third solution is to create a dike structure in the South of the hotel area, to retain the incoming flood wave. These three solutions are developed in detail in the following section.

4.2. WALL STRUCTURE

In this section an elaboration is given on the location of the wall solution and its four different executions to fulfil the requirement to protect the area against flooding.

4.2.1. LOCATION

The wall structure can be built around the waste water treatment plant (WWTP) only, shown in figure 4.1a, or around both the waste water treatment plant and the Blau Arenal Hotel (see figure 4.1b). When the wall is built around the waste water treatment plant solely, the length of the structure is shorter due to the significant decrease in the area that is protected against flooding. The length of the wall structure around the waste water treatment plant only is approximately 175 m and around both the Blau Arenal Hotel and the waste water treatment plant is around 980 m (measured with Google Earth [5]). Therefore, the solution around the waste water treatment plant solely has lower investment costs and the construction time is shorter. On the other hand, the wall only prevents contamination of the mangrove forest, while the variant of a wall that protects both the Blau Arenal Hotel and the waste water treatment plant against flooding. Despite the higher costs and larger length of this wall structure, next to the protection of the mangrove forest against contamination, also the safety of the tourists and the employees is ensured.

(a) Wall structure around the WWTP

(b) Wall structure around the Blau Arenal Hotel and the WWTP

Figure 4.1: Satellite images with the visualisations of the location of the wall structures, satellite image from 02-10-2009 [5]

4.2.2. DRAINAGE

It should be taken into account that the solution of a wall structure around the hotel area causes a problem with the drainage of water. Normally, in case of heavy rainfall, the water can flow into the lagoon and the river. Due to the wall, which surrounds the area to protect against flooding, this drainage is not possible. This leads to water inconvenience for the Blau Arenal Hotel, like puddles at the parking lots and other low lying areas. In more extreme cases, the buildings with rooms at the ground floor can even get flooded. This drainage problem is a disadvantage that should be dealt with if the wall structure is chosen to work out as a final solution.

4.2.3. FOUR OPTIONS

The four options of the wall structure are explained here, including their advantages and disadvantages. The elaboration on their design can be found in appendix Q.

GRAVITY WALL

The gravity wall is the first option of the wall structure to prevent the area from flooding (see figure 4.2). The gravity wall uses its own concrete mass solely to retain the water.

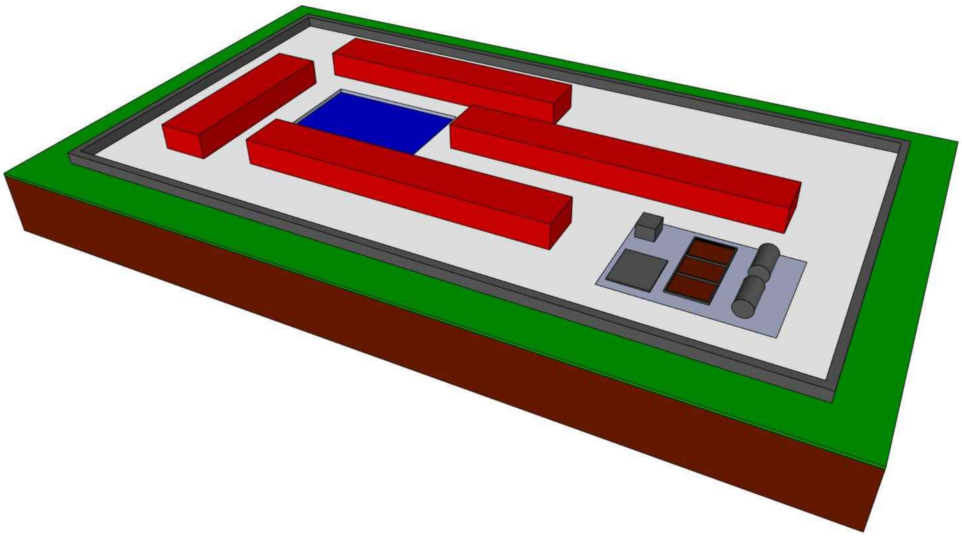


Figure 4.2: 3D schematisation of the gravity wall around the Blau Arenal Hotel and the waste water treatment plant

ADVANTAGES

- The solution is cheap, as it has low investment costs.
- The gravity wall is easy to construct, as it is a simple structure of concrete and reinforcement with low complexity.
- The solution is aesthetic, but only when the wall is built around the waste water treatment plant. In this way, the view of this plant is more or less blocked by the wall.

DISADVANTAGES

- Piping, the flow of water underneath the structure, can be a problem.
- The gravity wall can be prone to settlements.
- The solution is not aesthetic when it surrounds the Blau Arenal Hotel, because it is an unpleasant structure to look at for the tourists. They prefer to see the mangrove forest.
- The gravity wall has a large width, as only its own mass is used to retain the water.

EMBEDDED WALL

The second option is the embedded wall in the soil, as can be seen in figure 4.3. The embedded wall uses the passive soil to increase the resistance of the structure against the water. With this solution seepage will not occur, because the wall is embedded deep enough into the soil.

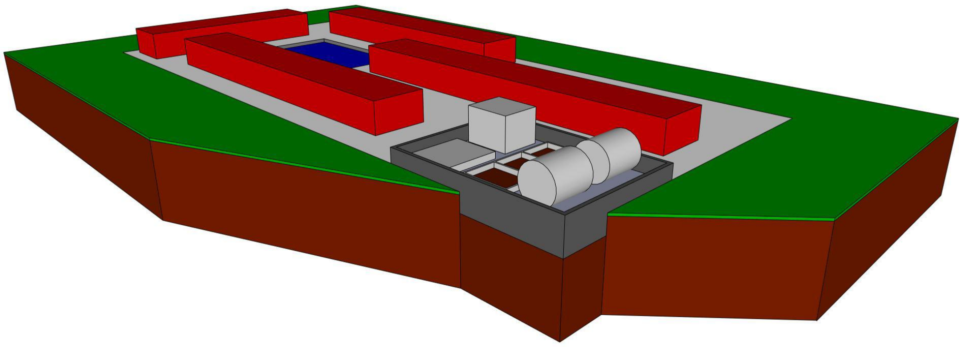


Figure 4.3: 3D schematisation of the embedded wall around the waste water treatment plant

ADVANTAGES

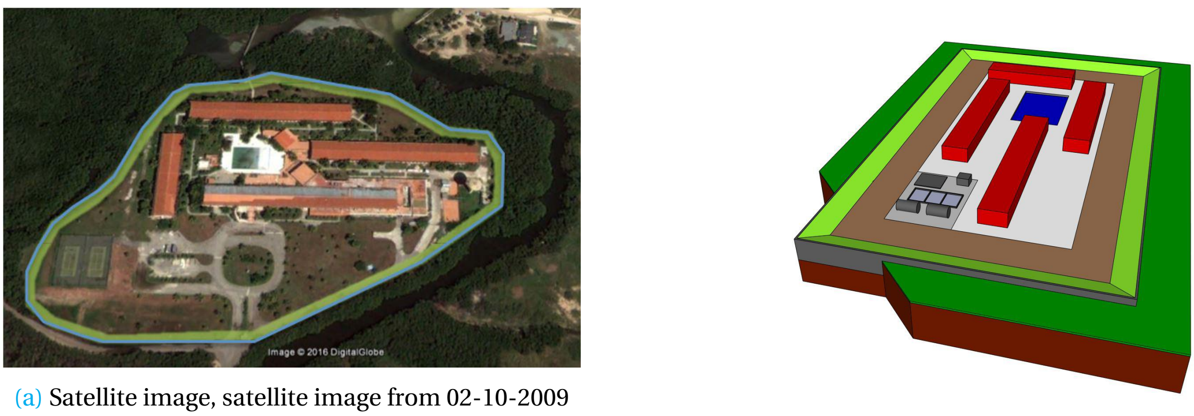
- The embedded wall can have a smaller width than the gravity wall, because the structure is stronger due to the additional strength of the passive soil.

DISADVANTAGES

- The solution is not aesthetic, as the landscape is polluted with a decreased view of the mangrove forest, which surrounds the Blau Arenal Hotel. However, the wall structure around the waste water treatment plant solely will possibly increase the aesthetic value of the area.
- The construction method is more complex, because part of the wall is embedded in the ground.
- The option is more expensive due to its complexity. More equipment is need to realise the embedded wall.

WALL WITH SLOPE

Besides the possibility to construct a wall only, a natural slope can be added at the inner side of the wall (see figure 4.4a). This solution of a wall with slope structure focuses more on the aesthetic view of the wall, as the slope will take away the effect of looking to a high wall. In figure 4.4b a three-dimensional schematisation of the wall with slope is shown.



(a) Satellite image, satellite image from 02-10-2009 [5]

(b) 3D schematisation

Figure 4.4: Visualisation of the wall with slope around the Blau Arenal Hotel and the waste water treatment plant

ADVANTAGES

- This solution has the most aesthetic value of the four options, because the slope will improve the view of the structure. The wall is hidden behind the slope, which is covered with vegetation. This solution will get more support from the Blau Arenal Hotel, as they want to satisfy the tourists, which will appreciate this solution more than the others.

DISADVANTAGES

- The solution is more expensive than the other options, because extra costs are caused by the slope due to an increase in dimensions of the wall and the material to create the slope.
- The solution is more complex than the other options.
- There is not sufficient space available at some locations to create a gentle, natural slope.

SHEETPILE WALL

The sheetpile wall is the fourth option. It can be seen in figure 4.5. This solution differs from the others due to its material. Not concrete, but steel S235 is used to retain the water. The type of sheetpile is HOESCH 2305 (finger-and-socket interlock). The characteristics of the sheetpile can be found in appendix Q.

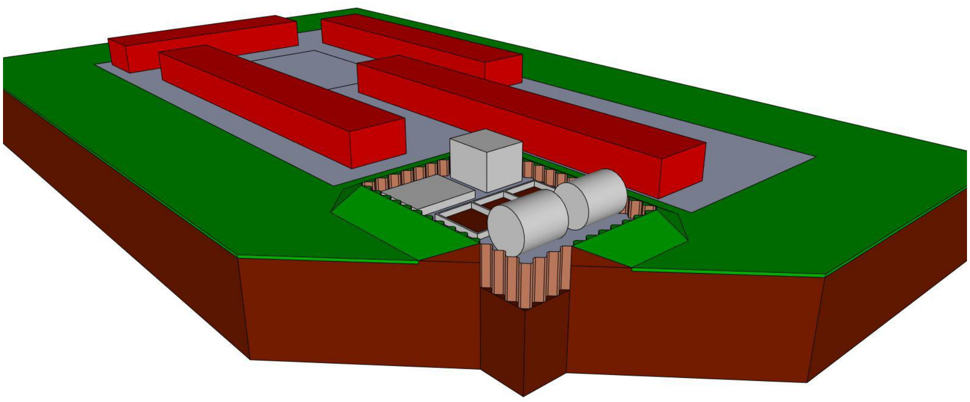


Figure 4.5: 3D schematisation of the sheetpile wall around the waste water treatment plant

Because a steel wall has low aesthetic value, it is chosen to add a slope. In this way, the calculation of the height of the sheetpile wall is the same as that of the concrete wall with slope. The width of the wall decreases significantly due to the difference in material properties.

ADVANTAGES

- The construction method of the sheetpile wall is easy.
- The investment is relatively cheap.
- Hardly any maintenance is needed. Only regular inspections to check corrosion of the steel.

DISADVANTAGES

- The solution has very low aesthetic value, as the landscape is polluted with a steel wall. A concrete wall surrounding the area will get more acceptance than a sheetpile wall.

4.2.4. DIMENSIONS

The main dimensions of each option for a wall around the waste water treatment plant only and a wall around both the waste water treatment plant and the hotel are listed respectively in tables 4.1 and 4.2 (for calculations, see appendix Q).

Table 4.1: Main dimensions of the four options for a wall around the waste water treatment plant only

	Gravity wall	Embedded wall	Wall with slope	Sheetpile wall
Total height of wall above ground [m]	1.4	1.4	1.4	1.4
Embedded depth [m]	0	3.1	3.6	3.6
Width of wall [mm]	1000	350	350	11.5

Table 4.2: Main dimensions of the four options for a wall around both the hotel and the waste water treatment plant

	Gravity wall	Embedded wall	Wall with slope	Sheetpile wall
Total height of wall above ground [m]	2.4	2.4	2.4	2.4
Embedded depth [m]	0	5.2	5.2	5.2
Width of wall [mm]	1600	500	650	9.4 - 11.5

4.2.5. CONCLUSION

From the above described advantages and disadvantages, it can be concluded that each of the four walls is an interesting option to protect the area against flooding. Therefore, it is decided to elaborate more on each of these walls.

The gravity wall, the embedded wall and the sheetpile wall can be realised for both locations (WWTP only or WWTP and hotel). The wall with slope will only be worked out as a wall structure around the Blau Arenal Hotel and the waste water treatment plant, because it has an aesthetic value. It is not used as an option around the waste water treatment solely, because the wall does not need to be aesthetic, as the waste water treatment already has low aesthetic value. Besides, the available space is not sufficient around the waste water treatment to construct the wall with slope option.

4.3. FLOOD RETENTION BASIN

4.3.1. GENERAL

In order to reduce the discharge at the Blau Arenal Hotel area, a flood retention area can be constructed more upstream of the Itabo River. The main idea of this solution is to store a certain amount of water temporarily. This measure can be effective for this specific area, because of the relatively short duration of the flood wave; the total duration of the 225 year hydrograph is 33 hours, see chapter 2.11. The controlled flooding of the flood retention area should start at the right time in order to reduce the peak river discharge. If the retention basin is filled at the start of the flood, it would not have influence on the peak river discharge. This is because no additional water can be stored when the peak arrives. The retention principle is more extensively described in appendix R.

4.3.2. LOCATION FLOOD RETENTION BASIN

For this solution one possible location of the flood retention area is investigated. However further analysis for other suitable locations is recommended. For this analysis a more detailed map of the Itabo catchment area is vital.

A suitable location for the flood retention basin should comply to several requirements:

- The flood retention area should be large enough to store the amount of water needed for the reduction of the peak discharge. The storage is determined based on an iterative analysis of the discharge reduction, given flooding of the basin at a certain discharge, see section 4.3.3.
- Flooding of the flood retention area should be controlled flooding. No flooding of the surrounded area of the flood retention basin should occur. This might might imply that dikes has to be be built around the basin to guarantee the controlled flooding. However, preferably an already existing elevation is used as flood defence.
- The area should be low enough with respect to the water level of the river to be able to store water. The area can be excavated in order to accommodate more water. However, a deep excavation will lead to higher costs.

Based on these criteria, a location is selected close to the Itabo River mouth. The total area is approximated, using Google Earth measuring tool, to 176,700 m^2 [5]. The area, south of the Blau Arenal Hotel, can be seen in figure 4.6. The reasons that make this area suitable as flood retention area are:

- Area next to the Itabo River.
- Relatively low lying area, see appendix R, figure R.3 and the explanation.
- Agricultural land, no houses and buildings in the area that need to be removed. This leads to less costs and more social acceptance.

Figure 4.6: Flood retention area [5]

4.3.3. DIFFERENT SCENARIOS

In appendix R, a natural inlet structure is chosen to discharge the extra water into the retention basin in case of high river discharge. This inlet structure is optimised for two different scenarios, the one in 100 and 225 year condition. These two variants are optimised for the two return periods because the 100 year condition is relevant for flooding of the Blau Arenal Hotel and the 225 year for the waste water treatment plant, see section 2.9. The design of the variants for both scenarios are elaborated in appendix R.

ADVANTAGES

The main advantages of the flood retention basin are:

- Reduction of the (peak) discharge downstream in the critical area. In this case, the Blau Arenal Hotel area. This is the main objective of the measure.
- Controlled flooding instead of uncontrolled flooding. The solution could reduce the flooding over the whole area downstream of the basin. Therefore, it might be feasible to adapt this solution more upstream.

DISADVANTAGES

The main disadvantages of the flood retention basin are:

- Loss of land, which could be available for agriculture and other purposes. However, the land will be used as flood retention basin in extreme conditions, it can still be used for agriculture as losses due to flooding will occur very seldom.
- The inlet of the retention basin should work properly in conditions that occur very rarely. Therefore, maintenance and management should not be neglected.
- Construction of dikes and excavation of soil might result in high costs.

4.3.4. CONCLUSION

The flood retention basin is a different solution than the others, as it reduces the flooding level, instead of retaining it. Therefore, the two different variants of the basin, corresponding to the one in 100 year and one in 225 year conditions, are chosen to be simulated.

4.4. SOUTHERN DIKE

This solution was created when examining the results from the model. In section 3.2.2, figure 3.11 the time series of flooding of the area is shown. In this figure it can be seen that the flow has a lot of momentum, resulting in direct flooding of the land that is on the path of the flood wave. Since the hotel and the waste water treatment plant are directly in the way of this flood wave, this area seems to flood very rapidly compared to the rest of the area. The water comes from the South, and this southern area, just south of the hotel area, is some lower than the rest of the area directly next to the hotel. From this observation the idea rose to make a southern dike. The location of this dike can be seen in figure 4.7. The function of this southern dike is to change the way the water flows. It pushes the water towards the East and the West, resulting in the water flowing around the hotel towards the lagoon (on the west side) and towards the river mouth and consequently to the sea (on the east side).

Figure 4.7: Planned location of the southern dike [5]

ADVANTAGES

- This solution does not barricade the whole hotel area, nor only the waste water treatment plant. In case of heavy rainfall, precipitation falling in the hotel area can still flow away easily. The current drainage system of the hotel area can still be used.
- No construction into the ground gives the French drainage system the chance to discharge the cleaned water into the river (see appendix C).
- This dike solution is out of sight for most tourists. No structure around the entire hotel has to be made. This avoids the decrease of aesthetic value of the surrounding mangrove forest.

DISADVANTAGES

- Because the structure will be built on top of the existing soil, it is possible that seepage will play a role. However, because of the clayey soil, there will not be a lot of seepage in the hotel area during a flood period.
- The structure will be built on top of the existing soil, resulting in a much wider dike structure. This structure will be constructed close to the mangrove forest south of the waste water treatment plant, possibly resulting in loss of mangroves.
- A lot of sand has to be transported to the area for the construction of the dike.

4.4.1. CONCLUSION

The southern should be investigated further. Especially due to the advantage that the runoff of the precipitation in the hotel area is still possible. Moreover, the loss of aesthetic value of this solution is minimised. However, before the final design is made, a simulation is made first to check the effects of the southern dike. Water might flow around the dike, still resulting in flooding. If this is not the case, the dike will be worked out.

4.5. CONSTRUCTION METHOD

In this section a brief discussion of the construction method of the different variants, is described. This qualitative analysis will be used as input for the Multi Criteria Analysis (MCA), because constructability is an important factor that should be accounted for. Based on the MCA a measure, or a combination of measures, is chosen. The construction method of this final solution is developed in more detail, see section 7.1.

4.5.1. GRAVITY WALL

The characteristics of the gravity wall can be found in section 4.2. The wall can be built around the waste water treatment plant (WWTP) or around the WWTP and hotel. There is no difference in construction method, only the length differs. The wall can be cast in-situ. An advantage of this method is that no heavy blocks need to be lifted. A disadvantage is that hardening of the concrete needs a lot of time and cooling of the concrete might give problems. Prefab elements are very heavy. Special lifting equipment is needed to carry out this operation. The biggest element weights approximately 10 tonnes per meter length. For this reason in-situ concrete is chosen as construction method. A lot of concrete has to be transported to the site.

4.5.2. EMBEDDED WALL

The concrete embedded wall is used as a flood retaining structure. The wall can be built around the hotel and waste water treatment plant (WWTP) or only around the latter one. The wall and water level characteristics for this solution can be seen in section 4.2. The total embedded depth is quite large, the construction method is as follows (diaphragm wall construction). The soil is excavated to the lowest level and a bentonite mixture is put into the gap to prevent collapsing of the soil. Formwork and falsework is placed at the edges of the gap, from ground level to the top of structure (ToS). The reinforcement is placed in the bentonite mixture, after which the concrete is poured into the gap to ToS. The density of concrete is higher than the bentonite density, so the bentonite will float on the fresh concrete. The bentonite mixture is removed and will be used again at the next part of the wall.

The big drawback of this construction method is the complexity of the bentonite operation. The method is mainly used for big excavation depths. The bentonite is transported through pipes with pumps, failure of this pumps can cause collapsing of the excavation. Failure of the electricity supply is likely to occur in Cuba. Furthermore, the construction method is expensive due to the use of specialised equipment.

4.5.3. WALL WITH SLOPE

The wall characteristics of the concrete wall with slope solution are presented in section 4.2. All the values are based on a one in 225 year event. The difference between the concrete wall with slope and the wall without slope is an additional aesthetic value. The technical difference between the two variants is the different load situation that is governing. The governing load situation for the concrete wall without slope is when the water level at the outside is at the maximum level. The governing load situation for the concrete wall with slope is when there is no water at the outside and the soil at the inner side is fully saturated.

The construction method of the wall is the same as for the concrete wall without slope, see section 4.5.2. The slope is constructed after hardening of the concrete. The same drawbacks as described in section 4.5.2 hold for this variant.

4.5.4. SHEETPILE WALL

The wall characteristics of the sheetpile wall with slope can be found in section 4.2. The sheetpile wall can be built around the WWTP and hotel or around the WWTP only. The only difference in construction method is the total construction length, see appendix Q. The sheetpiles have to be transported to the site. They will be driven down by a sheetpile driver. Soil will be transported and placed and levelled, to construct the slope.

4.5.5. FLOOD RETENTION BASIN

The flood retention basin is constructed upstream from the Blau Arenal Hotel. Construction of this basin consists of the following tasks:

- Excavation of the flood retention area to accommodate more water. The total area of the flood retention basin is 176,700 m^2, the excavation depth is approximately 4 m, the total amount of soil that has to be removed is 706,800 m^3.
- Construction of the inlet structure. The length of the inlet structure is assumed to be 100 m. Construction work consists of lowering of the crest of the river bank and construction of the slope protection at the outer side of the river bank, to prevent erosion due to inflow of water.
- Construction of flood defence structures around the flood retention area. Dikes have to be built in order to prevent flooding area surrounding the flood retention area.

4.5.6. SOUTHERN DIKE

The southern dike is an earthen structure. The construction consists of the following tasks:

- Removing of trees and vegetation from the existing river bank.
- Transportation of soil to the river bank.
- Construction of the earthen dike, with use of excavators.
- Construction of the soil cover of the dike, to prevent seepage and piping problems.

5

SIMULATION

In this chapter, the solutions from chapter 4 are implemented in the XBeach model. In table 5.1 the combinations of solution and river discharge, which are run in XBeach, are shown. For the wave field the summer conditions have been chosen, with the closed river mouth (and not the winter conditions). The summer conditions are chosen, because the rain season coincides with the summer.

Table 5.1: Scenarios with solutions

Nr.	Solution	River discharge [1/year]	Wall height [m +MSL]
6a	Retention basin 100 year	1/100	-
6b	Retention basin 100 year	1/225	-
7a	Retention basin 225 year	1/100	-
7b	Retention basin 225 year	1/225	-
8a	Wall around WWTP	1/100	3.40
8b	Wall around WWTP	1/225	3.40
9a	Wall around both	1/225	3.40
10a	Southern dike	1/100	3.40
10b	Southern dike	1/225	3.40

The first four runs are done with the two different retention basin solutions, for both river discharges. Then two runs with only a wall around the waste water treatment plant are executed. With run 8a is checked whether the hotel will flood or not for for the 1/100 year conditions when implementing the wall around the waste water treatment plant. In this run both the hotel and the waste water treatment plant should not flood, in order to satisfy the requirements mentioned in section 2.9. Run 8b is done to check the height of the wall around the waste water treatment plant during the 1/225 year conditions. For this run it is not a problem if the hotel floods, but the waste water treatment plant should not flood. Run 9a is executed to check if the waste water treatment plant will not flood during the 1/225 year conditions, when building a wall around the whole area.

It is important to implement this solution in a run, because the area in which water can flow, gets significantly smaller. A 1/100 year condition is not necessary to run for a wall around both, because the wall around the waste water treatment plant always has to withstand a 1/225 year condition. Finally, two runs are done with the southern dike. Both a river discharge of 1/100 and 1/225 years are simulated, to check if respectively the hotel and the waste water treatment plant are flooded.

The expected flooding level is determined based on the flooding level at the waste water treatment plant. These flooding levels are shown in table 3.1 in section 3.2.2. For a 1/100 year river discharge this value is 30 cm, which is equal to a water level of MSL +2.71 m. For a 1/225 year river discharge this value is 49 cm, which is equal to a water level of MSL +2.90 m. The height of the wall is determined based on these expected water levels in appendix Q.

5.1. IMPLEMENTATION

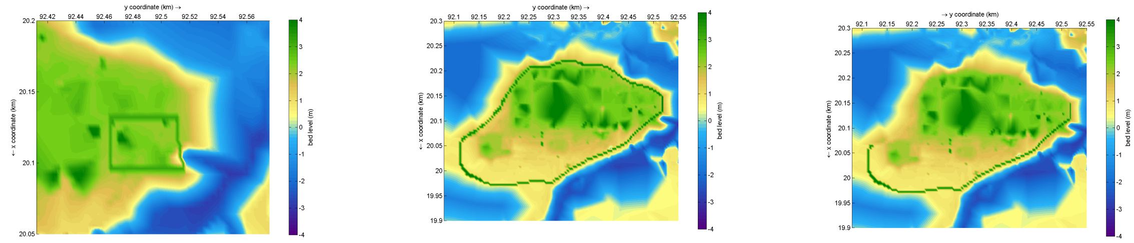
In this section the implementation of the solutions in the XBeach model is described.

5.1.1. RETENTION BASIN

Because of the fact that the retention basin is located upstream of the model domain, the only difference will be the river discharge input. The use of a retention basin results in a decrease of the peak discharge. The functioning of the retention basin is described in section 4.3. The resulting hydrographs are shown in section R.6 in appendix R. The 10% discharge reduction, described in section 2.7 is not yet taken into account. After this has been done, the new discharge time series are put in the model.

5.1.2. WALL STRUCTURE

The wall structure is harder to implement in the XBeach model. It needs to be implemented in the bathymetry file. Using the program *QUICKIN*, all cells at the location of the wall are given the correct value (MSL +3.40 m). Because of the size of the grid cells, the smallest width of the wall that can be put in the model is 2 meters. Some of the grid cells have a width of 4 meters. This is not a problem, because the wall is put in the model to check its ability to withstand the water level purely from a flooding perspective, and not from a structural perspective. The structural part is calculated in section 4.2. The southern dike is implemented in the same way as the walls. Figures 5.1a, 5.1b, 5.1c show respectively the implementation of the wall around the waste water treatment plant only, around both the waste water treatment plant and the hotel and the southern dike. It can be seen that the areas inside both walls differ a lot. The wall around the waste water treatment plant is protecting a much smaller area.



(a) Waste water treatment plant

(b) Both the hotel and the waste water treatment plant

(c) Southern dike

Figure 5.1: Implementation of the wall solutions in the XBeach model

5.2. Results

The results of the solutions implemented in XBeach are shown in table 5.2. In column two the water depth at the waste water treatment plant is shown, and in column three the water depth at the hotel. The water level in the river (+MSL) can be observed in column four.

Table 5.2: Results of the scenarios with solutions

Nr.	Flooding WWTP [m]	Flooding hotel [m]	Water level river [m +MSL]
6a	0.10	0.42	2.52
6b	0.35	0.73	2.84
7a	0.22	0.54	2.67
7b	0.33	0.69	2.82
8a	-	0.71	2.8
8b	-	0.98	3.11
9a	0.15	0.03	3.44
10a	-	0.17	3.11
10b	0.23	0.65	3.44

As can be seen in this table, the retention basins (run 6a up to 7b) did not reduce the water level enough to make sure the waste water treatment plant would not flood. In all four runs both the waste water treatment plant and the hotel are flooded. A solution for this could be to excavate the whole retention basin to a lower level, and lower the eastern river bank as well. However, when lowering the river bank with for example 30 cm, the whole area should be excavated more than 30 cm to accommodate the extra water that has to be stored in the basin. For an explanation of the inlet structure, see section 4.3. Besides that, for both the 1/100 and the 1/225 basin the water depth at the waste water treatment plant is more than 30 cm. This amount of water is not reduced easily, as the hydrograph gets wider and wider for lower water levels. This implies that reducing the peak discharge more leads to the need of a lot more storage area, resulting in even more excavation. Excavation will quickly reach levels of around MSL, which is not beneficial. This problem could be solved by searching for a larger storage area, or a storage area more upstream, so deeper excavation is less of a problem. However, such an area was not found. Therefore, the flood retention basin is not chosen as a solution to continue working on.

When looking at run 8a, it can be seen that for a 1/100 condition the calculated wall around the waste water treatment plant is high enough. This was expected, because the wall has been dimensioned on the 1/225 year condition, and this run is only with the 1/100 river discharge. However, the water depth at the hotel increases from 63 cm in the run without wall (run 4a) to 71 cm due to this wall around the waste water treatment plant and the hotel.

When looking at run 8b, the wall around the waste water treatment plant also is high enough for the 1/225 year river discharge. However, the water depth at the hotel increases from 95 cm to 98 cm.

When looking at run 9a, the wall appears not to be high enough. At the waste water treatment plant the water depth is 15 cm and at the hotel a water depth of 3 cm is found. This result was not expected, but can be explained by the fact that the water of the flooded river is not able to flow into the area of the hotel and the waste water treatment plant. Because of this, the water level around the wall rises significantly. Even the extra height of 50 cm, which was added for safety when designing the wall, is not enough to stop the water from flowing over the wall, because this piling up results in 54 cm of extra water. This leads to an adaptation in the wall height in section 5.3.

Run 10a shows that the southern dike protects the waste water treatment plant against flooding for a 1/100 year condition. However, the hotel is flooded by 17 cm of water.

Run 10b shows that the southern dike fails for a 1/225 year river discharge. Both the waste water treatment plant and the hotel are flooded during this event.

During the analysis of the results of the simulations from table 5.2 the idea was born to check some combinations of the solutions that fail on their own.

The retention basin 100 year does not prevent the waste water treatment plant from flooding, but it significantly decreases the amount of water at the hotel during a 1/100 year river discharge; from a water depth of 63 cm to a water depth of 42 cm. A combination of this retention basin with a 1/225 year wall around the waste water treatment plant decreases the negative effects for the hotel, while keeping dry the waste water treatment plant. This is still not in accordance with the requirement of a dry hotel during a 1/100 year river discharge. However, this requirement is not enforced by the client. If this solution works and if it results in a good MCA value, it might be a valuable solution. It is important to keep in mind that the 1/100 year requirement for the hotel is not considered as a hard requirement, while the 1/225 year condition for the waste water treatment plant is a hard requirement (see section 2.9).

Another combination is a wall around the waste water treatment which will withstand a 1/225 river discharge condition, and a wall around the hotel which will withstand a 1/100 river discharge condition. The reason for this is that this lower wall might be a lot cheaper.
A final combination that is worth checking, is a retention basin with a southern dike. The southern

dike reduces the flooding levels at both the waste water treatment plant and the hotel, for both river discharges. This reduction solely is not sufficient. The retention basin also reduces the water levels at both spots. Together with the southern dike the total reduction could be enough to comply to all the requirements. This results in four more combinations, which can be observed in table 5.3.

Table 5.3: Scenarios with combinations of solutions

Nr.	Solution 1	Solution 2	River discharge [1/year]	Wall height [m +MSL]
11a	Retention basin 100 year	Wall around WWTP	1/100	4.0
12a	Retention basin 100 year	Southern dike	1/100	4.0
12b	Retention basin 100 year	Southern dike	1/225	4.0
13a	Retention basin 225 year	Southern dike	1/100	4.0
13b	Retention basin 225 year	Southern dike	1/225	4.0
14a	Wall around WWTP	Wall around the hotel	1/225	4.0 & 3.7

5.3. ADAPTATION WALL LEVELS

Because of the piling up of the water against the wall, when it is built around both the hotel and the waste water treatment plant, the water flows into the protected area. This effect has to be taken into account when determining the height of the wall. The new flooding level that follows from the model is MSL +3.44 m. Again for safety an extra 0.5 m is added to the height of the wall. To get to a round number, a level of MSL +4.0 m is used for the top level of the wall. The calculations, performed in appendix Q, are done with the new value. The results are shown in figures 5.2, 5.3 and 5.4.

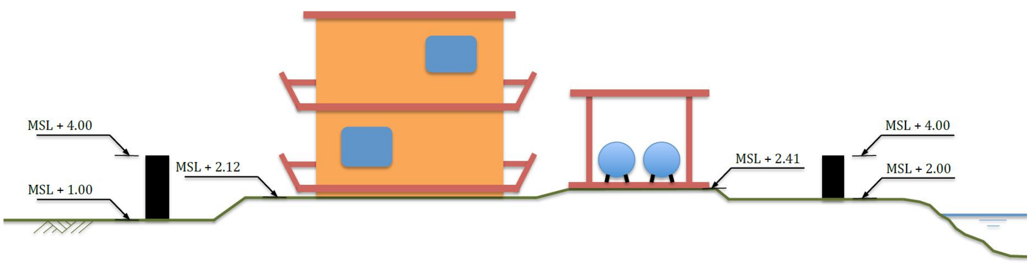


Figure 5.2: Gravity wall

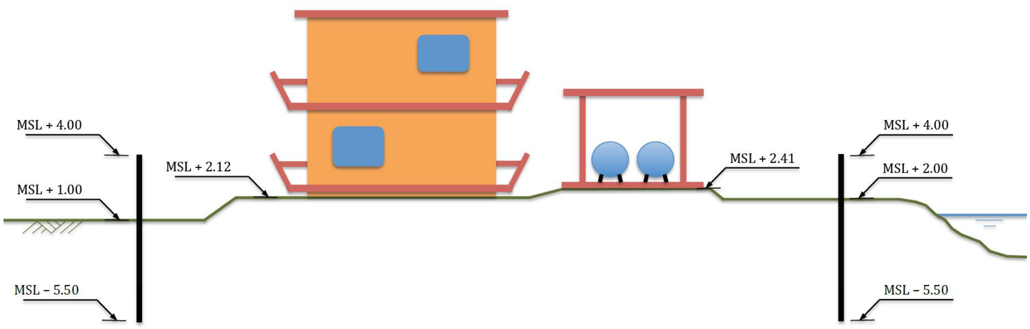


Figure 5.3: Embedded wall

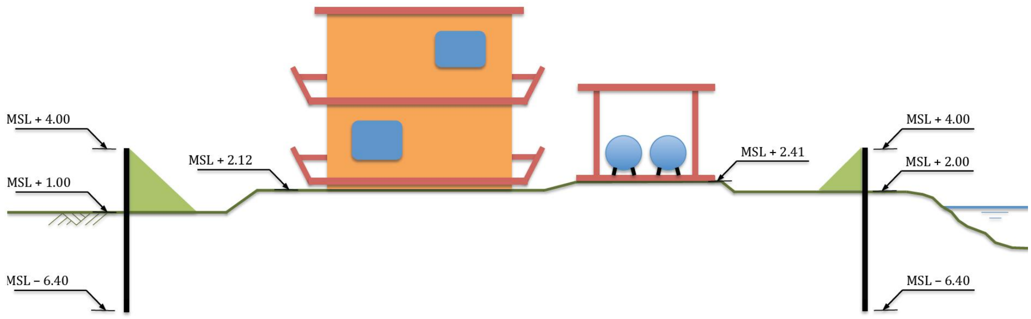


Figure 5.4: Wall with slope

5.4. COMBINATIONS

The results of the combinations introduced in section 5.2 are shown in table 5.4.

Table 5.4: Results of the scenarios with combinations of solutions

Nr.	Flooding WWTP [m]	Flooding hotel [m]	Water level river [m +MSL]
11a	-	0.43	2.50
12a	-	-	3.11
12b	0.02	0.28	3.15
13a	-	-	2.93
13b	-	0.21	3.12
14a	-	-	3.44

Run 11a shows that the retention basin in combination with the wall (with a new height) around the waste water treatment plant decreases the water depth at the hotel with 28 cm, from 71 cm to 43 cm. This is one centimeter less than the reduction without wall, but it might still reduce the damage on the hotel for a 1/100 year discharge.

Run 14a shows that the combination of a wall around the waste water treatment plant and a lower wall around the hotel is able to withstand a 1/225 year flooding. Because of the 0.5 meter extra wall height, the wall around the hotel, which is designed for a 1/100 year conditions, is able to withstand the 1/225 year conditions.

Run 12a shows that the retention basin 100 year in combination with the southern dike ensures no flooding both at the hotel and at the waste water treatment plant. However, in run 12b the waste water treatment plant is flooded with 2 cm for the 1/225 year conditions.

From run 13a can be seen that the retention basin 225 year in combination with the southern dike meets the requirements. Also during a 1/225 year storm the waste water treatment plant is not flooded, as can be seen in run 13b.

Besides the combination of the small retention basin with the wall around the waste water treatment plant, which reduces the flooding at the hotel, a combination of the retention basin 225 year and the southern dike seems to be a solution that meets all requirements. However, for all these runs a morphological factor of ten was used, with as a consequence the fact that the flood wave discharge is imposed ten times as quickly as it should be normally. This might have effect on the way the flooding propagates through the domain. From engineering intuition one would argue that during a flood wave with a normal duration of 33 hours, the water level would increase gradually, and after the peak it would decrease gradually again. However, in the model the flood wave had a big momentum, and because of this the steering of the water with the southern dike was preventing flooding in the area behind the dike. This effect might be smaller for a lower morphological factor; flow velocities of the flood wave will be smaller and a more gradual increase of the water level might result in flooding behind the southern dike.

To check this reasoning, an extra run with the 1/225 river discharge and the 225 year retention basin is done. This run is executed with a morphological factor of five (instead of ten), so the water level will rise more gradually. It is expected that the water level will rise more gradually. A resulting time series of the flooding is shown in figure 5.5.

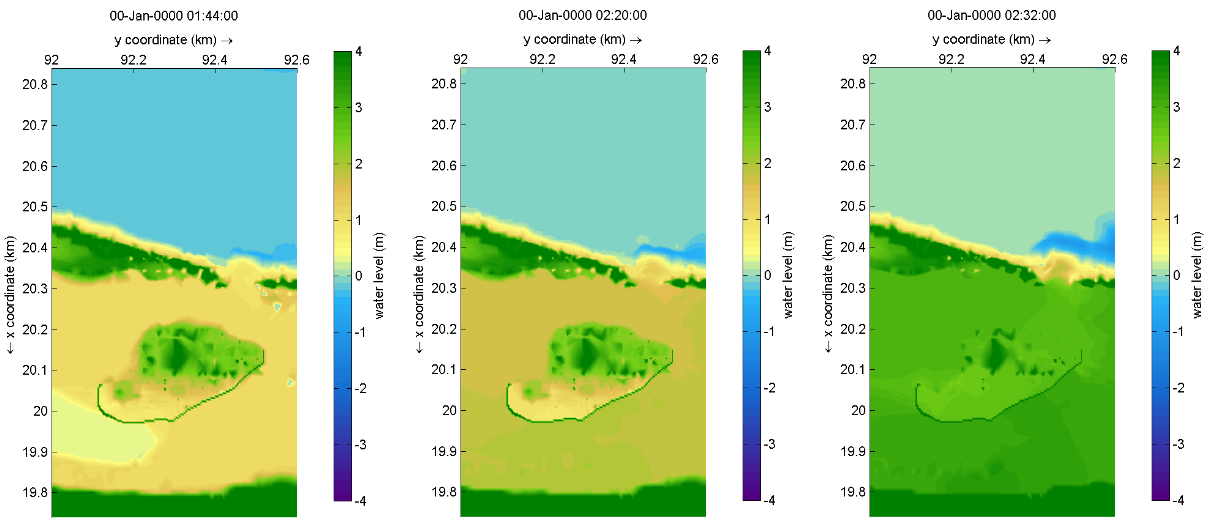


Figure 5.5: Time series of the run with a morphological factor of 5

From figure 5.5 can be concluded that the southern dike solution does not work. When running the model with a longer simulation time and a lower morphological factor, the water level is indeed rising more gradually. This results in inflow of water at the northern boundary of the hotel area. In the end, both the waste water treatment plant and the hotel are flooded. According to the model, a solution like the southern dike is not applicable.

5.5. CONCLUSION

From the model is concluded that the following solutions work and will be continued working on:

- A wall around the waste water treatment plant only, dimensioned on a 1/225 year event.
- A wall around the waste water treatment plant and the hotel, both dimensioned on a 1/225 year river discharge
- A wall around the waste water treatment plant, dimensioned on a 1/225 year event and a wall around the hotel, dimensioned on a 1/100 year event
- A wall around the waste water treatment plant, in combination with the 1/100 year retention basin

6

EVALUATION

6.1. INTRODUCTION

The fourteen possible solutions to prevent flooding at the Blau Arenal Hotel area are evaluated in this section. In the end one suitable solution is presented. The evaluation of the solutions will be based on the Multi Criteria Analysis (MCA) and the costs. The MCA compares several factors of the variants, using six criteria. These different criteria are not weighted equally, for a detailed description of the MCA see section 6.2. Based on the MCA and the costs a final solution is determined.

6.2. MULTI CRITERIA ANALYSIS

In this section a Multi Criteria Analysis (MCA) is done. This MCA will be used to assess the 14 possible solutions of the simulation.

6.2.1. CRITERIA

The criteria of the Multi Criteria Analysis are elaborated here, including an explanation of the aspects that are taken into account per criterion. The solutions are evaluated by means of the criteria. They will get a value that lies between zero (very poor) and five (excellent). The scores of the solutions per criterion can be found in appendix U.

- **Prevention of contamination of the mangrove forest**
  The main function of the solution is the prevention of flooding of the waste water treatment plant (WWTP), due to high river discharge. In this way, the unique ecosystem of the mangrove forest, which surrounds the Blau Arenal Hotel, is protected against contamination. This will be valued here.

- **Water damage hotel**
  Next to the protection of the mangrove forest, the water damage to the Blau Arenal Hotel is a criterion the solution needs to comply to. The protection of the Blau Arenal Hotel against flooding and the resulting water damage is judged.

- **Constructability**
  The complexity of the construction of the solution is judged, taking into account the local experience with the construction method and the design.

- **Aesthetic value**
  The aesthetic value of the solution is judged here. The realisation of the solution can pollute the landscape.

- **Maintenance**
  The amount of monitoring and regular checking of the solution in operation falls under the criterion maintenance (visual inspection of gaps or corrosion). The durability and robustness of the solution are also taken into account.

- **Social acceptance**
  The criterion social acceptance is about the resistance of people and organisations against (parts of) the solution, for example the relocation of local residents and the opinion of the tourists.

- **Sustainability**
  The criterion sustainability is about the damage done to the environment by the solution during the life cycle of the solution, which consists of the construction, operational and removal phase.

6.2.2. WEIGHING FACTORS

The MCA has weighed criteria (see figure 6.1), because not every criterion is valued the same. The criteria in the first column are compared with the criteria in the first row. If the criterion in the row is more important than the criterion in the column, a green ball is placed in the box. When the criterion in the column is more important, the box is filled with a red ball. The weighing factor of the criterion is the total number of green balls in the row of that particular criterion plus one. For instance, constructability has a higher weighing factor than maintenance. The plus one is due to the fact that the weighing factor can never be zero.

As can be seen in figure 6.1, the criterion prevention of contamination of the mangrove forest has the highest weighing factor. This is done, because it is the main goal of the project. The water damage to the Blau Arenal Hotel is valued with a weighing factor of four. This is valued lower than the protection of the mangrove forest, because the flooding level of the hotel is below one meter. Therefore, the hotel experiences water damage and the risk of human life loss is not at stake. The aesthetic value of the solution is more important, because the Blau Arenal Hotel will be more satisfied with a solution that has aesthetic value, as the tourists will appreciate this more. Therefore, a weighing factor of five is chosen for this criterion. The constructability is valued with a weighing factor of 6, because the complexity of the construction of the solution is an important aspect to take into account, as the possibilities and construction methods are different in Cuba than in Europe. The criteria maintenance, social acceptance and sustainability are valued lower than the other four, but are also judged in the MCA.

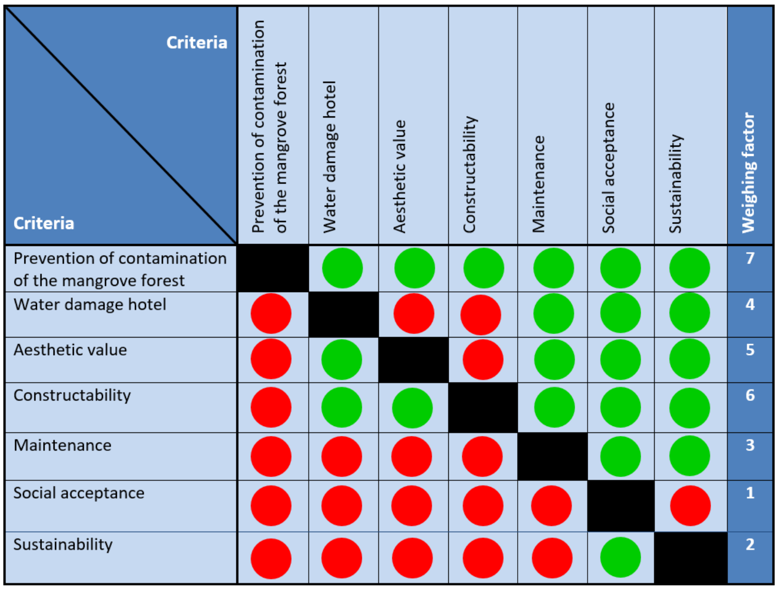


Figure 6.1: Determination of the weighing factor of each criterion

6.2.3. RESULTS

With the scores of each solution per criterion (appendix U) and the weighing factors of the criteria, the table of the MCA can be filled in (see figure 6.2). The total score of a solution is obtained by adding up the values of the score of each criterion multiplied with the weighing factor of this particular criterion.

		Wall around WWTP 1/225			Wall around WWTP 1/225 and hotel 1/225				Wall around WWTP 1/225 and hotel 1/100				Wall around WWTP 1/225 and retention basin 1/100		
Weight	Criteria \ Solutions	Gravity wall	Embedded wall	Sheetpile wall	Gravity wall	Embedded wall	Sheetpile wall	Wall with slope	Gravity wall	Embedded wall	Sheetpile wall	Wall with slope	Gravity wall	Embedded wall	Sheetpile wall
7	Prevention of contamination of the mangrove forest	28	28	28	28	28	28	28	28	28	28	28	35	35	35
4	Water damage hotel	4	4	4	20	20	20	20	16	16	16	16	8	8	8
5	Aesthetic value	25	25	20	10	10	5	15	15	15	10	20	10	10	5
6	Constructability	18	12	24	12	6	18	6	12	6	18	6	6	6	12
3	Maintenance	12	12	15	9	9	12	9	9	9	12	9	6	6	9
1	Social acceptance	5	5	5	4	4	4	4	4	4	4	4	1	1	1
2	Sustainability	10	6	8	6	4	6	4	6	4	6	4	4	2	6
	Total	102	92	104	89	81	93	86	90	82	94	87	70	68	76

Figure 6.2: Multi Criteria Analysis

As can be seen in the table, the gravity wall and the sheetpile wall around the waste water treatment plant for 1/225 year river discharge conditions are the solutions with the highest score (shown in green). Moreover, the scores of the solutions of the walls around the waste water treatment plant in combination with a retention basin are significantly lower than the options without the retention basin. In every case, the sheetpile wall is the best option of the walls, followed by the gravity wall.

6.3. COSTS

6.3.1. COST ESTIMATION IN CUBA

The Cubans have a different way of assessing the costs of construction. Research is done to achieve an estimation of the costs of each solution. The information is received from the Cuban Ministry of Construction *(Dirección de Presupeustos y Precios del Ministerio de la Construcción)* and consists of the *PRECONS (Sistema de precios de la Construcción)*, which is the document that contains the data [33]. The costs are determined by dividing the total costs into two categories. All direct related costs to the construction are called primary costs and all indirect related costs are called secondary costs. These two main categories have been subdivided in sections. In appendix V, section V.1 and section V.1 an elaboration is given on the primary and secondary costs and their sections. Because the costs are determined by estimation, the risk of underestimation needs to be taken into account. This procedure is explained in appendix V, section V.3.

The costs are calculated and given in *PRECONS* in the Cuban currency, which is called CUP (CUban Peso nacional). Cuba is one of few countries that uses two currencies, CUP that is used by the Cubans and CUC (CUban peso Convertible) that is created for the tourists. For convenience, the final costs will also be presented in Euros, because the project group, the sponsor and the supervisor are originating from the Netherlands. The rates of these currencies are shown below.

$ 1.00 CUC = $ 24.00 CUP $ 1.00 CUC = € 0.93

6.3.2. RESULTS

An overview of the costs per solution is given in table 6.1. The entire determination of the costs of each solution can be found in appendix V, section V.4.

Table 6.1: Primary, secondary and total costs of all solutions

Solution		**Primary costs**		**Secondary costs**		**Total costs**	
		CUP [$]	EUR [€]	CUP [$]	EUR [€]	CUP [$]	EUR [€]
Wall around WWTP 1/225	Gravity wall	144,805	5,587	85,900	3,314	230,705	8,901
	Embedded wall	167,098	6,447	88,883	3,429	255,981	9,876
	Sheetpile wall	134,290	5,181	154,272	5,952	288,563	11,133
Wall around WWTP 1/225 and hotel 1/225	Gravity wall	1,920,097	74,078	366,013	14,121	2,286,109	88,199
	Embedded wall	2,754,789	106,280	474,542	18,308	3,229,340	124,589
	Sheetpile wall	447,222	17,254	238,852	9,215	686,073	26,469
	Wall with slope	3,664,180	141,365	587,516	22,667	4,251,696	164,031
Wall around WWTP 1/225 and hotel 1/100	Gravity wall	1,652,863	63,768	330,609	12,755	1,983,472	76,523
	Embedded wall	1,941,582	74,907	371,741	14,342	2,313,323	89,249
	Sheetpile wall	435,455	16,800	237,378	9,158	672,833	25,958
	Wall with slope	2,496,128	96,301	439,868	16,970	2,935,996	113,271
Retention basin 1/100		1,323,060	51,044	2,188,807	84,445	3,511,866	135,489

From table 6.1 can be concluded that the solutions of a wall structure around the waste water treatment plant only have the lowest costs, with the gravity wall as the cheapest option. The costs of the wall structures, that are built around both the hotel and the waste water treatment plant, are higher than around the waste water treatment plant only, due to the larger length of the wall structure. The costs of the concrete walls increase in the order of ten, while the costs of the sheetpile only doubles. The retention basin and the wall with slope are expensive options compared to the other wall structure solutions.

6.4. CONCLUSION

In this section the conclusion of the project is drawn. The final solution is given in section 6.4.1. The explanation of this advise is described in section 6.4.2.

6.4.1. FINAL SOLUTION

The final solution to prevent flooding at the Blau Arenal Hotel area is to build a gravity wall around the waste water treatment plant, with a safety level of one in 225 year. In this way, the objective to prevent contamination of the mangrove forest is accomplished and the demand of CITMA, the stakeholder with the most power and interest (see section 2.2.4), is fulfilled.

No measures are taken to prevent flooding of the Blau Arenal Hotel. Further research on the Blau Arenal Hotel area is advised to prevent flooding of the hotel. A more detailed description of the final solution is given in chapter 7.

If the Ministry of Tourism also wants to invest in a flood protection for the hotel, a wall can be built around the hotel as well. It is advised to build a wall structure that complies to the safety level of the hotel of a one in 100 year river flood protection. In this way, the hotel is protected against flooding occurring at return period of one in 100 year river discharge conditions and lower.

6.4.2. EXPLANATION FINAL SOLUTION

Two different safety levels are defined for protection against flooding of the waste water treatment plant and the Blau Arenal Hotel. The safety level of the WWTP is set to one in 225 year conditions. The safety level of the hotel is set lower, to a one in 100 year event. The background of this distinction can be found in section 2.9. The flood characteristics, such as the inundation depth, are not taken into account in this safety standards, as they where not known before the model results where available. Flooding of the waste water treatment plant will immediately lead to the worst case scenario, namely contamination of the unique ecosystem of mangrove forest in the surrounding area. However, the consequences of flooding of the Blau Arenal Hotel are highly dependent on the flood characteristics. For example a low inundation depth will lead to some damage at the hotel, but expensive measures might not be cost effective.

A solution that meets the above mentioned safety requirements is a gravity wall around the waste water treatment plant and hotel, for the design see chapter 4. The characteristics of the wall around the waste water treatment plant (WWTP) and hotel can be seen in table 6.2. The top of the structure is at a fixed level with respect to Mean Sea Level (MSL). However, the height of the structure varies, as the soil elevation varies over the whole trajectory of the wall. The hotel is at a higher elevation compared to the location of the wall, so the height of the structure is larger at this location than the height of the wall when it is built close to the hotel. However, the crest level of the wall is the same.

Table 6.2: Wall solution around WWTP and hotel

	Gravity wall WWTP (1/225 year)	**Gravity wall hotel (1/100 year)**
Top of structure +MSL [m]	4.0	3.7
Lowest ground level +MSL [m]	2.0	1.0
Max height wall [m]	2.0	2.7

A wall around both the WWTP and the hotel is effective as a flood retention, but it will have a negative influence on the functionality and aesthetic value of the hotel. Furthermore, from the the Multi Criteria Analysis (MCA) (see section 6.2) and the costs (see section 6.3) the wall structures around both the waste water treatment plant and the hotel are assessed with less credits than the wall structure around the waste water treatment plant solely. Based on these considerations another approach is advised.

From the MCA follows that the most optimal solution is to build a wall around the waste water treatment plant only, with a safety level of one in 225 year. The solution fulfils the requirements to prevent contamination of the mangrove forest. However, in this case, no measures are taken to prevent flooding of the Blau Arenal Hotel. Taking into account that CITMA is the primary stakeholder, as can be seen in section 2.2, the WWTP has the highest priority.

Taking no additional measures at the Blau Arenal Hotel implies that the area floods with even lower return period conditions than the standard of one in 100 year. The inundation depth of the hotel area, based on the XBeach models, is shown in table 6.3. The third column of the table shows the inundation depth with respect to the lowest ground level of the hotel, 2.12 m +MSL (see section 2.9).

Table 6.3: Inundation depth hotel area

Return period [year]	**Inundation level +MSL [m]**	**Inundation depth +ground level [m]**
25	2.32	0.20
50	2.60	0.48
100	2.80	0.68
225	3.11	0.99

If no flood mitigating measures are taken at the Blau Arenal Hotel, flooding of the area is accepted. Due to this acceptance, the probability of life loss and the amount of damage to the hotel need to be determined. This is done with the flood characteristics. The flood characteristics determine the probability of life loss and the amount of damage. The life loss estimation is based on equation 2.2 from section 2.8. The flood characteristics that determine this life loss are the inundation depth and the rising time of the flood. The rising time is in the order of hours (see section 3.2.2), the inundation depth is 0.68 m. The evacuation fraction reduces the probability of life loss even more. Therefore, the probability of life loss is very low during a one in 100 year river discharge condition. The amount of damage to the hotel during a flood is a function of inundation depth. This low inundation depth will lead to some damage to the hotel.

The water at the hotel area, even at fairly short return periods, is not desirable. Therefore, measures to prevent flooding at the hotel can be taken more close to the hotel at the lowest parts. The elevation map of the hotel (see figure L.4) shows that not the whole area is at the threshold level of the hotel MSL +2.12. There are several low areas at this level, but a large part of the hotel is at a higher level, going up to 3.43 m +MSL. Local heightening of the soil, construction of sills and low walls at weak spots can mitigate flooding problems for short return periods. This analysis is not included in this research because the uncertainty in for example the river discharge is high, so that these detailed measures can not be designed properly. Furthermore, the modelling software XBeach is not the best program to model these detailed measures against flooding of a river. An analysis with Delft3D is advised to model the river in a better way.

7

FINAL SOLUTION

In this chapter, the final solution is presented. As described in section 6.4, the gravity wall around the waste water treatment plant solely is chosen as the final solution. The dimensions of this wall are summarised in table 7.1. A schematisation of the cross-section of the situation, including heights, is shown in figure 7.1. A 3D schematisation of the wall can be seen in figure 7.2. The total costs of the wall around the waste water treatment are determined in section 6.3, resulting in $ 230,705 CUP, which is equal to € 8,901.

Table 7.1: Dimensions of the gravity wall around the waste water treatment plant

Parameter	**Value**
Height [m +MSL]	3.70
Maximum height above the ground [m]	1.70
Width [m]	1.20
Total length [m]	175
Length of a short side [m]	40

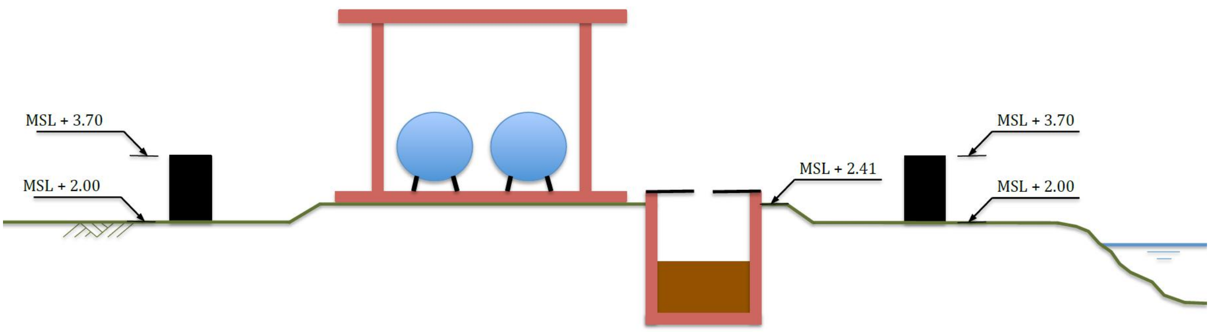


Figure 7.1: Schematisation of the final solution of the gravity wall around the waste water treatment plant (dimensions are in m +MSL)

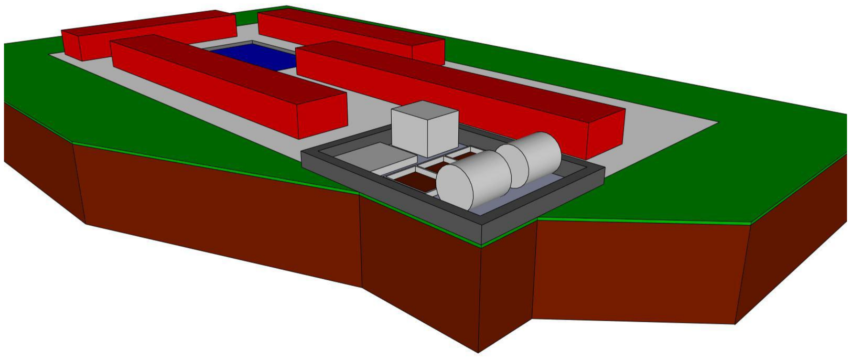


Figure 7.2: 3D schematisation of the final solution of the gravity wall around the waste water treatment plant

Additional elaboration of the different aspects and possible problems that come with the gravity wall around the waste water treatment plant are assessed. They are described briefly below. For a detailed explanation of each aspect, see appendix W.

- **Bearing capacity of the soil**
  Using the Brinch Hansen method, the bearing capacity of the soil underneath the wall is determined. For different clay categories, the calculation is performed. From the results can be concluded that soil improvement is not needed for the sandy clay soil with the gravity wall around the waste water treatment plant.

- **Piping**
  Piping will not be a problem, because the bottom consists of sandy clay. The difference in water head is only present during a period of flooding of 17.5 hours. Therefore, seepage does not have time to become a problem.

- **Drainage of precipitation**
  A submersible pump with a discharge of 8.5 L/s is proposed to pump the total rain volume of 570 m^3 (1/225 year conditions) over the wall. It will start pumping when the water level at the waste water treatment plant exceeds 10 cm.

- **French drainage**
  The maximum freshwater consumption and waste water production per day are used to obtain the volumes during a flooding event. For 1/225 year conditions, the maximum freshwater usage is 120 m^3 and the maximum waste water production is 108 m^3. To make sure that there is sufficient freshwater and waste water storage, the freshwater storage of 320 m^3 should be at least 40% full. An extra waste water storage of 60 m^3 is needed to store all the waste water during a flooding event.

- **Accessibility**
  In order to avoid weak spots in the wall strucure, two stairs are built for accessibility to the waste water treatment plant.

7.1. CONSTRUCTION PLAN

In this section, the steps for construction of the gravity wall around the waste water treatment plant are described. It elaborates on the different project stages and contains instruction on execution methods and the maintenance strategy. First, the time schedule of the entire construction period is shown. This time schedule is a proposal of how the project can be executed (see figure 7.3). The red line in the time schedule indicates the critical path of the construction. The critical path needs to be monitored closely, because when a single event adjacent to this path gets delayed, the whole project will be delayed. The real time schedule can have a different appearance, depending on the contractor.

As can be seen in the time schedule, the construction takes place in three stages. An additional stage is the maintenance, which is part of the lifetime of the structure and includes a preferred strategy. All four stages are explained in appendix X, including a complete visualisation of the steps of the construction of the wall.

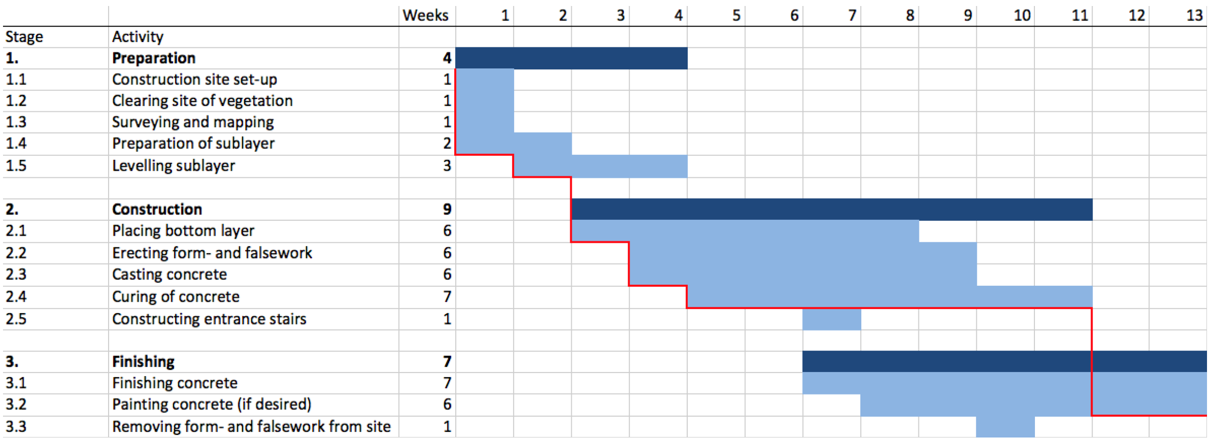


Figure 7.3: The schedule of the construction of the gravity wall around the waste water treatment plant

8

DISCUSSION

8.1. RISK ANALYSIS

According to the primary stakeholder, CITMA, prevention of the mangrove forest contamination caused by flooding of the waste water treatment plant at the Blau Arenal Hotel is the main goal. Only one large flooding event at the area was known. This event happened in 1982, three years before the hotel was built. After analysing the available data with respect to precipitation in the catchment area of the Itabo River, the size of the disaster in 1982 appeared to be much larger than a one in 1000 year event. From the risk analysis followed that the waste water treatment plant should be protected against a one in 225 year event and the hotel against a one in 100 year event. However, after analysis of the inundation depth at the hotel area during a one in 100 year rainfall it was decided not to protect this hotel against flooding. With discharge data of the Itabo River catchment area and with better knowledge of the costs of flooding at a certain inundation level, this trade-off can be made more accurately.

8.2. SENSITIVITY ANALYSIS RIVER DISCHARGE

The river discharge turned out to be important for the (dimensions of the) final solution. However, the reduction of this river discharge is uncertain due to losses of water. For that reason a sensitivity analysis on this reduction percentage is done. The most important aspect is the relation of the river discharge reduction percentage with the height of the wall. In this way, the sensitivity of the model results based on the river discharge can be quantified. Runs are done for a discharge in the model of 60%, 70%, 80%, 90% and 100% of the calculated total discharge, for both the one in 100 year conditions and the one in 225 year conditions. This is done by reducing the time series of the discharge in the XBeach input. In table 8.1 the results of the different reduction percentages are shown.

Table 8.1: Water levels and water depths for different reduction percentages

Return period	1/100			1/225		
Discharge in model	Water level in river [m +MSL]	Water depth at hotel [m]	Water depth at WWTP [m]	Water depth in river [m]	Water depth at hotel [m]	Water depth at WWTP [m]
60%	1.82	-	-	2.03	-	-
70%	2.19	0.06	-	2.49	0.37	0.06
80%	2.50	0.37	0.08	2.74	0.60	0.28
90%	2.80	0.68	0.31	3.09	0.94	0.48
100%	3.11	0.96	0.49	3.41	1.26	0.67

From the table can be deduced that a reduction of 10% in the river discharge gives a reduction in water depth of at least 25%. However, for an event with a larger return period the differences get smaller. For a reduction of 40% no water is flowing to both of the measurement locations. As very little is known about the amount of reduction for discharges with different return periods, a conservative value should be chosen. By taking a reduction of 10% this is done, because of the presence of culverts (see figure 2.15 in section 2.7) and the presence of a 90° bend in the river just before it flows to the hotel area. Also, this 10% is the only reduction made for the whole river. By observing the width of the river more upstream it becomes clear that during a flood wave more water will flow out of the river, and this is also included in this reduction of 10%.

8.3. MODEL CHOICE

The choice to use the XBeach model was made in the first place because of expected importance of the waves. The waves turned out not to be an important factor to cause flooding. Adequate modelling of the river and a flood wave appeared to be more important. This could have been done more exactly in Delft3D Flow, a program made to model flows. XBeach is especially made to model the behaviour of the coast during storm conditions. However, storm conditions did not have a large impact on the coast due to the offshore shoal.

Also, the question if alongshore sediment transport is causing structural erosion at the project site, can better be answered by using Delft3D. In XBeach no alongshore sediment transport was observed. Another grid has to be used to check for this phenomenon; a longer stretch of the coast has to be modelled. With XBeach this would take too much computational time. The grid has to be more coarse, and Delft3D can better solve for this alongshore transport caused by currents. The presence of scarps in October (see figure A.12 in appendix A), at the start of the summer season (October to May) is very common. Seasonal erosion takes place in the winter and rebuilding of the coast starts in the summer season [9]. This can also clearly be seen on the satellite images of the last 14 years, in which the beach width was always smaller in October than it was further on in the summer season.

9

RECOMMENDATIONS

Parts of the study are based on limited and insufficient data. Furthermore, simplifications and assumptions are made in the calculations. In this section recommendations are given, about the most relevant parts that determine the outcome of the calculations, to improve the final outcome.

9.1. RIVER CHARACTERISTICS

GENERAL

Flooding at the Blau Arenal Hotel area is only due to high river discharge, according to the XBeach model results. No river discharge time series are available. Actually there is not a single discharge measurement. The discharge of the river is calculated based on the triangular unit hydrograph method and the rational method. Both peak discharges do not deviate too much, as can be seen in section 2.6. However, these methods used are very rough methods to determine the discharge. Furthermore, the Intensity Duration Frequency precipitation data is used as input in these methods. Based on this data a synthetic rain shower is constructed for several return periods. Relevant questions to be answered about the river characteristics are:

- What is the maximum discharge the river can accommodate? Is it even possible that the calculated discharge can flow through the river? Or are river banks upstream already flooded before the calculated discharge is reached? If that is the case it would have large implications for the real discharge at the Itabo River mouth.
- The rational method assumes a uniformly distributed rain intensity during the concentration time of the catchment area, the concentration time of this catchment area is 2.8 hours. However, a large intensity short duration precipitation might result in a much larger peak discharge. This cannot be calculated with the rational method.

RECOMMENDATIONS RIVER CHARACTERISTICS

As the discharge is one of the most important parameters determining the outcome of the result, more in depth research on the Itabo River discharge is strongly recommended. It is advisable to do field measurements during heavy rain to calibrate the methods used.

9.2. MODELLING

GENERAL

To calculate the flooding level at the Blau Arenal Hotel, XBeach has been used. XBeach is used because of the special wish of the client and the expectation of flooding due to waves. However, due to the presence of an offshore shoal, waves are not of importance. The river discharge is the cause of flooding at the study area. The modelling of measures to prevent flooding of the Blau Arenal Hotel can better be executed with models like Delft3D. Delft3D is more focused on simulating flows, while XBeach focuses on nearshore phenomena. Therefore, Delft3D is more appropriate.

RECOMMENDATION MODELLING

It is recommended to use Delft3D for further research on flooding at the Blau Arenal Hotel area.

9.3. SHOAL

GENERAL

The presence of the offshore shoal determines the nearshore wave height. As modelled with SWAN, waves break on the shoal, which reduces the wave height nearshore. The shoal is put in SWAN by the bathymetry. From this could be concluded that waves are not important with respect to the flooding problems in the study area. However, the presence of the shoal, which functions like a offshore emerged breakwater, is not guaranteed for the future. Especially since the lifetime of the solutions is 50 years.

RECOMMENDATION SHOAL

It is recommended to investigate the possible disappearance of the shoal in the future, because then waves might become of more importance. This research should check whether the shoal consists of rock or loose material.

9.4. COST BENEFIT ANALYSIS

GENERAL

In the final solution the safety level of the Blau Arenal Hotel is loosened due to the low inundation depth. The damage to the hotel will certainly depend on the inundation depth. However, no cost-benefit analysis is done, as there was too little information available. If even low inundation depths during the presented return periods is judged as unacceptable the robust solution of a wall around the whole hotel area can be implemented. Furthermore, the construction costs are estimated based on rough estimations as this data was also insufficient.

RECOMMENDATION COST BENEFIT ANALYSIS

It is recommended to do a more detailed cost benefit analysis before implementing the recommended solution of only a wall around the waste water treatment plant. Furthermore, before constructing one of the walls, a more detailed cost analysis of the different wall structures is advised.

Bibliography

[1] Cubanacan Grupo, ***Hotel Arenal: High resolution photos***, http://www.cubanacan.cu/en/images?slug=arenalname=Arenal, ⟨Website visited on 22nd August 2016⟩.

[2] Lonely Planet, ***Cuba*** (Lonely Planet Publications Pty Ltd, 2000).

[3] Google, ***Google Maps***, http://maps.google.nl, ⟨Website visited on 22nd August 2016⟩.

[4] Centro Nacional del Clima del Instituto de Meteorología, ***Informe***, (2015).

[5] Google, ***Google Earth***, (2016), ⟨Program used on 11th August 2016⟩.

[6] ***Subtropical Storm Number One, Preliminary Report*** (1982).

[7] L.F. Córdova López et. al., ***Informe final de la visita téchnica realizara al hotel Arenal***, (2016).

[8] Projectgroup 199, ***Interview with David, Head of Investment Office of Playas del Este*** (2016).

[9] J. Bosboom and M. J. F. Stive, ***Coastal Dynamics I*** (DAP, 2015).

[10] J. van Lier, ***Wastewater Treatment*** (TU Delft, 2014).

[11] L. Rietveld, ***Dictaat CTB2120 Gezondheidstechniek*** (TU Delft, 2013).

[12] NOAA, ***Tide Predictions, Havana 2016***, (2016).

[13] Servicio Hidrográfico y Geodésico de la República de Cuba, ***Tablas de marea de las costas de Cuba. Mapa de Amplitud de la Marea en las costas Cubanas***, GEOCUBA, Edimar, Agencia de Cartografía Náutica. P5101. La Habana, Cuba, 10 pp (2005).

[14] A. Morales Abreu et. al., ***Informe científico técnico, proyecto Malecón Habanero, especialidad de Oceanografia***, (1995).

[15] BMT ARGOSS, ***Waveclimate data***, http://www.waveclimate.com/cgi-bin/query, ⟨Website visited on 5th September 2016⟩.

[16] S. Solomon et al., ***IPCC, Climate Change 2007: The Physical Science Basis, Contribution of Working Group I to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change***, Cambridge (2007).

[17] R.F. Guerra García et. al., ***Evolution of the texture and composition of beach sand in playas del Este, Havana, Cuba***, Serie Oceanológica. No. 6, ISSN 2072-800x (2009).

[18] Instituto de Oceanología, ***Evaluación de la amenaza de inundaciones en zonas costeras teniendo en cuenta el incremento del nivel del mar previsto***, (1999).

[19] GeoCuba, ***Levantamiento Hotel Arenal*** (2016).

[20] M.T. Jones, ***Centenary edition of GEBCO Digital Atlas*** (2009).

[21] H. H. G. Savenije, ***Hydrology of Catchments, Rivers and Deltas*** (Lecture notes CT5450, TU Delft, 2007).

[22] H. H. G. Savenije, ***Hydrologie lecture notes,*** TU Delft .

[23] J. D. Ruiz Sinoga et. al., ***Estudio geoambiental de la cuenca de guanabo. Cuba. Aproximacion a la problematica del ague,*** (2010).

[24] L. Gonzáles Spíndola et. al., ***Hydrologia superficial para ingenieros,*** (2007).

[25] National Employee Development Staff Soil Conservation Service United States Department of Argiculture, ***Engineering Hydrology Training Series Module 106: Peak discharge,*** (1989).

[26] GeoCuba and Grupo Empresarial, ***Levantamiento Hotel Arenal*** (2016).

[27] S.N. Jonkman and T. Schweckendiek, ***Flood defences Lecture notes CIE5314*** (TU Delft, 2016).

[28] S.N. Jonkman et. al., ***Probabilistic Design: Risk and Reliability Analysis in Civil Engineering*** (TU Delft, 2015).

[29] P. Dayaxny Hernández et. al., ***Análisis de los peligros naturales y antrópicos que inciden en el turismo de Playas del Este*** (Instituto Superior de Technologias Ciencias Aplicadas (InSTEC)).

[30] L.H. Holthuijsen, ***Waves in Oceanic and Coastal Waters*** (Cambridge University Press, 2007).

[31] SWAN team, ***SWAN User Manual*** (TU Delft, 2016).

[32] Deltares, ***XBeach Manual*** (Deltares, 2015).

[33] Direccíon de Presupuestos y Precios del Ministerio de la Construccíon, ***PRECONS: sistema de precios de la construccíon*** (Emprese de Informíatica y Automatizacíon de la Construccíon, 2005).