PROTECTING ST. BERNARD PARISH, NEW ORLEANS
REVISION OF THE COASTAL DEFENCE ZONE

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Protecting St. Bernard Parish, New Orleans

Revision of the coastal defence zone

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Master Thesis
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In Cooperation with:
ARCADIS
And
ComCoast

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Preface

In order to achieve the degree of Master of Science in Civil Engineering a graduation project has to be executed as final part of study Civil Engineering and Geosciences at Delft University of Technology. The Master study Civil Engineering comprises several specialisations. This Master thesis is about coastal defence systems, which is a subject of the specialisation Coastal Engineering.

This document is the final report of my Master thesis research. The research was executed as cooperation between Delft University of Technology, ARCADIS and ComCoast as part of The Road and Hydraulic Engineering Institute of the Directorate-General of Public Works and Water management (RWS DWW). In this report, the hurricane defence system of St. Bernard Parish is analysed and ComCoast like solutions are sought in order to make the area safe for a new “Katrina”.

This research was carried out under the supervision of Prof. drs. ir. J.K. Vrijling. I would like to thank him for his guidance and advice throughout this research. I would like to thank the other members of the committee for their advice and review during the eight months of this research. I also would like to thank ARCADIS for facilitating this research and special thanks are there for ComCoast and again ARCADIS, because they made it possible for me to do a worktrip to New Orleans.

At last, I want to thank my family and friends, who inspired me during my years in Delft and have given me a great preparation for the life after studying.

Mathijs Dijkman,
January 2007
Summary

Introduction

New Orleans is situated in the low-lying wetlands of the Mississippi delta and is enclosed on all sides by water; the river in the south-west, Lake Pontchartrain in the north and in Lake Borgne in the east. Throughout history New Orleans has been subjected to numerous flooding, both due to river flooding or flooding caused by hurricanes. In August 2006, New Orleans was hit again, this time harder then ever by Hurricane Katrina. However, it appears that Katrina was not the big one, which scientist has feared for many years. Therefore, questions arise: Could the disaster been prevented? And, maybe even more important, can New Orleans be protected for a similar attack in the future?

New Orleans has always been protected for a hurricane driven storm surge by multiple lines of defence. There are the natural defences like the barrier islands and the wetlands, behind which are the manmade defences like the dike systems. In the south-eastern part of metropolitan New Orleans lays St. Bernard Parish, this area has a special hurricane protection system. It contains a primary and secondary dike with a transitional wetland in between and long stretches of wetland in front of the primary dike. The coastal defensive system at St. Bernard Parish is very well comparable with the concept of ComCoast – “COMbined functions in COASTal defence zones”. This is a European project, which develops and demonstrates alternative solutions for flood protection in coastal areas. These alternative solutions comprise solutions that are sought beyond conventional defense systems like heightening of dikes. The configuration of the defence system in St. Bernard Parish makes it a particularly valuable study area to determine how a ComCoast solution performs in reality. The ComCoast concept also offers opportunities to rehabilitate St. Bernard Parish coastal defences. Based on the St. Bernard Parish situation the Objectives for this work can be stated.

Objectives:

- Analyse the performance of the St. Bernard Parish’s coastal defence zone
  Analyse the failure on the basis of field observations and a failure calculation. Consider and formulate the lessons learned from the flooding and the performance of the system.

- Develop spatially integrated solutions for the St. Bernard Parish coastal defence zone
  Investigate and evaluate how the conceptual ComCoast solutions would fit in the coastal defence system. This will be done by means of a quick scan on environmental impacts, supported by short calculations. Detect the problems and opportunities that arise with the application of these solutions in this area.

- Investigate the effects of wetlands on a storm surge.
  In order to gain insight whether a wetland leads to a reduction in the surge height or not, an analytical and a numerical approach is made to picture the relevant processes and parameters,
Failure analysis for the St. Bernard Parish dike system

Pre-Katrina observations indicate that most of the dikes in the system were up to design level. Though it must be said that this design level was often met by the use of additional sheet piling. Post-Katrina observations showed heavy erosion and breaching of large parts of the primary dike. It appears that the use of bad non-cohesive construction material had a large contribution on the scale of this erosion. Overtopping and breaching of the primary dike flooded the transitional wetland in an early stage. Eventually the secondary dike was also overtopped causing flooding of the residential areas of St. Bernard Parish. The secondary dike, being constructed of much better construction material, performed much better than the primary dike and suffered only minor erosion.

In the failure calculation three cases have been investigated; 1) Design level = height of the dike crests at the design level, 2) Pre-Katrina = height of the dike crests as observed before Katrina, 3) Post-Katrina = height of the dike crests as observed after Katrina. For these cases both the actual situation and the situation as if there were no secondary dikes, are calculated using spreadsheets. In addition, a sensitivity analysis is performed to determine the sensitivity of the calculation concerning the correctness of the hydraulic boundary conditions.

The calculations showed that a catastrophe could have been averted, if the primary dike did not erode as it did during Katrina. Both the Design level and the Pre-Katrina cases showed no flooding in the residential areas. The calculated flood depth of the Post-Katrina case matched the field observations within the residential area remarkably well, while the other cases did not. This indicated that the erosion of the primary dike took place in an early stage. The results for situations without a secondary dike showed that it could have been highly effective in protecting the residential area. The sensitivity analysis showed that the system is highly sensitive for exceedence in the hydraulic design conditions. A small increase of surge height leads to a serious increase in flood depths.

### Table 0.1

<table>
<thead>
<tr>
<th></th>
<th>Design level (m)</th>
<th>Pre-Katrina (m)</th>
<th>Post-Katrina (m)</th>
<th>Observations (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flood depth</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>wetland</td>
<td>0.54</td>
<td>1.58</td>
<td>3.75</td>
<td>No observations</td>
</tr>
<tr>
<td>Flood depth</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>residential area</td>
<td>No flooding</td>
<td>No flooding</td>
<td>3.41</td>
<td>3.21</td>
</tr>
</tbody>
</table>

Design of St. Bernard Parish coastal defence zone

Five ComCoast alternatives are proposed as coastal defence in St. Bernard Parish:

1. Zero + / Regulated tidal exchange – Heightening of primary dike
2. Overtopping resistant – Primary dike with reinforced revetment
3. Managed realignment – Permanent gaps in primary dike
4. Foreshore recharge – Creation of wetlands in front of primary dike
5. Foreshore protection – Breakwater in front of primary dike
For the five alternatives a quick analysis is made concerning the construction aspects. Basic calculations are performed concerning crest heights and required construction material. The results of this construction analysis can be seen in Table 0.2.

<table>
<thead>
<tr>
<th>Volume dike material (clay)</th>
<th>Zero + / RTE</th>
<th>Overtopp. R.</th>
<th>Managed R.</th>
<th>F. Recharge</th>
<th>F. Protection</th>
</tr>
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<tbody>
<tr>
<td>13.1 x 10^6 m^3</td>
<td>7.9 x 10^6 m^3</td>
<td>16.2 x 10^6 m^3</td>
<td>9.8 x 10^6 m^3</td>
<td>9.8 x 10^6 m^3</td>
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</table>

Subsequently the ComCoast alternatives are evaluated by means of a quick scan on environmental impacts. This evaluation showed that the alternatives; Overtopping resistant and Foreshore recharge, are the best solutions for this case. Foreshore protection and Managed realignment appeared to be no suitable solutions for the St. Bernard area. This was based on the high construction costs and the lack of natural benefits. The conventional Zero+ alternatives scored moderately and can still be seen as an acceptable solution. A good solution might be a combination of the overtopping resistant and the Foreshore protection alternatives.

**Wetlands and Storm Surge**

For the influence of wetlands on a storm surge, two main hydraulic processes can be distinguished; 1) Surge propagation 2) Wind set-up. In the dynamic situation of surge propagation, increased bottom roughness leads to decrease in surge height. However, in the quasi-static situation of wind set-up, increased bottom roughness, leads to an increase in surge height. Which of these two processes is dominant depends on elapsed time, and timescales involved. In order to gain insight in this problem an analytical and numerical solution is formulated for the situation in which both processes play a role.

From the analytical approach it is derived that the system behaves, conform an advection-diffusion relation in which parameters \(c\) and \(K\) describe respectively the advection-coefficient and the diffusion-coefficient. In this relation, \(c\) depends on the friction-coefficients and the wind speed and \(K\) depends on the water depth. The eventual effect of a wetland on a storm surge mainly depends on the length of the wetland, the initial water depth, the storm duration and the roughness of the wetland.

A numerical model has been set-up in Matlab which gives a solution for this problem. A critical storm duration \(\tau_{\text{crit}}\) is introduced accounting the time which is needed for the water elevation at the dike to reach the same height as the ocean’s water elevation. If the storm duration is less then this critical storm duration, the wetland have caused a reduction in the surge height at the dike, if it is longer then the critical storm duration it has led to an increase. (Figure 0.1)

Long stretches of wetland, at least 25km or longer, are required to induce a reduction of the eventual surge heights and even then its effectiveness depends on the initial flood depths, vegetation etc. High initial water depths are very unfavourable for the reduction of the eventual surge heights.
Critical storm duration

**Conclusion and Recommendations**

This study provides no final designs or exact solutions to the problems in St. Bernard Parish. However, it has provided deeper insights in the problems and processes concerning: The flooding of St. Bernard Parish, how it was protected and how the dike system failed. How a ComCoast solution can be implemented in a coastal area and what the possibilities are for such a solution in St. Bernard Parish area. How a storm surge behaves when it passes wetlands and what the processes involved are.

**Conclusions**

- **Failure analysis**  →  Catastrophic failure could have been prevented if the primary dike was in good condition (constructed of good materials and at the design height) In that case the system with a primary dike and a secondary dike should have functioned well.
- **Design**  →  The wide spacious area makes it suitable for application of the ComCoast-concept. However the Hydraulic conditions in the area are not ideal for the ComCoast solutions. From the quick scan on environmental impacts followed that the Overtopping resistant and Foreshore recharge are the best alternatives. It might turn out that a combination of those two is the best solution in St. Bernard.
- **Wetlands & Storm Surges**  →  Wetlands can both lead to an increase as a decrease of the eventual surge heights. Very long stretches of wetland are needed to be effective in surge reduction. Low initial food depths are favourable for surge reduction.

**Recommendations**

- Further investigation and elaboration on the ComCoast concepts is needed.
- ComCoast should look for Case-study areas with more suitable hydraulic conditions.
- Further elaboration on the effect of wetlands on storm surges is advisable.
- It is advisable to adapt an approach which is more based on damage control instead of damage prevention.
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In the southeast of the city of New Orleans, in St. Bernard Parish lies a coastal defensive area, which is formed by a primary dike, a secondary dike and a transitory wetland area in the middle. During Hurricane Katrina on the 29th of August 2005, a huge storm surge from the east overtopped and destroyed large sections of the primary dike. The surge continued through the wetland overtopping the secondary dikes and flooded large parts of St. Bernard Parish. The flooding killed over a hundred people and made many homeless.

The coastal defensive system at St. Bernard Parish is very well comparable with the concept of ComCoast – “COMbined functions in coastal defence zones”. This is a European project, which develops and demonstrates alternative solutions for flood protection in coastal areas. One of its main solutions is to use a wide coastal defence zone, containing a transitional area between an overtopping resistant primary dike and a lower protective secondary dike. The transitional zone will be a buffer for the storm surge and will be an area suitable for multipurpose use, with great opportunities for both man and nature.
1.1 COASTAL LOUISIANA: MULTIPLE LINES OF DEFENCE AREA

The metropolitan city of New Orleans is separated from the Gulf Mexico by more than 100 kilometres of low lands and shallow seas. This means that a hurricane and its storm surge approaching from the Gulf of Mexico will have to cross these wetlands and sounds before it hits the city’s most important manmade sea defence, the dikes. The configuration of this long stretch of coastal land will largely affect the hurricane’s wind, waves and surge set-up, before it hits these dikes. This is why it is often said that New Orleans is protected by multiple lines of defence.

In the paper “The multiple lines of defence strategy to sustain Louisiana’s coast”, by Lopez, J.A. 2005 [10], coastal Louisiana is presented as a multiple line of defence system. In this paper 11 lines of defence are distinguished: 1st Offshore shelf, 2nd Barrier island, 3rd Sound, 4th Marsh land bridges, 5th Natural ridges, 6th High found./spoil, 7th Flood gates, 8th Flood dikes, 9th Pump stations, 10th Elevated homes, 11th Evacuation routes.

In Figure 1.2 These lines of defence are presented in a cross-profile view.

![Multiple lines of defence cross-profile](image)

The first five lines of defence can be seen as the natural defences, like barrier islands and marshes. The next four lines can be seen as the manmade defences, like the dikes and floodgates. The last four can be seen as damage control, like elevated homes and evacuation routes. (Figure 1.2). Figure 1.3 on the next page shows the multiple lines of defence in a map of coastal Louisiana. The red arrows show the paths of the surge onto the St. Bernard coastal defence zone and lines of defence it had to cross, before it reached the dikes of St. Bernard parish.

The natural defences in coastal Louisiana are disappearing at an alarming rate. In the last decennia, enormous amounts of wetlands and barrier islands were lost to the Gulf of Mexico. It is often said that the disaster caused by Katrina can be largely contributed to this excessive wetland loss. However, the exact influence of a wetland on a storm surge is not known.
In this coastal Louisiana, multiple line of defence system lies the St. Bernard Parish dike system (See Figures 1.1 & 1.3). They are manmade defences, which should provide the ultimate defences to the residential areas of St. Bernard Parish. Especially this dike system is very much comparable with the ComCoast principle. The residential areas are protected by a primary and a secondary dike, with a transitional wetland in between. (Figure 1.4 & 1.5) You can speak of a multifunctional transitional defensive zone, or of a ComCoast like; “coastal defence zone”. However, the development of the configuration of this dike system was actually more a coincidence, then a well thought application of a sophisticated defence strategy of the ComCoast type.
1.2 PROBLEM DESCRIPTION

It appears that Katrina was not the “big one” which many scientists had feared for years, so questions arise: could the disaster been prevented? And maybe even more important, can New Orleans be protected for a real big one? Another interesting question is what the impact is of the disappearing wetlands.

In St. Bernard Parish, a ComCoast like coastal defence zone failed with large consequences. It would very useful to implement the knowledge and experience of ComCoast in the area of St. Bernard Parish, in order to make it safe for future hurricane attacks. On the other hand, ComCoast has no experience in what happens if the coastal defence zone fails and what happens if the dikes are heavily overtopped. Also for ComCoast, it would be wise to evaluate the St. Bernard area and learn the lessons, which can be learned from the performance of St Bernard Parish’s dike system.

1.3 OBJECTIVES

The three main objective of the thesis will be:

- **Analyse the performance of the St. Bernard Parish’s coastal defence zone**
  
  Analyse the failure on the basis of field observations and a failure calculation. Consider and formulate the lessons learned from the flooding and the performance of the system.

- **Develop spatially integrated solutions for the St. Bernard Parish coastal defence zone**
  
  Investigate and evaluate how the conceptual ComCoast solutions would fit in the coastal defence system. This will be done by means of a quick scan on environmental impacts, supported by short calculations. Detect the problems and opportunities that arise with the application of these solutions in this area.

- **Investigate the effects of wetlands on a storm surge.**
  
  In order to gain insight whether a wetland leads to a reduction in the surge height or not, an analytical and a numerical approach is made to picture the relevant processes and parameters,
1.4 RESEARCH APPROACH

This report is divided in six chapters, starting with the Introduction in chapter 1. Chapter 2 provides the background information needed for the research. It contains an outline of the situation and it treats the most important (hydraulic) processes. Subsequently, chapter 3, 4 and 5, are dedicated to the three research objectives. Therefore, chapter 3 analysis the failure of the St. Bernard coastal dike system. This is done by the hand of available field observations and a failure calculation. Chapter 4 evaluates a few coastal defence solutions for the area, inspired on the ComCoast concepts. Chapter 5 elaborates on the effect of wetlands on storm surges. It includes an analytical and a numerical approach. Finally in chapter 6 the conclusions and recommendations are given, following from this research.

Figure 1.6 gives the schematic approach of the research.
In this chapter, the background information is given to support this research. This includes, an introduction on New Orleans, St Bernard Parish and Katrina in paragraph 2.1. In paragraph 2.2 the ComCoast concept will be explained. Paragraph 2.3 finally will give attention to the physical processes, which play a role in this research, i.e. the hydraulic processes, dike failure mechanisms and related formulas.

2.1 NEW ORLEANS, ST. BERNARD PARISH AND KATRINA

In the fall of 1718 Frenchman Jean Baptiste le Moyne founded New Orleans on the banks of the Mississippi river. As the hurricane season had just passed and the land was dry he had no idea that in the future the port city would be subjected to numerous floods from the Mississippi river and of hurricanes.

New Orleans grew to mayor city with a population of over a million people. The metropolitan area, composed of Orleans, Jefferson, St. Charles, St. Bernard, and St. Tammany Parishes, sits in the tidal lowlands between the Mississippi in the south and Lake Pontchartrain in the north. Lake Pontchartrain is a tidal basin, which is about 1650 square kilometres and connected with the Gulf of Mexico by the Mississippi sound and Lake Borgne.
Historically New Orleans experienced many river floods, like the Great Flood of 1927, where 246 people died and 720,000 were displaced. Through the years, construction of series of dikes and other flood control structures along the Mississippi river have reduced this threat. Since then the greatest natural threat to New Orleans continued to be, the by hurricanes induced; storm surge, waves and rainfall. Several hurricanes have struck the area over the years, including Hurricane Betsy in 1965, Hurricane Camille in 1969, Hurricane Lily in 2002 and of course recently, Hurricanes Katrina and Rita in 2005. The hurricane’s storm surge is known as the biggest killer, along the Gulf Coast it can develop heights of 6m (20ft) or more, wrecking everything, which comes in his path. The continuing loss of protective wetland and subsidence, at this moment large parts of the metropolitan city are below sea level, has made the New Orleans area even more vulnerable to these storms.

In “The Lake Pontchartrain and Vicinity Hurricane Protection Project”, which was authorized in “The Flood Control Act” of 1965, the construction of a series of control structures, concrete floodwalls and earthen dikes was proposed to provide hurricane protection, to the New Orleans’s Parishes. The US Army Corps of Engineers was responsible for project design and construction; local interests were responsible for the maintenance and control of the dikes and flood controls.

The project designs were dimensioned on a storm that might strike the area once in a 200-300 years, it was determined that this came down to a storm roughly equivalent to a fast-moving category 3 Hurricane. (See Table 2.1 for explanation on hurricane categories) St. Bernard Parish and the neighbouring part of the Lower 9th Ward were some of the hardest hit parts of New Orleans. It is the area between the Mississippi river in the southwest, the Orleans Parish in the north, Lake Borgne in the east and the wetlands of the Mississippi delta in the south. The area contains of 210 km$^2$ of polder, which is protected by a complex dike system. The polder has elevations between the -1.2 to 3.7m + MSL (-4 to 12ft), with the higher parts in the southwest near the Mississippi river. In the south and west are the two residential areas. These areas had a pre-Katrina population of about 65,000 people, six months after Katrina, only 12,000 of those have returned.
2.1.1 HURRICANES AND STORM SURGES

Hurricanes are tropical cyclones formed in the Atlantic basin, with counter clockwise rotating surface winds of 33 m/s or greater. A tropical cyclone has a much lower air pressure in its centre (the eye) than in his periphery; this is what causes the strong spiralling winds towards the eye. Hurricanes develop from tropical storms as they gain strength above warm ocean water, they are classified by wind strength using the Saffir-Simpson Hurricane Scale, as shown in Table 2.1. Hurricanes are well known to do a lot of damage, caused by its strong winds, heavy rainfall, high waves and storm surges.

<table>
<thead>
<tr>
<th>Scale (Category)</th>
<th>Pressure (Mbar)</th>
<th>Winds (Mph)</th>
<th>Winds (m/s)</th>
<th>Surge (m)</th>
<th>Hs (m)</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&gt; 980</td>
<td>74-95</td>
<td>33-42</td>
<td>4 to 5</td>
<td>4-8</td>
<td>Minimal</td>
</tr>
<tr>
<td>2</td>
<td>965-979</td>
<td>96-110</td>
<td>43-49</td>
<td>6 to 8</td>
<td>6-10</td>
<td>Moderate</td>
</tr>
<tr>
<td>3</td>
<td>945-964</td>
<td>111-130</td>
<td>50-58</td>
<td>9 to 12</td>
<td>8-12</td>
<td>Extensive</td>
</tr>
<tr>
<td>4</td>
<td>920-944</td>
<td>131-155</td>
<td>59-69</td>
<td>13 to 18</td>
<td>10-14</td>
<td>Extreme</td>
</tr>
<tr>
<td>5</td>
<td>&lt; 919</td>
<td>&gt; 155</td>
<td>&gt; 70</td>
<td>&gt; 18</td>
<td>12-17</td>
<td>Catastrophic</td>
</tr>
</tbody>
</table>

A deficit of the Saffir-Simpson Hurricane scale is that it does not includes a distinction in storm size (radius) and propagation speed. Both can have a large influence on hurricane impact.

A storm surge is a fast water level rise driven by a hurricane as it approaches the coast. Low air pressure and strong winds continually pull water up towards the centre of the storm. The height of a surge depends strongly on the bathymetry of the shore face. If the hurricane is still above deep water, the surge is not able to develop, as the water can be dispersed down as a return flow, away from the hurricane. If a hurricane passes a long shallow shore face before it reaches the shore, the water cannot escape downwards anymore and the surge grows (Figure 2.8).

The height of a storm surge, in a real complex geometry, depends on so many things that it’s very difficult to predict. Some complex calculation models do exist who can calculate the surge heights quite accurate. (Delft 3D, ADCIRC and SLOSH). A lot of input information like hurricane characteristics, bathymetry and tides are needed to run such models.

Figure 2.8
The growing of a surge [6]
The official hurricane season for the Atlantic Ocean, the Caribbean Sea and the Gulf of Mexico runs from 1 June to 10 November. The Peak of the season is from mid-August to late October. However, deadly hurricanes can occur any time in the hurricane season.

The National Oceanic & Atmospheric Administration (NOAA) has assigned categories to all hurricanes that have made landfall on the mainland of the US coastline since 1851. Between 1851 and 2004, 49 hurricanes category 1 or greater have made direct hits on the Louisiana coastline of which 18 of them where category 3 or greater.

2.1.2 HURRICANE KATRINA AND ST. BERNARD PARISH

The day before Katrina hit, high water elevations created by the storm’s outer bands already flooded the low-lying wetlands outside the dike system. (Figure 2.9A) The strong wind pushed Katrina’s storm surge up against the primary dike. After a while, the dike sections along the MRGO were overtopped and heavily damaged. The water from Lake Borgne advanced into the wetlands towards the residential areas of St. Bernard Parish. (Figure 2.9B) The southern dike, protected by a large stretch of wetland, did withstand the surge from the south.

As the hurricane preceded the surge, build up in the Intracoastal Waterway’s funnel and into the Industrial Canal. (Figure 2.9C) The floodwalls were overtopped on both sides and the water was still rising. Eventually two floodwall sections on the eastside of the Industrial Canal fell (Figure 2.9D), releasing a wall of water into the Lower 9th Ward, tossing homes and cars around like toys. The water proceeded more or less half way the western residential area of St. Bernard Parish.
The water from Lake Borgne still advanced into St. Bernard Parish’s wetland and eventually the second line of defence, the 40-Arpent Canal dike was overtopped and the remaining parts of St. Bernard Parish were flooded. Less than three hours after Katrina made landfall in Louisiana, entire St. Bernard Parish was flooded, while the large flooding of the New Orleans Parish was yet to come. (Figure 2.9E)

2.1.3 COASTAL LOUISIANA’S WETLANDS

Hurricane Katrina made landfall in southern Louisiana as a category 4 but by the time it passed New Orleans it already shrunk to a category 3 storm. Healthy wetlands with trees and other vegetation cause high friction on the winds, slowing them down. In this way, long stretches of wetland have a good ability to limit a hurricane’s inland destruction. Healthy swamplands with lot of (cypress) trees are supposed to be more effective than poorly vegetated grassy marshlands.

Wetland preservation or even restoration seems essential in the area. At this moment, it is highly unlikely that New Orleans will be hit by a full category 5 storm as it is still surrounded by its large quantities of protective wetland, which the storm has to cross before it can hit the city. However, as the wetlands are disappearing, at the enormous rate of two football fields every hour, it is loosing its protective capability. (Figure 2.10)

It is often said that wetlands are well capable of dissipating storm surge energy leading to lower surge elevations. Some scientists say that every four miles of wetland is able to absorb 0.3 m (1ft) of storm surge. Other scientists are rather sceptical, they say that, when flooded, the wetlands will only increase wind set-up, leading to a higher surge elevation. In chapter 5 special attention will be given on the influence of wetlands on a storm surge.
2.2 COMCOAST

ComCoast – “COMbined functions in COASTal defence zones” is a European project, which develops and demonstrates alternative solutions for flood protection in coastal areas, trying to address new functions to those areas. Rijkswaterstaat, a part of the Dutch Ministry of Public Works and Water Management, is the leading partner; other participating countries besides the Netherlands are Denmark, Great Brittan, Belgium and Germany.

In the coming years climate change will increase the physical loads on coastal defences all over the world. Traditionally the Netherlands has protected it selves against the growing threat of flooding, by heightening our dikes. However, with the continuing sea level rise, it becomes more and more evident to find alternative and innovative strategies, without just heightening our dikes. ComCoast develops such flood risk management strategies, with gradual transitions from sea to land, in order to create integrated defensive zones including wider environmental functions, such as recreation, fisheries, tourism and nature creation.

The wide defensive ComCoast concept is formulated as a “coastal defence zone” which contains two principles for this spatial water defence:

1. Load reduction → foreshore sea defence zone → reduction wave attack (blue Figure 2.11)
2. Load admitting → landward sea defence zone → allow and handle water behind the dike (red Figure 2.11)

ComCoast’s concept of a coastal defence zone has let to the appointment of technical functions and components. They will be treated in the next paragraph.

2.2.1 COMCOAST TECHNICAL FUNCTIONS AND COMPONENTS

The ComCoast project searches for alternative coastal defensive solutions using a multiple line of defence strategy. In comparison with a single line defence, a coastal defence zone has a range of components (lines) each with its own function. First these technical functions and its components are formulated from which the main ComCoast solutions can derived.

Water retaining
The primary dike retains high sea levels and wave run-up, up to the design level. The inner slope can have an overtopping-resistant revetment, which permits a greater overtopping discharge.
Water storage
The area behind the primary dike is a transitional area able to store the overtopping seawater. A secondary dike or higher grounds, encircles the transitional area. Or the water is handled by large ditches or pumping stations.

Water control / management
During storms and in normal weather conditions the coastal defence zone should be able to drain off water when necessary. First, a drainage system facilitates water control in the transitional area. For larger quantities of water, a pump installation can be installed to support the discharge of water by the drainage system. If desired, a culvert can be added to increase tidal influence in the transitional area. A culvert can also be used to drain off excessive salt water after a storm.

Wave reduction
Several elements in front of a dike yield wave reduction. First, a shallow foreshore creates a moderate wave climate in front of the dike. In addition, wave reduction can also be achieved when there is a previously constructed lower dike, a breakwater or a summer dike.

Multifunctional use of area
The transitional area can be used for several purposes, for example aquatic sport, recreation, the development of aquatic areas and to enhance environmental values. This is only the case when the area is flooded regularly. This can also be obtained by Managed realignment.

In Table 2.2 and Figure 2.12 these functions and components are summarized and sketched.

<table>
<thead>
<tr>
<th>Function</th>
<th>Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water retaining</td>
<td>Primary dike</td>
</tr>
<tr>
<td></td>
<td>Secondary dike</td>
</tr>
<tr>
<td>Water storage</td>
<td>Drainage system</td>
</tr>
<tr>
<td></td>
<td>Culvert facility</td>
</tr>
<tr>
<td></td>
<td>Pump installation</td>
</tr>
<tr>
<td>Water control / management</td>
<td>Former dike</td>
</tr>
<tr>
<td>Wave reduction foreshore</td>
<td>Former dike</td>
</tr>
<tr>
<td></td>
<td>Breakwater</td>
</tr>
<tr>
<td></td>
<td>Summer dike</td>
</tr>
<tr>
<td></td>
<td>Shallow foreshore</td>
</tr>
<tr>
<td>Multifunctional use of area</td>
<td>Transitional area</td>
</tr>
</tbody>
</table>

Table 2.2
Functions and components of a coastal defence zone

Figure 2.12
Systematic overview of ComCoast concept components
2.2.2 COMCOAST SOLUTIONS

The distinguishing of the functions and components as discussed in the previous paragraph has led to five main ComCoast-concepts:

Landward solutions:
- **Regulated tidal exchange** is the regulated exchange of seawater to an area behind fixed sea defences, through engineered structures such as sluices, tide gates, or pipes to create saline or brackish habitats.
- **Managed realignment** involves the placement of new Managed realignment flood defence landward of the existing flood defences. This would be achieved through the partial or complete removal of the existing flood defences.
- **Overtopping resistant dike** involves the replacement of the top of the dike and its inner slope with a revetment that will not wear away by severe overtopping. The overtopped sea water will be handled in the coastal zone at the landward side of the dike (drainage/storage).

Seaward solutions:
- **Foreshore recharge** involves the placement of material in front of the existing coastal defence system.
- **Foreshore protection** involves reclamation works to maintain or to create higher ground and in some situation small dikes in front of the primary dike, which act as breakwaters in case of a big storm.

Combinations of the defensive schemes sketched above are also good possibilities.
2.3 PHYSICAL PROCESSES

Hurricanes approach coastal Louisiana over the warm ocean water from the Gulf of Mexico. Its heavy winds and low atmospheric pressures generates high surge and waves. As the hurricane reaches the coast, the geometry of its shallow near shore will start to affect the storm winds, waves and surge elevations, finally leading to the boundary conditions at the location of the hurricane protection dikes. Known processes inside this near shore area are i.e.: Wind friction, wind set-up, wave generation, wave reduction and wave set-up. In Figure 2.15 these physical processes are formulated in a hydraulic scheme leading to the design loads at the coastal defence zone. These processes will be discussed in the next paragraphs. Thereafter the hydraulic processes are discussed which take place inside the coastal defence zone, i.e. the processes set-up, run-up and overtopping. Finally the involved formulas will be given.

Figure 2.15
Hydraulic scheme for a hurricane/storm surge [2]
Wind & atmospheric pressure
As a hurricane makes landfall and starts to move over land, it misses its natural propulsion, the warm ocean water. This means that the atmospheric pressure will start to drop, slowing down the spiralling winds. The friction of the land is much higher than the friction of the sea surface; this will also cause the winds to slow down. A long roughly vegetated stretch of land will be more effective in reducing a storm’s proportions than a small poorly vegetated strip of land.

Waves
Waves are affected by several coastal landscape properties. These properties include the water depth, bottom roughness, water column friction and bottom sediment characteristics.

As waves propagate into shallow water the effect of the water depth becomes fundamental. Shallow water processes includes generation, shoaling, refraction, diffraction, reflection, breaking, set-up, run-up, bottom friction, water column friction and dissipation of wave energy through wave/bottom interaction. The water depth can be particularly important when it causes wave breaking.

The wave energy loss due to vegetation depends on the drag force on the wave current induced by the plants. The amount of energy loss depends on the geometry of the individual plants and on the number of plants.

Surge elevation
It is often said that long stretches of wetland are well capable of absorbing the energy of a storm surge. The friction caused by the vegetation dissipates the energy in the surge, leading to decreased surge heights. The amount of decrease depends mainly on the type of vegetation, inundation depth and length of the wetland. Studies about surge reduction by wetland in the Louisiana coastal area indicate that 200-250 m of wetland is capable in reducing the surge elevation 1 cm.

Another study by Stone et. al 2003 [16], indicated that surge elevations greater the 4.6 m (15ft) where hardly effected by the decay of the wetlands in coastal Louisiana. Those findings might imply that the influence of submerged wetland decreases as the surge height increases.

This can be explained by the fact that wetlands will increase wind set-up if the surge is large enough to flood the wetlands in an early stage. Roughness of the wetland will now only increase the set-up as is it retards the return current.

The influence of the wetlands on the storm surge is a rather difficult problem. In chapter 5 this will be elaborated and further insight will be given into the processes which take place.
2.3.1 HYDRAULIC PROCESSES INSIDE THE COASTAL DEFENCE ZONE

The near shore processes described in the previous paragraph will lead to the boundary conditions at the edge of a coastal defence zone. It has a form of a storm water level excitation in time \( h_{SWL} \), topped with waves with a certain significant wave height \( H_s \) and peak period \( T_p \).

In case of a shallow foreshore, waves will start to break, dissipating the wave energy. The breaking of waves will cause an additional (wave) set-up in the surge elevation. In this study this effect of wave set-up is not calculated, but it must be said that it can be rather substantial. The influence of the foreshore on the surge elevation is treated separately in chapter 5.

As the waves reach the dike’s slope the waves will run-up onto the slope and in some cases will overtop the crest. Then water will be admitted into the storage area behind the primary dike with a flow rate \( q \). Local winds will generate waves and will cause set-up against the secondary dike. If the secondary dike is not high enough, water will be admitted into the hinterland, and if that happens the coastal defence zone has failed.

In Figure 2.16 and Table 2.3, these processes are sketched giving the important parameters and boundary conditions.

<table>
<thead>
<tr>
<th>Relevant strength parameters</th>
<th>Boundary conditions</th>
<th>Other parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest level primary dike ( h_{c,1st} )</td>
<td>Wave height ( H_i )</td>
<td>Water level polder ( h_{wp} )</td>
</tr>
<tr>
<td>Geometry primary dike</td>
<td>Wave period ( T_p )</td>
<td>Surface level polder ( h_p )</td>
</tr>
<tr>
<td>Strength of the revetment</td>
<td>Wave direction</td>
<td>Overtopping discharge ( q )</td>
</tr>
<tr>
<td>Storage volume ( V_p )</td>
<td>Duration of the storm ( t_s )</td>
<td>Water level polder at toe of</td>
</tr>
<tr>
<td>Crest level secondary dike ( h_{c,2nd} )</td>
<td>Storm Water Level ( SWL )</td>
<td>secondary dike ( h_{wp,2nd} )</td>
</tr>
<tr>
<td>Angle of the outer slope ( \alpha )</td>
<td>Wind speed ( U_w )</td>
<td>Fetch length in the polder ( L )</td>
</tr>
</tbody>
</table>

In paragraph 2.3.4 all formulas are given corresponding to the processes which take place inside the coastal defence zone. First a conceptual dike’s configuration and its failure mechanisms are treated in the next paragraphs.
2.3.2

DIKE CROSS-PROFILE

In Figure 2.17 the most important elements of the cross-profile of a sea dike are drawn. Not all sea dike designs look the same, i.e. the berm is optional and also an inner berm is possible.

![Figure 2.17 Example of possible dike design](image)

The most important design parameter of a dike is the crest height; it determines the dike’s capability in retaining high water levels. The crest height primarily determines the overtopping rate. Other elements that influence the overtopping rates are: a wide outer berm, a rough revetment or a gentle outer slope. In Figure 2.18, is schematically shown how the construction crest height is usually determined in the Netherlands.

![Figure 2.18 Determination of design crest height](image)

In the New Orleans dike system sheet pile walls are often used placed into a dike’s crest in order to achieve the desired design height. Such floodwalls are used along channels which go down into the heart of the city.
2.3.3 FAILURE MECHANISMS OF COASTAL DEFENCE ZONE

In Figure 2.19 the Fault tree of a coastal defence zone is given. Failure of a coastal defence zone as in St. Bernard Parish occurs if the secondary dike fails due to piping (P), instability (I), erosion (E) or overtopping. Failure due to overtopping of the secondary dike can only be the case if the water level inside the polder exceeds the design water level. This can only be the case if the primary dike failed or by failure of the storage capacity. Again, failure mechanisms piping (P), instability (I), erosion (E) and overtopping can be the cause of failure of the primary dike or the exceedance of the storage capacity. In the next paragraphs, the separate failure mechanisms are discussed briefly.
**Piping**

A difference in water levels between the inner side and outer side of the dike causes groundwater to flow through the more permeable layers of soil underneath the dike. When the difference in water heights exceeds the critical vertical seepage length, uplifting of impermeable clay layers will occur, causing cracks. Water will start to flow through these cracks, transporting sand out of the foundation of the dike and pipes can start to develop, endangering the dike’s stability.

**Instability**

Instability occurs when the stability of a large portion of a dike fails. The stability of a dike is often approached by the slip-circle analysis, i.e. the Bishop method. The method is based on the idea that a slip-circle fails when the driving moment $M_d$ of the load due to the soils weight, exceeds the resisting moment $M_r$ of the soils shear stresses along the circle. Instability can occur on both the inside and outside slope of a dike.
Wave overtopping and overflow
The process in which the water level is still lower than the crest height, but waves forces water to flow over the dike’s crest, is called wave overtopping. When the water level exceeds the crest height, the process is called overflow. There are two ways in which overtopping or overflow can lead to failure: Erosion of the inner slope and failure of the storage capacity.

Erosion
A dike fails if the revetment is damaged and subsequently the body of the dike erodes. Erosion of the revetment can be caused by heavy wave attack caused by a storm or by the erosion of the inner crest due to overtopping or overflow. Failure occurs if the time needed to damage the revetment and the time needed to erode the base is assumed to be shorter than the duration of the storm.

Failure of the storage capacity
The area of the polder between the dikes and the crest heights determine the amount of storage capacity. If too much water is admitted over the primary dike, whether it is to overtopping, erosion, piping or instability, the storage capacity will be exceeded and leading to failure of the coastal defence zone. Failure of the primary dike will not automatically lead to failure of the storage capacity.
HYDRAULIC FORMULAS

The next paragraphs give the relevant formulas corresponding to the hydraulic processes which take place within the coastal defence zone as described in paragraph 2.3.1 Viz.: Wave breaking, wind set-up, run-up, wave, overtopping, overflow and storage capacity.

**Wave breaking**

When waves approach shallow grounds, bottom friction will decrease its speed and wavelength, as a result, the waves will get higher and steeper, this process is called shoaling. At a certain point the waves will get to steep and they will start to break and a lower wave is admitted. When waves cross a shallow area, the height of the admitted waves strongly depends on the water depth. For preliminary design calculations, Schiereck 2004 [12] gives the following rule of thumb:

\[
\gamma = \frac{H_s}{h};
\]

Where:
- \( \gamma \) = Breaker index \( \gamma = 0.6 \) [-]
- \( H_s \) = Significant wave height [m]
- \( h \) = Water depth [m]

**Wind set-up**

When the wind blows over water, the friction between the air and the water, causes a force on the water this force is compensated by gradient in water height. See Figure 2.22. In Baars 2003 [2] the total wind set-up in the equilibrium state can be approached by:

\[
S = C_s \frac{u^2}{gd} \cdot L; \text{ if } S << D
\]

In which:
- \( S \) = total wind set-up [m]
- \( C_s \) = Friction coefficient \( = 2 \times 10^{-6} \) (wind drag and bottom) [-]
- \( d \) = water depth [m]
- \( u \) = wind speed [m/s]
- \( L \) = Fetch length [m]
**Run-up**
As waves reach a dike they will be pushed against the outer slope. This process is called run-up. In TAW 2002 [18] run-up is given as $Z_{2\%}$ which is the run-up height which is exceeded by 2% of the incoming waves. A rule of thumb to calculate the run-up height is given by:

$$Z_{2\%} = H_s \cdot \gamma \cdot \xi^2;$$

In which:
- $Z_{2\%}$ = Run-up height of 2% of the waves [m]
- $H_s$ = Significant Wave height [m]
- $\gamma$ = Combined reduction factor [-]
- $\xi$ = Breaker index [-]

**Equation 2.3**
Run-up

**Figure 2.23**
Wave run-up

**Wave overtopping**
When the run-up height exceeds the crest height wave overtopping will occur. A model to determine the overtopping is given by van der Meer as described in TAW 2002 [18]. It describes overtopping for breaking and non-breaking waves. Waves are breaking if the breaker index $\xi < 2$ and waves are non-breaking if $\xi > 2$. The breaker index $\xi$, can be calculated by:

$$\xi = \frac{\tan \alpha}{\sqrt{S_{op}}};$$

Where:
- $\xi$ = Breaker index [-]
- $\tan \alpha$ = Angle of the outer slope of the dike [-]
- $S_{op}$ = Wave steepness [-]
The wave steepness is given by:

\[ S_{op} = \frac{2\pi \cdot H_s}{g \cdot T_p^2}; \]

Where:
- \( g \) = Acceleration of gravity \([\text{m/s}^2]\)
- \( H_s \) = Significant Wave height near toe of the slope \([\text{m}]\)
- \( T_p \) = Wave period \([\text{s}]\)

**Breaking waves:**
In the case of breaking waves the overtopping discharge \( q_0 \) is given by:

\[ q_0 = Q_b \sqrt{g \cdot H_s^3 \cdot \frac{\tan \alpha}{S_{op}}}; \]

Where:
- \( q_0 \) = Overtopping discharge \([\text{m}^3/\text{s}]\)
- \( Q_b \) = Dimensionless overtopping discharge for breaking waves [-]
- \( H_s \) = Significant wave height near toe of the slope \([\text{m}]\)
- \( \tan \alpha \) = Angle of the outer slope of the dike [-]

The dimensionless overtopping discharge coefficient \( Q_b \) for breaking waves can be determined by:

\[ Q_b = 0.06 \exp \left( f_b \cdot \frac{h_w - h_c}{H_s} \cdot \sqrt{S_{op}} \cdot \frac{1}{\tan \alpha \cdot \gamma} \right); \]

Where:
- \( f_b \) = Factor for breaking waves [-]
- \( h_c \) = Crest level of the dike \([\text{m}]\)
- \( h_w \) = Water level \([\text{m}]\)
- \( \gamma \) = Combined reduction factor [-]

The combined reduction factor \( \gamma \) is composed out of reduction factors related to the angle of wave attack, the existence of a berm and the roughness of the outer slope. More information about these reduction factors can be found in the TAW 2002 technical report on “Wave Run-up and Wave-overtopping for dikes” [18].

The formula’s for wave overtopping only apply for \( h > h_w \). Else also overflow plays an important role. See Equation 2.9, for combined wave overtopping and overflow.
Non-breaking waves:

\[ q_o = Q_o \sqrt{g \cdot H_s^3} \]

Where:

- \( Q_o \) = Dimensionless overtopping discharge coefficient for non-breaking waves

\[ Q_o = 0.2 \exp \left( f_n \cdot \frac{h_n - h_c}{H_s} \cdot \frac{1}{\gamma} \right) \]

Where:

- \( f_n \) = Factor for non-breaking waves

The factors for breaking and non-breaking waves are presented in Table 2.4.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Mean</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_b ) - Breaking waves</td>
<td>5.2</td>
<td>0.55</td>
</tr>
<tr>
<td>( f_n ) - Non-breaking waves</td>
<td>2.6</td>
<td>0.35</td>
</tr>
</tbody>
</table>

The formula‘

Overflow

When the water level on the outer side of the dike exceeds the dike’s crest height water will flow over the dike. The amount of water which will flow over the dike can be calculated with the Weir-formulas in TAW 2003 [19] :

\[ q_o = m \sqrt{g \cdot (h_n - h_c)^{3/2}} \]

In which:

- \( q_o \) = Overflow discharge \([m^3/s/m]\)
- \( m \) = Discharge coefficient = 0.55 \([-]\)
  (for a round weir with slopes on both sides)
Combination of wave overtopping and overflow

In the case that during a storm surge the water level exceeds the crest height, wave overtopping and overflow combined determines the overtopping flux over the crest. If the revetment is strong enough and a large storage basin exists behind the sea dike those overtopping values might be acceptable.

In TAW 2003 [19] a relation is given in order to determine this overtopping flux. The relation exists of two parts, a part for the maximum wave overtopping and a part for the overflow.

For the combined overtopping flux is found:

\[
q_w = 0.13 \cdot \sqrt{g \cdot (H_c)^{3/2}} + 0.55 \cdot \sqrt{g \cdot (h_w - h_c)^{3/2}};
\]

Flux by: wave overtopping overflow

Allowable overtopping

The amount of overtopping water that can be permitted over a dike depends on the construction of the dike and the condition of the inner revetment. For the Netherlands the allowable overtopping values for dike bodies with regular non-reinforced grass revetments are presented in the TAW 2002 [18].

- 0.1 l/s/m → on sandy structures with a bad grass cover
- 1.0 l/s/m → on clayish ground with a reasonable grass cover
- 10 l/s/m → on a clay cover with a good clay cover confirm the demands of the outer revetment

Storage capacity

The storage capacity of the transitional area between a primary and secondary dike is determined by the Storage Surface area, \( A \) and the Maximum Acceptable Water level in the transitional area \( h_{MAW} \). The Maximum Acceptable Water level is determined by the crest height of the secondary dike minus local set-up, run-up and settlement. TAW 2003 [19] The storage capacity of the wetland can be calculated by:

\[
S = A \cdot h_{MAW}
\]

In which:

- \( S \) = Storage Capacity [m³]
- \( A \) = Storage Surface area [m²]
- \( h_{MAW} \) = Maximum Acceptable Water depth in transitional area [m]
CHAPTER 3

Failure analysis for the St. Bernard Parish coastal defence zone

In this chapter, the failure of the St. Bernard Parish dike system will be analyzed. Special attention is given to the performance of the transitional storage area, as it is also known within the ComCoast concept of a coastal defence zone. In paragraph 3.1 an introduction is given on the St. Bernard coastal defence zone, in paragraph 3.2 the failure is analysed by means of field observations and in paragraph 3.2.4 a failure calculation will be done in order to gain inside in the behaviour of the coastal defence zone.

3.1 INTRODUCTION IN THE ST. BERNARD PARISH COASTAL DEFENCE ZONE

The St. Bernard Parish dike system consists of three different dike types:

I. The primary dike (red line in Figure 3.25)
II. The Mississippi river dike (blue line in Figure 3.25)
III. The secondary dike (yellow line in Figure 3.25)

Between the primary dike and the secondary dike lays the transitional wetland and behind the secondary dike are the residential areas of St. Bernard Parish.

Figure 3.25
St. Bernard Parish dike system

Figure 3.26
Cross profile of St. Bernard coastal defence zone
3.1.1 DIKES

The up to 5.3m (17.5ft) primary dike lies all along the Mississippi River Gulf Outlet (MRGO), the Gulf Intracoastal Waterway (IWW) and the Inner Harbour Navigation Channel (also called the Industrial Canal). It forms the first manmade line of defence of the coastal defence zone against a storm surge from Lake Borgne. Therefore, it is referred as the primary dike. In the primary dike along the MRGO, there are two floodgates, one at Bayou Dupree and one at Bayou Bienvenue. Those gates allow water to flow in or out of the area under normal tidal conditions and they can be closed in case of a storm surge. The USACE is responsible for the maintenance and operation of this primary dike. In the south of the Parish, the primary dike bends away from the MRGO-channel and runs straight east along the southern residential area. Here it has no buffer backup; the houses are directly behind the primary dike. However, at this section there are kilometres of natural wetlands in front of the dike. Which appears to protect the dikes against the full brunt of the storm surge.

The 3m (10ft) or lower secondary dike separates the wetland from the two residential parts of St. Bernard Parish. The secondary dike serves two purposes: (1) it acts as a hydraulic boundary for nearby pump stations, which discharge water into the wetland, and (2) it forms a temporary holding basin that protects the residential areas from flooding in the event of limited overtopping of the primary dikes. (See paragraph 3.2.4, for calculated overtopping values) The local Lake Borgne Levee\textsuperscript{1} District is responsible for the maintenance and operation of the secondary dike system.

The 7.6m (25ft) high, Mississippi river dike protects St. Bernard Parish from river flooding. Due to indirect role in the hurricane defence system, the Mississippi river dike falls outside the scope of this research.

The primary dikes of St. Bernard Parish are mainly constructed of relatively poor materials, sand and shell sand, and the only erosion resistance material they are equipped with, is grass. The secondary dikes are made of relatively strong material, clay and well covered with grass. The slopes at the primary dike are in the order of 1/6 and of the secondary dike are 1/3.

At the point where the dike is crossed by other infrastructure, like bridges and water in/outlets, there are transitions containing sheet pile walls, concrete I-walls and T-walls.

**T-walls** – A type of floodwall that looks like an inverted “T” in cross section, usually supported on deeply penetrated bearing piles, and usually with a sheet pile wall beneath it to cut off seepage.

**I-walls** – A type of floodwall consisting of sheet piles embedded within a levee, and projecting above the levee crest. In cross section view, the wall is “I” shaped.

**Concrete I-walls** Are more or less the same as the I-wall as described above, but the difference is that they are capped with a concrete beam or wall.

\textsuperscript{1} In the US a Dike is called a Levee
3.1.2 TRANSITIONAL STORAGE AREA

The area enclosed between the primary dike and the secondary dike is a natural swampy wetland, which can be regarded as a storage area in case of overtopping of the primary dike. This configuration fits perfectly in the ComCoast concept of a wide coastal defence zone with additional environmental functions. At present, there are no significant human activities in the wetland, and it is the livelihood of many birds, crocodiles and other aquatic life.

In the nineteenth century, the area used to be a cypress swamp, but as the timber industry grew in the area, men started harvesting the cypress trees. At this moment only very few cypress trees still live in the area and they are disappearing due to the sinking of the land and the increased salt intrusion (Figure 3.27). Freshwater and sediment diversions will be needed in order to bring back the cypress tree in the area.

At the time the primary dike was constructed the transitional area was planned to be industrially developed. This development appeared to be disappointing and nowadays the marsh is in consideration for wetland preservation.

In the report “Salinity Measurements and General Condition of Violet Marsh, Post Hurricane Katrina“, Lin, J. P. et al. 2006 [9] is concluded that salinity increased considerably in the area between 1993 and 2005. The researchers concluded that this increase may largely be contributed to the inundation caused by Katrina. New measurements in February of 2006 showed that salinities had dropped a little, pointing out that the salinities after Katrina were indeed higher than usual.

![Figure 3.27 St. Bernard land use](image)
3.1.3 DEWATERING SYSTEM OF THE RESIDENTIAL AREA

In St. Bernard Parish, there are eight pump stations responsible for the dewatering of the residential area. These are actually designed to pump rainwater over the dikes and into the wetland. The wetland itself is drained by gravity through Bayou Dupree and Bayou Bienvenue into Lake Borgne. Exception is pump station #8, which pumps the water over the southern hurricane protection dike directly into the wetlands in the south.

The total discharge capacity is 200 m$^3$/s, for a total drainage area 85km$^2$. All pumps are powered by diesel engines, which are mechanically connected to the pumps. Five stations (representing 80% of total capacity) have operating floors approximately 3.7m above the natural ground surface, which substantially reduced storm-induced damage. Stations #2, #3 and #5 were flooded to a depth of six to eight feet above the operating floor, which destroyed the diesel engines, vacuum pumps, and many accessories. The three flooded stations accounted for 90% of the total estimated damage of $10.7 million.

3.1.4 ASTRONOMICAL TIDE

Lake Borgne is influenced by a small diurnal tide. The tidal range is about 0.35m. The water level in the wetland area behind the MRGO largely moves with the same amplitude as Lake Borgne, as it is connected with the Lake and the MRGO-channel through the Bayous Bienvenue and Dupree.
3.2 OBSERVED PERFORMANCE OF THE DIKES DURING KATRINA

In the following paragraphs, the entire dike system will be divided in separate reaches by means of their physical characteristics. (Figure 3.29 on next page). Their pre Katrina status and their performance during Katrina will be discussed based on observations done before and after Katrina.

The sections are numbered and they start with STB 01 at the IHNC, at the beginning of the primary dike. (Figure 3.29) From there it follows the primary dike west and south along the MRGO and subsequently straight to the east where the primary dike ends at the Mississippi river (at STB26). Sections STB27-STB32 covers the Mississippi river dike. From the point where the counting began, the sections wind inland along the secondary dike (STB33-42). (Figure 3.29)

Not all reaches are treated separately; some sections are discussed as a whole because they are of much lower interest in this research. The most important sections are the ones that are in the primary dike along the MRGO and the secondary dike.

For each reach will be discussed what type of construction it was made of, the crest height and its performance during Katrina. In Figure 3.29, the different reaches of the dike system can be distinguished.

3.2.1 PRIMARY DIKE

Reaches STB01 – STB06:
This combination of capped I-walls, I-Walls and T-Wall reaches, lies 2508m (8228ft) along the IHNC. The average height of these reaches is 4m (13ft). At some places along this section major breaching occurred due to I-wall displacement. Those breaches caused major flooding in the Lower Ninth Ward.

Reach STB07:
This reach is defined by a 7840m (25722ft) long dike lies along GIWW, with a crest height varying between 4.1-5.0m (13.5-16.5ft). There were some signs of overtopping at this reach, but no significant damage.

Reach STB08:
This reach is the 310m (1016ft) long Capped I-wall at the Paris road, which crosses the GIWW. The elevation of this reach was 4.1 m (13.4ft) prior to Katrina. This section was overtopped during Katrina, but only suffered scour damaged.

Reach STB09:
This reach is defined by a 2213m (7260ft) long dike between Paris road and the Bayou Bienvenue control structure. It lies along the transition of the GIWW and the MRGO. The average height prior to Katrina was 5.5m (17.9ft). Some overtopping occurred, but no major damage.
Figure 3.29
St. Bernard dike reaches
Reach STB10-STB12 (Bayou Bienvenue control structure): 
This section contains the Bayou Bienvenue control structure and the surrounding floodwalls and transitions on either side. The total length is 548m (1797ft), and contains capped and non-capped I-walls, T-walls a dike section and the gate structure itself. The height differs from 4.6-15.6m (15ft-18.5ft). Complete failure occurred with an I-wall at an intersection with another section. Further heavy damage and scour occurred along the entire length.

Figure 3.30
Breaches in reaches STB11-12 near Bayou Bienvenue structure

Figure 3.31
Picture of failed Bienvenue I-wall
Reach STB13
This reach is defined by a 6052m (19858ft) linear stretch of dike. The section was overtopped and heavily damaged during Katrina. The average top elevation was 5.3m (17.5ft).

Reach STB14
This reach is defined by a 740m (2427ft) long section of uncapped I-wall embedded within the dike. Installed in 1992 as a part of USACE repairs. This section of wall was heavily damaged during Katrina. The averaged height of this section was 5.6m (18.5ft) and had a freestanding height of 1.1m (3.5ft).
Reach STB15
This reach consists of a 532m (1745ft) long section of dike. This section was overtopped and heavily damaged during Katrina. The averaged height of this section was 5.0m (16.4ft).

Reach STB16
This 780m (2560ft) uncapped I-wall reach, was identical to reach STB14 and was also overtopped and heavily damaged during Katrina.

Reach STB17 & STB18
This 282m (925ft) long section of uncapped I-wall was overtopped and heavily damaged. It’s height differed between 5.0-5.6m (16.5-18.5ft).

Reach STB19
This 1522m (4994ft) linear stretch of dike and ends at the Bayou Dupree control structure. The average weighted height of this section was 5.7m (18.7ft). This reach was overtopped and heavily damaged.

Reach STB20
This reach includes the Bayou Dupree control structure and the adjoining transition floodwalls. The total length is 142m (465ft) and contained a concrete sheet pile wall, a T-wall and the gate structure itself. Settlement left the concrete walls roughly at 4.6m (15.2ft). On the northwest side of the structure, the I-wall failed.

Figure 3.34
Damage at Bayou Dupree control structure
Reach STB21
This 7791m (25562ft) linear reach starts from the southeast point of the Bayou Dupree floodwall. It’s averaged height was 5.8m (19.1ft). This reach was heavily damaged during Katrina due to overtopping.

Reach STB22
This reach was defined by a 427m (1401ft) long uncapped I-wall section. It’s averaged height was 5.8m (19.0ft). The I-wall was placed with 1992 USACE repair works. There was scour damage in this area as a result from Katrina.

Reach STB23
This 823m (2700ft) long section contains the last part of the dike along the MRGO and where it turns away inland. The weighted average of the section is 5.9m (19.5ft). Sections of this reach did obtain damage.

Reach STB23a
This reach lies between the MGRO to the intersection point where the secondary dike meets the southern primary dike. It is 1740m (5709ft) long with an averaged height of 5.2m (17ft). It only suffered some minor damage due to Katrina.

Reaches STB23b – STB26
This section is the southern primary hurricane dike and lies from where the secondary dike intersects the primary dike. It has a total length of 14337m (47039ft) and has its height differs between 5.2m (17ft) in the east to 3.9m (12.8ft) at the Mississippi river dike. There are some floodwall sections near the pump station and the intersection point with the Mississippi river dike. Only at a few places along the entire stretch, some minor damage has occurred.

Mississippi River dike
Reaches STB27 – STB32
This 20947m (68725ft) long section is the Mississippi river dike. Its height is between the 5.4-6.4m (17.6-20.9ft). This whole section did not suffer any damage during Katrina.
3.2.2  SECONDARY INTERIOR DIKE

Reach STB33-STB35
This reach consists of a 6460m (21195ft) long line of uncapped sheet pile I-wall. The reach starts in the north at IHNC the intersection of the secondary dike with the primary dike. The averaged height of the I-wall was 4.1m (13.4ft). There was a 1372m (4500ft) long stretch of uncapped I-wall that was damaged during Katrina and which is in need of repair. At the Jean La Fitte pump station, there is a 4.3m (14.0ft) high concrete floodwall which did not suffer from Katrina.

Figure 3.35  Transition Reaches STB35-36 secondary dike

Reach STB36a-c
This section contains a dike with a total length of 11072m (36325ft) and goes from Paris road to the Violet channel. It had an average height of 2.7m (8.9ft) and was significantly damaged by overtopping during Katrina. The section near the pump station was constructed by floodwall.

Reach STB37-STB41
This is the 2190m (7186ft) long section along the north side of the Violet channel. It is constructed by a combination of dikes and floodwalls. The height differs between 2.1-2.7m (7.0-8.7ft). The reach was overtopped and damaged.

Reach STB42
This reach starts south of the Violet channel and continues until it reaches the intersection point with the southern primary dike. The total length of the dike is 16833m (55227ft) long. This section had an averaged height of 2.3m (7.7ft) and was overtopped and damaged during Katrina.
3.2.3 CONCLUSIONS OF THE OBSERVATIONS

From performance observations in the area, a few things are concluded:

- Most parts of the primary dike where up to the hurricane protection design level (mainly 5.3m). Although it must be said that at many places sheet pile walls were applied to compensate deficient crest heights.

- The primary dike, which runs along the MRGO, has the most damaged sections of the whole system. Most of it is completely destroyed, probably at an early stage of the storm. This implies that it failed to give any protection against the storm surge.

- It appeared that the MRGO sections of the primary dike were largely constructed of relatively poor material. Observations showed that large parts consisted of fine sand or even shell sand, making it highly vulnerable to erosion due to wave attack or overtopping. It may be assumed that this fact contributed to the destruction of these sections.

- Accelerated erosion of soils was observed at transitions between soil and concrete or sheet pile structures. Like at the Dupree and Bienvenue gate structures. Lack of capping has also caused scour behind and around the sheet pile walls. See paragraph 3.2.4, for more information on those overtopping values.

- Although also heavily overtopped, the secondary dike performed much better than the primary dike. It was constructed well from solid clay with a good erosion resistant grass cover. Some sections had endured scour but only few minor breaches had occurred.

- After the storm some parts of the secondary dike and the section of the southern primary dike at Mississippi river were deliberated in order to help the pump stations in dewatering the Parish.

- The southern stretch of the primary dike was largely intact after Katrina. The enormous stretch of wetland which lies in front of this dike gets most of the credits for this. Although the orientation with respect to the wind direction, of this section was way more favourable than the MRGO section, which had a perpendicular orientation with respect to the wind direction.

Figure 3.36 on the next page is a map produced by the LSU Hurricane Centre showing damage to the dikes and the flood depths in the area. It must be said that they made no distinction between natural breaches which developed during the storm and the after the storm, manmade breaches.
Figure 3.36
Flood depths and dike breaches St Bernard Parish
3.2.4 FAILURE CALCULATION KATRINA

During Katrina, overtopping of the secondary dike caused flooding in the majority of the residential areas of St. Bernard Parish. This can only be the case if the primary dike has admitted large amounts of water into the transitional polder and exceeded its storage capacity. As denoted in paragraph 3.2, observations made clear that Katrina heavily eroded the primary dike. The report “The Failure of the New Orleans Levee System during Hurricane Katrina” by Team Louisiana [7], suggest that this erosion is caused by a combination of the local short period waves (3-5s) and the in height reduced long period ocean waves (15s). The locally generated short period, waves caused a continuously pounding on the outer revetment, while in the meanwhile the long period waves caused overtopping and scour at the backside of the dike. They also denote that it is very likely that this process started very early and that the primary dike was very quickly eroded.

In this paragraph, the overtopping of the primary dike and the flooding in the area is investigated. Interesting is to see what would have happened if the primary dike did not erode and what would be the difference as the crest height was at its intended design level. Therefore, the overtopping and flooding values are calculated for three different cases considering different dike configurations, with corresponding primary dike crest heights:

- Design level = Configuration with crest heights at the design level (5.3m)
- Pre-Katrina = Configuration with crest heights as observed before Katrina
- Post-Katrina = Configuration with crest heights as observed after Katrina (after heavy erosion)

In order by the USACE Airborne LIDAR surveys were performed on the primary dike in the year 2000, and repeated after Hurricane Katrina in 2005. The results of these surveys can be found in the IPET-report [20]. The data from these surveys are transited into simplified length profiles, divided in dike sections with a constant averaged height. The resulting Pre-Katrina and Post-Katrina length profiles are plotted in Figure 3.37. The original design crest height of the primary dike was 5.3m as determined in the “Lake Pontchartrain LA and Vicinity”[20]. So 5.3m will be the crest height for the design level case (yellow line Figure 3.37).

Another interesting question is what the contribution is of a secondary dike in the coastal safety. For each case, calculations will also been done without the presence of a secondary dike. The flood levels in the residential area will be compared to the case with a secondary dike present.

---

**Figure 3.37**
Length profiles primary dike

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![MRGO - dike height](chart)

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>Distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>1.0</td>
<td>2000.0</td>
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<tr>
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<tr>
<td>5.0</td>
<td>10000.0</td>
</tr>
<tr>
<td>6.0</td>
<td>12000.0</td>
</tr>
</tbody>
</table>

**Legend:**
- Pre katrina
- Post Katrina
- Design Level
3.2.5 SIMULATION SETUP

The main goal of the calculation is to determine the flood depth inside the residential area of St. Bernard Parish. To do this a model has been set-up under the simplification that there will be no wind set-up or wave growth in the transitional area and there is no wave overtopping over the secondary dike. (In chapter 5 something will be said about the justification of this assumption) Under this simplification, the schematisation of a coastal defence zone (Figure 2.16) becomes in the St. Bernard situation as presented below in Figure 3.38. Overtopping and overflow are the vital processes with the hydraulic boundary conditions at the right side of the scheme. The hydraulic boundary conditions will be treated in the next paragraph.

Figure 3.38
Systematic overview of overtopping system

Table 3.5
Parameters in overtopping system

<table>
<thead>
<tr>
<th>Relevant strength parameters</th>
<th>Boundary conditions</th>
<th>Other parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest level primary dike ( h_{cp} )</td>
<td>Wave height ( H_s )</td>
<td>Water level polder ( h_{wp} )</td>
</tr>
<tr>
<td>Angle of the outer slope ( \alpha )</td>
<td>Wave period ( T_p )</td>
<td>Overtopping discharges ( q_{1st} )</td>
</tr>
<tr>
<td>Surface area of the wetland ( A_w )</td>
<td>Duration of the storm ( t_s )</td>
<td>Overflow discharge ( q_{2nd} )</td>
</tr>
<tr>
<td>Crest level secondary dike ( h_{cs} )</td>
<td>Storm water level ( h_{SWL} )</td>
<td>Water level residential area ( h_r )</td>
</tr>
<tr>
<td>Surface of residential area ( A_r )</td>
<td>( )</td>
<td>( )</td>
</tr>
</tbody>
</table>

The area is schematized as an overtopping system. It is assumed that overtopping of the primary dike only took place at the reach, which lies along the MRGO with, a length of 20000m. (Figure 3.39) This is not completely true but in paragraph 3.2 was observed that, i.e. the GIWW section had only minor overtopping. The wetland storage area comes down to of 130km\(^2\) and the secondary dike has a length of 28000m.
Calculation
The first step is to calculate the flood depth inside the wetland storage area. The primary dike is divided in several reaches, with the same averaged height and is attacked by a storm surge topped by waves, assumed homogenous over the whole length. For each section the overtopping discharge per meter of dike, $q$ (m$^3$/s/m) is determined. This will be done by using the formula for combined overtopping (Equation 2.9). When this overtopping discharge is multiplied with the length of the section and subsequently all sections are summated it will lead to overtopping value $Q$ (m$^3$/s) over the total length of the primary dike. The filling of the transitional wetland is schematised as the filling of a bowl. Dividing the cumulative volume, by the surface area of the wetland (130km$^2$), the water level inside the wetland is calculated. (Equation 2.10) When this level exceeds the height of the secondary dike, the water starts to flow over the secondary dike, flooding the residential area. Wave overtopping is neglected at the secondary dike, so the overtopping volume is calculated using the weir formula for overflow (Equation 2.8). Finally, the water level inside the residential area is calculated by dividing the total volume of water by the surface area of the residential area, taking the averaged height of the residential area into account (0.75m + MSL). The lowest parts of the residential area are at -1.2m + MSL.

For all three crest height configurations (design level, Pre-Katrina and Post-Katrina), flood depths within the residential area are calculated with and without the presence of a secondary dike. This will provide insight in the role of a secondary dike in a coastal defence zone.

Excel spreadsheets are used to execute the calculations.

3.2.6 HYDRAULIC CONDITIONS DURING KATRINA
The 5.3m (17.5ft) high primary dike along the MRGO, south of Bayou Bienvenue structure, was severely overtopped and breached by the high storm surge. For a decent failure analysis it is important to know what the hydraulic conditions were during the storm.

During Katrina, there was no hydraulic measurement equipment installed, that has given a usable result on the hydraulic conditions at the primary dike along the MRGO. Most of the equipment in the area could not handle the enormous deflections in the water elevation. The best way to approach these hydraulic conditions is to use computer models to translate the known offshore wave and water elevation data, translate it to the near shore locations, and calibrate the results with high watermarks and observations.

IPET [20] used respectively ADCIRC and STWAVE to calculate the surge elevations and the wave characteristics at the primary dike. The results they got were more or less validated by the report of by Team Louisiana [7]. The results of the research are presented on the next page.
**Storm surge elevation**

In the Figure 3.40 from the IPET-report, design water levels, maximum calculated water levels and the maximum measured water levels are given in feet. Important notice can be made that both measured and calculated levels are much higher than the design water level, differences are up to 2.7m (8.9ft). This suggests that large overtopping and maybe even overflow occurred during the storm.

During the peak of the storm, ADCIRC predicts surge heights a few decimetres lower than the watermark observations made after Katrina. ADCIRC is known to underestimate the water levels; partly because it neglects wave set-up. For this reason, some extra height was added the water levels produced by ADCIRC, leading to a maximum surge height of 5.3m +MSL during the peak of the storm.

In Figure 3.41 the ADCIRC still water elevations are presented during days of Katrina. The red line in the figure points out that the water was already raising days before the storm arrived.
**Storm wave heights**

In the Figure 3.40 from the IPET-report, the measured and the design wave heights and peak periods are presented. Maximum wave heights are calculated of 1.7m (5.7ft), with peak periods of 16.3s. Field observations confirm these values.

In Figure 3.43 the storm surge and wave heights are plotted in on figure using the hydrographs as presented in the IPET-report. This graph is used as the input for the failure calculation.

One should take care using the outcome of those hydraulic models as input for calculations when there is a lack of observation data for calibration of these models. For this reason, a sensitivity analysis is done to observe the difference in the flooding of the wetland, considering certain changes in the hydraulic boundaries.
3.2.7 RESULTS

In Figure 3.44 the simulation results are shown of the three different primary dike cases. Inside the wetland, maximum water levels are found of 0.54m and 1.58m for respectively the Design Level and Pre-Katrina cases. (Figure 3.44A&B) In both cases, the water level stays below the average 2.8m of the secondary dike. It is possible that some light local overtopping over the secondary dike will occur due to local wind and waves. However, it would not by far be as disastrous as the flooding, which actually took place during Katrina. These results are completely different with the outcome from the Post-Katrina configuration, which is the heavily eroded profile of the primary dike. Here the water level inside the wetland reaches a maximum of 3.75m massively overflowing the secondary dike leading to flood depths in the residential area of 3.41m. The rising of the water level inside the polder slows down as the water starts to flow over the secondary dike. Eventually it peaks and decreases as the surge height decreases too and a constant water level inside the whole Parish is reached (Figure 3.44C). The water level inside the residential area starts at 0.75m in this figure this is no actual water but the average height of the residential area, which is added to the flood depth.

In Figure 3.44D the water elevation inside the wetland area of all three cases and the height of the secondary dike are plotted in one figure.
Simulation without secondary dike

Now calculations are done with the same primary dike cases, but now without the secondary dike. The water from the wetland can now flow freely into the residential area. In this case, the wetland area and the residential area are considered as one and the same bowl. As the average soil elevation of the residential area is 0.75m + MSL, the average flood depth will not be very high in the case of moderate overtopping as in the Pre-Katrina case, sc. 0.48m (Table 3.6). For the Design level case the water will stay below the average residential elevation. But there are also parts in the residential area which have elevation below this 0.75m average. Those areas will suffer from more severe flooding. The lowest parts of the area are down to -1.2 + MSL. This lead to maximum flood levels of 2.43m and 1.63m in respectively the Pre-Katrina and the Design level cases. As there was no flooding for both cases inside the residential with a secondary dike this makes a big difference.

In the Post-Katrina case, flood depths increased with only 0.11m. This is because the wetland areas already filled up so quickly that the secondary dike was not much of a boundary too.

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<thead>
<tr>
<th>Table 3.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated flood depths residential-area</td>
</tr>
<tr>
<td>Flood depth in Residential Area With secondary dike</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Pre Katrina (m)</td>
</tr>
<tr>
<td>Design level (m)</td>
</tr>
<tr>
<td>Post Katrina (m)</td>
</tr>
</tbody>
</table>

*Negative as the flood level is below the average soil elevations

Overtopping discharge

In Figure 3.45 Left, the overtopping discharge over the primary dike \( q_{1st} \), is given in the Design Level case. The maximum overtopping discharge is 0.50 m³/s/m, which comes down to 500 l/s/m, which is very high. In paragraph 2.3.4 is stated that for a clay dike covered with good grass the maximum overtopping is 10 l/s/m. In Figure 3.45 Right, the overtopping discharge at the secondary dike is given in the Post-Katrina case. The maximum overtopping discharge here is even higher, 1.56m³/s/m, nevertheless the secondary dike performed way better then the primary dike.

There are two reasons for this difference in erosion, (1) the primary dike was exposed to much larger wave attack, causing erosion of the outer slope and (2) the primary dike was constructed of relatively poor construction material compared to the secondary dike. Obviously, the fact that it was constructed of poor material stimulated the erosion of the outer slope.
3.2.8 SENSITIVITY ANALYSIS FAILURE CALCULATION

Here the sensitivity of the hydraulic boundary conditions is analysed to give an insight in the effect on the flooding if actual hydraulic conditions are less favourable as the ones used in the previous calculations. Three cases are investigated.

- Increasing the surge height
- Increasing the wave height
- Shift the peak of the waves, so it coincides with the peak of surge

The case with the Design Level length profile is used for the sensitivity analyse.

**Increasing the surge height**

In Table 3.7, the results are presented of calculations with increasing the surge height with 10% and 20%. This comes down to an absolute increase of the peak of the surge of respectively 0.52m and 1.05m. This leads to an increase in maximum flood depth in the wetland of respectively 142% and 421% (0.76m and 2.26m). So an underestimate of half a meter can lead to an increase of more than a meter of flood depth in the wetland. This can definitely be the difference between flooding or no flooding, in the residential area. As stated in paragraph 3.2.6, the difference between the design surge level and the actual surge level during Katrina are between 1.0-2.7m and the difference between calculated surge level and observed surge level are between 0.1-1.0m. This indicates that the simulation with hydraulic conditions like the Katrina surge is highly sensitive for errors in surge height.

<table>
<thead>
<tr>
<th>Old (m)</th>
<th>New (m)</th>
<th>Increase (m)</th>
<th>Increase (%)</th>
<th>Old (m)</th>
<th>New (m)</th>
<th>Increase (m)</th>
<th>Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.32</td>
<td>5.84</td>
<td>0.52</td>
<td>10</td>
<td>0.54</td>
<td>1.30</td>
<td>0.76</td>
<td>142</td>
</tr>
<tr>
<td>5.32</td>
<td>6.37</td>
<td>1.05</td>
<td>20</td>
<td>0.54</td>
<td>2.80</td>
<td>2.26</td>
<td>421</td>
</tr>
</tbody>
</table>

**Increasing the wave height**

In Table 3.8, the results are presented of calculations with increasing the wave height with 10% and 20%. This comes down to an absolute increase of the peak of the wave height of respectively 0.18m and 0.36m. This leads to an increase in maximum flood depth of respectively 22.4% and 47.2% (0.17m and 0.34m). So this shows that an increase of 0.2m in wave height leads to an increase in flood depth of about 0.12m. This indicates that errors in wave height are of importance to calculate the flood depth, but it has far less influence than errors in storm surge height.

<table>
<thead>
<tr>
<th>Old (m)</th>
<th>New (m)</th>
<th>Increase (m)</th>
<th>Increase (%)</th>
<th>Old (m)</th>
<th>New (m)</th>
<th>Increase (m)</th>
<th>Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.83</td>
<td>2.01</td>
<td>0.18</td>
<td>10</td>
<td>0.54</td>
<td>0.66</td>
<td>0.12</td>
<td>22.4</td>
</tr>
<tr>
<td>1.83</td>
<td>2.19</td>
<td>0.36</td>
<td>20</td>
<td>0.54</td>
<td>0.78</td>
<td>0.25</td>
<td>47.2</td>
</tr>
</tbody>
</table>
Shift the peak of the waves, so it falls together with the peak of surge

In the original hydraulic conditions the wave heights peaks 2 hours later than the peak of the storm, when the surge has already decayed. So the waves will have less effect than if the two peak together. In order to determine the sensitivity of this time effect the wave peak is translated with 2 hours.

Table 3.9 shows that the peak translation lead to an increase of flood depth of 100% (0.53m), which is quite considerable, but it must been said that an error of 2 hours is very large, and highly unlikely.

<table>
<thead>
<tr>
<th>Increase (%)</th>
<th>Old (m)</th>
<th>New (m)</th>
<th>Difference (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>0.54</td>
<td>1.07</td>
<td>0.53</td>
</tr>
</tbody>
</table>

For the used wave period of 16.3s the breaker index, $\xi > 2$, which indicates that the waves are non-breaking and so the corresponding Equation 2.7, for non-breaking waves is used in the failure calculation. The wave period is not presented in this equation, which indicates that the peak period is of secondary importance with these wave characteristics.
3.2.9 CONCLUSIONS FAILURE CALCULATION

A few important conclusions can be drawn from the failure calculation:

- If the primary dike was not eroded the water storage would have been sufficient to avoid flooding of the residential area from that direction. Table 3.10 shows this as in both the Pre-Katrina and the Design level no flooding occurred in the residential area.

- Field observations of flood depths are also included in Table 3.10. The observations give an estimate 0.2m lower than the calculation of the Post-Katrina case, which considered a dike profile after the storm induced erosion at the primary dike. The field observations differ 2.17m with the Pre-Katrina case. This indicates that the erosion took place at a very early stage during the storm.

- The secondary dike can be highly effective, if the storage capacity of the wetland is not exceeded, which would not have been the case if the primary dike had not breached.

- Small errors in surge heights can have great effects on flood depths behind the dikes. This implies that one should take extreme care in setting the design water level and determine the crest height. And even then the designer should look at what happens if the surge exceeds the design level. How can you handle this so that it will not directly lead to catastrophic failure?

- In the case of Katrina’s hydraulic conditions, the surge height is dominant over the wave height. This will change as the difference between the surge height and the wave height is smaller. In the Katrina the surge was about 5.3m and the waves about 1.8m. This is due to the wave breaking on the existing shallow foreshore.

<table>
<thead>
<tr>
<th>Table 3.10 Flood depths</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Design level (m)</strong></td>
</tr>
<tr>
<td>Pre Katrina (m)</td>
</tr>
<tr>
<td>Post Katrina (m)</td>
</tr>
<tr>
<td>Observations (m)</td>
</tr>
<tr>
<td>Flood depth wetland</td>
</tr>
<tr>
<td>0.54</td>
</tr>
<tr>
<td>1.58</td>
</tr>
<tr>
<td>3.75</td>
</tr>
<tr>
<td>No observations</td>
</tr>
<tr>
<td>Flood depth residential area</td>
</tr>
<tr>
<td>No flooding</td>
</tr>
<tr>
<td>No flooding</td>
</tr>
<tr>
<td>3.41</td>
</tr>
<tr>
<td>3.21</td>
</tr>
</tbody>
</table>
3.3 GENERAL CONCLUSIONS REGARDING FAILURE ANALYSIS

From the failure analysis of the St. Bernard coastal defence zone a few important conclusions can be drawn:

- During Katrina the design elevation of the storm surge was exceeded which automatically resulted in failure of the hurricane defence system. Nevertheless it can be concluded from the failure analysis that the primary dike should not have failed as it did during Katrina if better construction materials were used. Observations showed completely destroyed sections of the primary dike along the MRGO, which can largely be contributed to the poor materials used during construction. During Katrina, the primary dike lost its complete function in retaining the hurricanes storm surge admitting lots of water into the transitional area. This primary failure led to the following complete and disastrous failure of the entire coastal defence zone.

- Calculations showed that the system as a coastal defence zone, with acceptance of severe wave overtopping, could have been highly effective if the dike system was in good shape, i.e. with the crest heights at design level and constructed well. The storage area would then have been sufficient to keep the residential area dry. The secondary dike would have been effective in this case.

- A sensitivity analysis showed weaknesses in a coastal defence system that allows wave overtopping. An exceedence in the design water level of a storm surge can lead to considerable increases in flood depths on the landside of the primary dike, which can easily lead to disastrous failure of the coastal defence zone. On the other hand if the surge elevation stays only a few decimetres beneath the design surge elevation, hardly any wave overtopping occurs.
CHAPTER 4 Design of the coastal defence zone

In this chapter a conceptual spatial design will be made for the St. Bernard Parish’s hurricane defence system. An important approach in this chapter will be to overlook the possibilities in using alternative coastal defence strategies. Instead of simply heightening the primary dike the ComCoast approach of a wide coastal defence zone will be proposed. Advantage could be gained from the existing wetlands in the area, both in landward as in seaward direction. After a brief introduction, paragraph 4.2 will treat the design conditions of the coastal defence zone, including the determination of the hydraulic design conditions. Paragraph 4.3 will give the principle ComCoast solutions including basic construction calculations mainly crest height calculations. Eventually in paragraph 4.4, a quick scan on environmental impacts will be studied in order to evaluate the alternatives.

4.1 INTRODUCTION

The St. Bernard Parish defence system is considered as a coastal defence zone and four defensive elements can be distinguished.

A. The secondary dike (water retaining)
B. The transitional area (water storage)
C. The primary dike (water retaining)
D. The shallow foreshore (energy dissipation)

The dimensions of those four components together determine the safety for the populated hinterland. In Figure 4.47 the different components are appointed.

Figure 4.47
Systematic overview of overtopping system with main elements

Hinterland

A  B  C  D
In chapter 3 was concluded that erosion and overtopping/overflow of primary dike, followed by overflow of the secondary dike caused the flooding in the St. Bernard’s residential areas. This means that either the storage capacity was too low or there was too much overtopping/breaching at the Primary, or both. In Table 4.11 those two main causes are given with their possible solutions, sorted by the coastal defensive elements as described in Figure 4.47.

<table>
<thead>
<tr>
<th>Causes</th>
<th>Solutions</th>
<th>Element (Figure 4.47)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lack of storage capacity</td>
<td>Increase of storage surface area</td>
<td>(B)</td>
</tr>
<tr>
<td></td>
<td>Heightening of the secondary dike</td>
<td>(A)</td>
</tr>
<tr>
<td>Too much overtopping</td>
<td>Heightening of the primary dike</td>
<td>(C)</td>
</tr>
<tr>
<td></td>
<td>Erosion prevention</td>
<td>(C)</td>
</tr>
<tr>
<td></td>
<td>Foreshore wave reduction</td>
<td>(D)</td>
</tr>
<tr>
<td></td>
<td>Foreshore surge reduction</td>
<td>(D)</td>
</tr>
</tbody>
</table>

The solutions as formulated in Table 4.11 are very well represented by the ComCoast alternatives: Managed realignment, Regulated tidal exchange, Overtopping resistant dike, Foreshore protection and Foreshore recharge, as described in paragraph 2.2.2.
4.2 DESIGN CONDITIONS AND ASSUMPTIONS

In this paragraph the most important design conditions and assumptions are treated. The two important assumptions, of an infinitely strength dike and the closure of the MRGO are treated in respectively paragraphs 4.2.1 & 4.2.2. Paragraph 4.2.3 will treat the most important design condition, namely the hydraulic design conditions.

4.2.1 STRENGTH OF THE DIKE

If the dikes inside a coastal defence zone are considered infinitely strong, then failure mechanisms piping, instability and erosion can be excluded from the fault tree as in Figure 4.48. Failure of a coastal defence zone is transformed into a single line of events, and depends mainly on the amount of overtopping at the primary dike. This is an easy approach to determine the safety of a coastal defence zone.

In this way the 1st phase of design, will consist a spatial configuration of the coastal defence zone, which will not fail due to lack of storage capacity. In most cases, the crest heights of either the primary dike or the secondary dike will be the most important variables. In a 2nd design phase the strength of the primary dike and the secondary dike are considered. Making sure, they will not fail by means of the other failure mechanisms. If this leads to dike configurations, which are not feasible, one can consider going back to the first design phase.

Figure 4.48
Simplified fault tree
4.2.2 CLOSURE OF THE MRGO/GIWW

One of the main causes of the flooding of New Orleans was the failure of the floodwalls along to the IHNC. This failure is highly subscribed to the funnel shape dike configuration in the west of Lake Borgne, where the GIWW and the MRGO intersect. In case of a storm surge the water is pilled up inside the funnel, onto the dikes and pushed into the GIWW, leading to high water elevations inside the IHNC, overtopping the floodwalls, which led to the catastrophic breaching. At this moment the hurricane protection system restoration plans are determined that (temporary) closure of the GIWW is an absolute must. Two main solutions are proposed for this closure. (See Figure 4.49):

A. One large flood control structure at the intersection of the MRGO and the GIWW, which is open under normal conditions, but can be closed at the time of a severe storm.
B. Two smaller flood control structures, one in the GIWW and one in the MRGO, with a dike in between connecting the two structures.

ARCADIS USA is doing the study for those flood control structures, at this moment it is not clear yet what will be the outcome of this research and which solution will be chosen. However, the fact that the GIWW will be closed, protecting St. Bernard’s and New Orleans Parishes against flooding from the IHNC, is quite sure. Therefore, for this research closure of the GIWW is held as an assumption. The floodwalls along the IHNC and the dike along the GIWW will become of secondary importance and falls behind the scope of this research. Of main concern is the primary dike along the MRGO dike, especially because closure of the GIWW will most likely lead to higher surge levels along the adjacent dikes.

Figure 4.49
MRGO closure alternatives

For the determination of crest heights in the next paragraphs, MRGO-closure alternative A is used (see Figure 4.49). If eventually alternative B will be chosen, this can be seen as an elongating of the overtopping length leading to a larger volume of overtopped water. In the meantime, one can see the “old funnel area”, which lies behind the new primary dike (green line in Figure 4.49) as new storage area. This will be more than a compensation of the increase in overtopped water. So as simplification only alternative A is considered in the calculation.
HYDRAULIC DESIGN CONDITIONS

The most important boundary condition for design of a coastal defence system are the hydraulic conditions which should be dealt with. Those hydraulic design conditions usually follow from a certain representative design storm set by decision makers. In the report: “The Lake Pontchartrain and Vicinity Hurricane Protection Project, 1965”, was decided that the hurricane protection system should be designed in such a way that it could stand a storm which occurred once in 200 or 300 years. In the same report, this storm was set on a fast moving hurricane 3. Though it has been revised and reformulated few times afterwards it is still a very vague approach as hurricane occurrences may change in time. Until now, a lot of debate is going on about on what level the design conditions should be set, it is a very complex engineering problem.

A few different approaches are discussed below, the Risk-based approach, the Maximum Hurricane and Probable Maximum hurricane. Eventually hydraulic conditions are chosen to fulfill some calculations for the design process.

The “Dutch” Risk-based approach

After the 1953 flooding, the Netherlands has adapted a risk-based approach. In this approach, the probability of flooding and the consequences are both taken into account. This is as risk is formulated by the probability of flooding multiplied by the consequence.

Risk = Probability * Consequence

An acceptable risk can been set and this has led to different flood area’s, dike rings, each with different safety standards depending on the economical an demographical value of this dike ring. This safety standard is expressed in an exceedance probability varying between 1/10 000 years and 1/250 years. From hereon the hydraulic conditions, which the dikes should deal with, are set, corresponding to these exceedance probabilities. In doing so coastal defence systems can be periodically checked if they still meet to the safety standards. In this way it should be prevented that “unexpected” catastrophic disasters can occur. In the Netherlands the coastal defence system is checked every five years looking both at the consequences and at failure probability (hydraulic conditions and conditions of the defences).

The risk based approach needs a very complex and time consuming analyze of hurricane induced hydraulic loads. For example, its storm surge characteristics depend on many factors like, bathometry, wind speed, translation speed, landfall location, storm size and shoreline orientation of the storm track at landfall. At the time of this research such an approach is not performed jet. Many storms will have to be simulated to obtain sufficient statistical data.
Potential Maximum Surge (PMS)
The potential maximum surge is the highest surge elevation for certain category storm, concerning all possible storm tracks, directions and approach speeds. The LSU and NOAA used the SLOSH model to calculate the potential maximum surge for the area for category 3, 4 and 5 storms. They run the model using a storm with a certain category and forward speed and varying its direction and its path. For each point the highest water level was plotted in a map. See Figure 4.50. Finally all highest elevations for all directions and paths were put together in one map, showing the Potential Maximum Surge elevation. (See Figure 4.51). Now we know the PMS for a certain category storm at all locations. The results of the PMS at the MRGO-dike are shown in Table 4.12.
From the SLOSH results can be concluded that a slow moving hurricane (5mph) leads to the highest PMS. The simulation results are with a high tide.

<table>
<thead>
<tr>
<th>Category</th>
<th>Category 3</th>
<th>Category 4</th>
<th>Category 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMS in feet</td>
<td>17</td>
<td>20</td>
<td>23</td>
</tr>
<tr>
<td>PMS in meters</td>
<td>5.1</td>
<td>6.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Table 4.12
Maxima (PMS)

Figure 4.50
Maximum surge for a certain hurricane paths

Figure 4.51
Maximum surge for all directions
PMH - Probable Maximum Hurricane

In LACPR-report, USACE 2006 [22], modelling has been done with the Probable Maximum Hurricane (PMH) as documented in NOAA’s Technical report NWS23 (1979). The PMH for the Louisiana coast describes a storm with a category 5 intensity with a central pressure of 890mb. This storm is used to simulate 10 different historical hurricane tracks with landfalls across the Louisiana coast with different approach angles. Surge-modelling is done with ADCIRC; Wave-modelling is done with STWAVE. The “worst-case” scenario results are given in Table 4.13.

<table>
<thead>
<tr>
<th>Surge Height $h_s$</th>
<th>Wave Height $H_s$</th>
<th>Peak Period $T_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.0m</td>
<td>4.3m</td>
<td>14s</td>
</tr>
</tbody>
</table>

In the same LACPR-report the central pressure return periods are calculated for a certain ‘zone B’ on the Gulf of Mexico. The return period of a PMH (890mb) is 200 years (Figure 4.52). But it must be stated that zone B is a coastal stretch of 650 km long. In general, one can say that a hurricane needs to be with a range of 50 kilometres in order to cause real danger. This will lead to a return period of PMH in the New Orleans area:

$$\text{Return Period} = \frac{650}{50} \cdot 200 = 2600 \text{ yrs}.$$ 

The probability that this Hurricane which makes landfall in zone B causes the “worst-case” depends also on exact location of landfall, path and forward speed. This indicates that the return period for the “worst-case” scenario results as in Table 4.13 is even larger than 2600.
Conclusion design hydraulic conditions

At this moment it is not clear what will be design hydraulic conditions for the New Orleans coastal defences. For this conceptual design study, design hydraulic conditions will be set following from the studies described above. It must been stressed that those hydraulic conditions are just an example so the conceptual design can be made. For a descent design of the coastal defences a much more thorough study is needed to determine appropriate design conditions.

For the remaining of this chapter the design hydraulic conditions will be set on a storm with:

- A maximum storm surge Water Level, $h_{swl}$ of 7.0m + MSL
  As calculated for the Potential Maximum Surge by the LSU/NOAA study.

- A Maximum significant wave height, $H_s$ of 4.3m, with a peak period, $T_p$ of 14s.
  As calculated in the LACPR study.

The forward speed of the storm is 5 Mph is used leading to a storm duration of 24hours.

The hydraulic peak conditions are given in Table 4.14 and used to translate the Katrina conditions as used in chapter 3 into design conditions (Figure 4.53).

<table>
<thead>
<tr>
<th>Storm surge height (m)</th>
<th>Wave height, $H_s$ (m)</th>
<th>Peak period, $T_p$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>4.3</td>
<td>14</td>
</tr>
</tbody>
</table>

Table 4.14
Used hydraulic conditions

Figure 4.53
Used hydraulic conditions

Storm surge and Wave heights
4.3 PRINCIPAL “COMCOAST SOLUTIONS”

In this paragraph, five principal solutions will be discussed, based on the five ComCoast solutions (see paragraph 2.2.2). In addition, a Zero+ alternative is considered which comes down to the preservation of the current coastal defence strategy, which will meet the new safety standards by simply heightening of the primary dike. Actually, the current defence strategy also corresponds to ComCoast concept of ‘Regulated tidal exchange’. This is as the Bayou Dupree and the Bayou Bienvenue control structures are opened during normal conditions and allow tidal action inside the transitional wetland area. There for the Zero+ and the Regulated tidal exchange alternatives will be treated as one. Leading to the five alternatives:

- Landward solution: Zero + / Regulated tidal exchange
- Landward solution: Managed realignment
- Landward solution: Overtopping resistant
- Seaward solution: Foreshore recharge
- Seaward solution: Foreshore protection

Each alternative will be treated in the paragraphs 4.3.1 to 4.3.5. They will be briefly explained and some first elaboration will be done on the required construction, which will be needed to withstand the hydraulic conditions as stated in paragraph 4.2.3.

The safety standard, which is that no overtopping occurs at the secondary dike, will be the same for every alternative. Therefore, in this evaluation, there will be no differences in safety between the solutions. However, there are big differences in what is needed to achieve the same level of safety. To gain insight in these required safety adjustments, for each alternative, quick calculations will be made about the required crest heights and construction materials. As it concerns only pilot calculation settlement and sea level rise are not included in the calculations. However, it must been said that especially settlement is of great importance as the dikes will have to be constructed on (former) marshlands. Higher crest heights will inevitably lead to more extensive settlement, demanding even higher construction heights.

Finally, the alternatives will be evaluated by means of a quick scan on environmental impacts. Herein the alternatives will be evaluated concerning a wide range of effects as: High water safety, coastal morphologies, soil and water quality, nature, landscape and cultural impacts, agriculture, costs and economical impacts. These effects will be more thoroughly discussed in paragraph 4.4.1.

In all cases strengthening and heightening of the primary and secondary dike is optional or even essential to cooperate with the safety demands.
4.3.1 ZERO+/ REGULATED TIDAL EXCHANGE ALTERNATIVE

In this case the hurricane protection is provided only by the primary dike. The dike should be heightened and strengthened until a level that it will become capable in resisting the design hydraulic conditions on its own. No or hardly any overtopping will be allowed, leading to a very high crest height design level. The secondary dike will remain just the hydraulic boundary for dewatering system of the residential area.

Construction

Here a quick calculation will be made in order to determine the required crest height for this alternative. The crest height will be calculated using the conventional Dutch approach using the run-up level (paragraph 2.3.2). This can be calculated with formula for run-up (Equation 2.3). For this quick calculation the combined reduction factor, $\gamma$ in the equation is set on 1.

The high peak period of 14s leads to high run-up levels of 6.0 m above the storm water level. The run-up added to the 7m high storm surge level leads to a crest height of 13m +MSL. With Van der Meer's formulas (Equation 2.5-2.8) the overtopping rate is calculated for this configuration and it comes down to 10.2 l/s/m. This points out that the revetment will still have to be well constructed of good clay and grass topping in order to prevent erosion and failure of the dike.

The total used material per meter of dike is 799.5 m$^3$/m. Applying a gentler slope may lead to reduction in crest height but on the other hand it will increase its base width and in the end required construction material will increase to. This will be shown in paragraph 4.3.6 were different outside slopes are considered for the Overtopping resistant alternative. For this case an outside slope of 1/6 is used, as is most common in the actual dike configuration.

In Table 4.15 the quick calculation results are presented.

<table>
<thead>
<tr>
<th>Surge Height (m)</th>
<th>Run-up height (m)</th>
<th>Crest height (m)</th>
<th>Overtopping rate (l/s/m)</th>
<th>Volume (m$^3$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>6.0</td>
<td>13.0</td>
<td>10.2</td>
<td>799.5</td>
</tr>
</tbody>
</table>

This 13m of crest height takes an enormous amount of material needed to achieve this. The heavy weight of the construction will cause large settlement problems due to the lack of solid foundation soils in the area, which is a general problem in the area.

The flood control structures will have to be adjusted or even be replaced in this alternative leading to higher construction works and costs.
4.3.2 LANDWARD SOLUTION: MANAGED REALIGNMENT

In this case, permanent gaps are created in the primary dike, so the transitional area comes openly but, with to restricted connections to the sea. Water can enter the area during high water levels but the waves will be reduced by the partially abandoned primary dike sea and wetland. This as the remainder of primary dike will be left alone and as it will decay and remain as a sort of breakwater. The secondary dike will become vital in the hurricane defence system, therefore it will have to be heavily upgraded. (Figure 4.55 & Figure 4.56)

**Construction**

It is assumed that the decayed primary dike and the wetland have no effect on the storm surge elevation but only on the wave height. Now if the wave reduction is set to be kept on 1 m, this will lead to a run-up reduction of 1.5m, corresponding to a crest height of 11.5m +MSL.

In Table 4.16 the quick calculation results are presented.

<table>
<thead>
<tr>
<th>Surge Height (m)</th>
<th>Run-up height (m)</th>
<th>Crest height (m)</th>
<th>Overtopping rate (l/s/m)</th>
<th>Volume (m³/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>4.5</td>
<td>11.5</td>
<td>10.2</td>
<td>629.6</td>
</tr>
</tbody>
</table>
4.3.3 LANDWARD SOLUTION: OVERTOPPING RESISTANT DIKE

In this solution, the primary dike is reinforced so it can handle large overtopping values. Overtopped water can be stored in the transitional area. The storage quantity depends on the height of the secondary dike, the storage area and drainage systems. Heightening of the secondary dike will only be needed in order to increase the storage capacity.

**Construction**

In this case, the crest and inner revetment is made overtopping resistant, so overtopping is allowed as much as the transitional area is able to store. There are two approaches in calculating the crest height; 1) is by restriction of the water level inside the transitional wetland, 2) is by limitation of the overtopping discharge. Both calculations will be done and evaluated which will be normative in this case.

![Figure 4.57](image1.png)

**Schematization of Overtopping resistant alternative**

![Figure 4.58](image2.png)

**Overtopping resistant dike**
**Storage area water level restrictive**

For this alternative, the maximum storage capacity is the starting principle for the overtopping calculation, with the maximum water level inside the transitional wetland bowl as the restrictive boundary condition. From hereon the overtopping rate and following the crest heights are calculated in an iterative way. The maximum water level inside the transitional wetland is set on 2m. The results are shown in Table 4.17.

<table>
<thead>
<tr>
<th>Surge Height (m)</th>
<th>Crest height (m)</th>
<th>$h_{\text{max}}$ wetland (m)</th>
<th>Overtopping rate (l/s/m)</th>
<th>Volume (m³/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>8.79</td>
<td>2.0</td>
<td>620.0</td>
<td>374.1</td>
</tr>
</tbody>
</table>

**Overtopping discharge restrictive**

For a more realistic value for an overtopping discharge at a reinforced revetment would 100 l/s/m. Now the restrictive boundary condition is the maximum allowable overtopping. The crest height for this situation is also calculated with a 1/6 slope. The results are shown in Table 4.18.

<table>
<thead>
<tr>
<th>Surge Height (m)</th>
<th>Crest height (m)</th>
<th>$h_{\text{max}}$ wetland (m)</th>
<th>Overtopping rate (l/s/m)</th>
<th>Volume (m³/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>10.7</td>
<td>0.31</td>
<td>100</td>
<td>537.3</td>
</tr>
</tbody>
</table>

**Conclusion**

For the 2m water depth in the storage area the maximum averaged overtopping discharge will be 0.62 m³/s/m, or 620 l/s/m, this is still exceptionally high. It appears that the overtopping discharge restrictive is the normative loading in this case. This implies that the transitional buffer has a more than a sufficient storage capacity, namely the calculated maximum water depth in the wetland is 0.31m, where 2.0m was allowed. This means that heightening of the secondary dike will not be needed.

A difficult problem is how to handle the massive settlement, which will occur after the construction of heavy dikes. It appears that the use of light reinforcement materials is evident.

Local wind effects inside the wetland will have to be further investigated. But for now it is plausible to consider 0.31m as a save value on the 2.8m high secondary dike.
4.3.4 SEAWARD SOLUTION: FORESHORE RECHARGE

In this case, the land directly in front of the primary dike is replenished so it becomes capable of dissipating energy from the surge and waves. Vegetation on the foreshore might be vital in order to prevent erosion and to dissipate the energy.

Construction

It will useful to investigate the effect of wave reduction on the crest height design level, due to foreland recharge. The same assumption can be done as for the managed realignment case, namely that the wave height will be reduced by a replenished foreshore with 1m. It will lead to the same crest height and construction volume of respectively 10.2m +MSL and 629.6m$^3$ (Table 4.16).

The difference now is that the primary dike will have to be reinforced, which comes done to less required construction material, as the primary dike is shorter and has a higher actual elevation.

How the configuration of the foreland should look like is difficult to determine. It depends on its construction height, length and vegetation. It will also have a positive or negative effect on the surge elevation. (See chapter 5)

To give an idea:
If 1km wide strip of foreland is elevated with 1m; $2.8 \times 10^7$ m$^3$ of landfill will be needed. Chapter 5 will show that this 1km of wetland will cause no reduction on the surge height
4.3.5 SEAWARD SOLUTION: FORESHORE PROTECTION

In this case a large breakwater is constructed in Lake Borgne at a certain distance from the primary dike. Waves will break on the breakwater and in doing so they lose their energy before they hit the primary dike, eventually leading to lower overtopping values.

Construction

It will useful to investigate the effect of wave reduction due to foreland protection on the crest height design level. The same assumption can be done as for the managed realignment case, namely that the wave height will be reduced by replenished foreshore with 1m. It will lead to the same crest height and construction volume of respectively 10.2m +MSL and 629.6 (Table 4.16).

In order to gain this reduction the breakwater be at least at the same height of the water elevation of the surge. This leads to a crest height of the breakwater of 7.0m +MSL.
4.3.6 COMPARISON OF CONSTRUCTIONS

In this paragraph, the previously calculated crest heights and crest volumes for the different alternatives will be compared and also additional constructions aspects will be discussed.

In order to compare the required construction material the crest volume per meter is multiplied by the dike’s length reduced by volume of the existing dike. The Managed realignment alternative is the only alternative in which the secondary dike is heightened. For all other alternatives, the primary dike is heightened. The existing dike configurations are given in Table 4.19.

<table>
<thead>
<tr>
<th></th>
<th>Crest height (m)</th>
<th>Crest volume (m^3/m)</th>
<th>Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary dike</td>
<td>5.3</td>
<td>142.3</td>
<td>20.000</td>
</tr>
<tr>
<td>Secondary dike</td>
<td>2.8</td>
<td>49.5</td>
<td>28.000</td>
</tr>
</tbody>
</table>

In Table 4.20 the construction characteristics are summoned including the crest heights, the total volume of required dike material and the additional construction elements are given.

<table>
<thead>
<tr>
<th></th>
<th>Crest Height (m)</th>
<th>Crest Volume per meter (m^3/m)</th>
<th>Total Volume Material (m^3)</th>
<th>Additional Construction Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero +/- RTE</td>
<td>13.0</td>
<td>799.5</td>
<td>13.1 x 10^6</td>
<td>Flood gate adjustments</td>
</tr>
<tr>
<td>Managed. R.</td>
<td>11.5*</td>
<td>629.6*</td>
<td>16.2 x 10^6 *</td>
<td>Dike along GIWW</td>
</tr>
<tr>
<td>Overtopping R.</td>
<td>10.7</td>
<td>537.3</td>
<td>7.9 x 10^6</td>
<td>Reinforced revetment</td>
</tr>
<tr>
<td>F. recharge</td>
<td>11.5</td>
<td>629.6</td>
<td>9.8 x 10^6</td>
<td>Landfill</td>
</tr>
<tr>
<td>F. Protection</td>
<td>11.5</td>
<td>629.6</td>
<td>9.8 x 10^6</td>
<td>Breakwater</td>
</tr>
</tbody>
</table>

*With alternative Managed realignment values are for the secondary dike

Crest heights
Calculated crest heights are very high and largely caused by the high surge and the long peak period of the waves leading to high run-up and overtopping values. Ways to reduce the run-up are to adjust the slope or the roughness of the outer revetment. To evaluate the influence of the slope, the crest heights are also calculated for the overtopping resistant alternative for various slopes.

A steeper slope results in a higher crest height on the other hand a less steep slope leads to a wider base. This leads to the effect that there is hardly any difference in total required dike construction material. (See Table 4.21).

Nevertheless, a less steep slope seems favourable as it will have less settlement due to a lower crest height and the revetments are better resistant against erosion.

<table>
<thead>
<tr>
<th></th>
<th>Crest height (m)</th>
<th>Crest volume per meter (m^3/m)</th>
<th>Total volume material (m^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope 1/4</td>
<td>12.1</td>
<td>548.7</td>
<td>8.1 x 10^6</td>
</tr>
<tr>
<td>Slope 1/6</td>
<td>10.7</td>
<td>537.3</td>
<td>7.9 x 10^6</td>
</tr>
<tr>
<td>Slope 1/8</td>
<td>9.6</td>
<td>535.7</td>
<td>7.9 x 10^6</td>
</tr>
</tbody>
</table>
In TAW 2002 [18] reduction factors are given for rough revetments. ($\gamma_E$) It varies between 1.0 and 0.55 for respectively short grass and a double layer of rip-rap stones. This means that a lot of reduction can be accomplished. For the zero plus alternative a reduction factor of 0.55 means a reduction of: $6\text{m} \times 0.45 = 2.7\text{m}$. This is quite substantial.

**Volumes dike construction material**

Large amounts of dike constructions material are needed for all alternatives. It is less favourable to heighten the secondary dike because it has a smaller existing crest volume and also the length of the secondary dike is longer than that of the primary dike. The availability of this construction material is not very good in this area.

**Flood gate adjustments**

If the crest height of the primary dike is going to be raised, the Bayou Bienvenue and Dupree floodgates will have to be adjusted or even be replaced. The higher the new crest elevation the more extensive and costive this adjustment will have to be. Therefore, this will be the less favourable in the Zero+ alternative case.

**Dike along GIWW**

If water is admitted into the transitional wetland area, the downgraded dike along the GIWW will have to be re-evaluated. As it can now be the case that this dike will be attacked from the backside and the closure of the MRGO will prevent the attack from the front side. In the overtopping resistant alternative, this will not be a problem as little water is admitted. But in the managed realignment alternative, this requires extra attention.

**Reinforced revetment**

A reinforced revetment should be applied in case of an overtopping resistant primary dike is. It would be favourable to use light reinforcement materials in order to prevent extra settlement due to the use of heavy materials. An advantage can be gained by using an erosion protective revetment, which increasing the outer slope’s roughness as the same time. This will lead to a decrease in required crest height.

**Landfill**

Large amounts of sediment will be needed to create a landfill capable of reducing the wave attack on the primary dike. An advantage is however that local relatively poor material can be used compared to the construction material of the dikes and will be much cheaper in m$^3$. An other advantage is that nature can benefit greatly from these regained wetlands. If the growth of vegetation and fresh water penetration is stimulated, new marshes and swamps can be created.

**Breakwater**

The construction of a breakwater will be costly, armoury stones will have to be used and it will be raised from the ground. It will have to be at almost the same crest height as the primary dike, in order to get the wave reduction as projected.
4.4 QUICK SCAN ON ENVIRONMENTAL IMPACTS

The principal ComCoast solutions are evaluated by means of certain criteria divided in seven categories of environmental impacts. This makes it possible to do an evaluation concerning also other values and deficits besides the construction aspects. The constructions aspects are not directly included in the effect study, but they play a vital role in other environmental impacts, i.e. Construction material and construction costs. The seven categories are summoned in the next paragraph below.

4.4.1 ENVIRONMENTAL IMPACTS

**High water safety**
This category is divided in by the following criteria.
- Performance during higher loads then design loads - What are the consequences if the hydraulic loads exceed the design loads. Will this lead to (partial) failure and how catastrophic will this failure be.
- Adaptability - Will it be possible to adjust the defence system in the future, if risk or safety levels change concerning, i.e. possible climate change.
- No-regret - Will it be possible to undo the engineering actions and in which level will it be able to add other defensive solutions in the future.

**Coastal morphology**
- To what extent will the adjustments in the system interfere with the natural sediment balance in the area? Either positive or negative.

**Soil and water quality**
- Salt ↔ freshwater - To what extent will the adjustments interfere with the salt and fresh water distribution. Will it increase the salt intrusion or will it do the other way around. We have seen already that the wetlands are suffering of the increased saltwater intrusion.
- Construction materials - How is the availability of the construction materials? What materials are used; rock, clay, concrete or sand? And are there good resources to obtain the material in the neighbourhood.

**Nature**
- What will be the impact on the ecological system? How will the wetland be preserved after the adjustments? The wetlands used to be a great resource for aquatic and birdlife and therefore they have a great value. Maybe the solution can have a contribution in bringing the cypress tree back in the area.

**Environmental quality**
- Influence on recreational values - How will the environmental adjustments influence the existing recreational value, and will it create new recreational opportunities in the area.
- Accessibility & Disturbance - In how far will the construction works disturb the daily life in the area. Will there be many road or navigation channels blocked? Both during the construction phase and the realization phase.

**Costs**
- Construction & Maintenance - What will be the cost for both the construction and the maintenance of the project?

**Economical impacts**
- Will the solution lead to any economical opportunities or will it be a drawback for the economical development of the area.
4.4.2 EVALUATION OF THE ALTERNATIVES

In this paragraph for each alternative scores are given for each of the environmental impacts, discussed in the previous paragraph. The scores are shown Table 4.22 on page 69. The scores are given by plusses and minuses. The in paragraph 4.3.6 calculated construction aspects are also added on Table 4.22.

**Zero +**
The Zero + alternative, which stands for the solely conventional heightening of the MRGO-dike, scores moderately with a few negative peaks following from the large amount of material needed. A very high crest height will be needed to deal with the high run-up levels. Also the lack of ability to handle larger loads then the design loads scores bad. When overtopped heavy erosion might occur, as the dike is not protected against overtopping. Also the settlement problems and the adjustments which have to be made for the floodgates pushes the costs higher.

**Overtopping resistant**
This alternative scores well on high-water safety as the revetment will have to be well protected against overtopping. The extra costs, which are needed to do this, could be well covered by the saving of material, as the crest height can be much lower than with the conventional heightening. On the other criteria it scores quite moderate, with a few negative aspects caused by the impact of the “hard” solution to make the dike overtopping resistant which will be needed for this alternative.

**Managed realignment**
Managed realignment has an overall bad score. It scores badly on high-water safety, as it will give up the use of the wetland as a buffer for the surge. The extend of the design storm and the uncertainties concerning it making it non-desirable to bring the surge closer to the residential area. Also will it enhance salt intrusion which will have a negative effect on the healthiness of the wetland, so no extra natural value will be gained, on contrary nature value is likely to be lost.

**Foreshore recharge**
This alternative scores well on most criteria. The only problem with this is that a tremendous amount of sediment and effort will be needed to achieve the amount of desired wave reducing ability. Also it will take a lot of time to vegetate the newly won land in order to make it effective to do this. Another positive effect is that it can prevent salt intrusion in the area, giving the wetlands upstream a better chance to rehabilitate. This alternative offers a great opportunity to win back some of the lost wetlands, therefore it score high on the nature effect.

**Foreshore protection**
This alternative score well on safety, but it might be difficult to apply. A high crest height will be needed to be effective on wave reduction as it with have to deal with high waves on top of a high surge level. It will therefore be costive and a disturbance for the environment. Also it’s expected to cause problems with settlement. Given the experience and problems with the rock revetment of the MRGO-channel. As the applied stones simply sink into the underlying soil layers of old decayed wetland. Despite the heavy breakwater, still a quite substantial primary dike is needed.
### Table 4.22
Evaluation of alternatives

<table>
<thead>
<tr>
<th>Overall rating</th>
<th>Economical impacts</th>
<th>Maintenance</th>
<th>Construction</th>
<th>Accessibility and disturbance</th>
<th>Recreational value</th>
<th>Environmental quality</th>
<th>Ecological impacts</th>
<th>No regret</th>
<th>Adaptability</th>
<th>Design Level/Exceedance</th>
<th>High-water safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>+</td>
<td>-</td>
<td>0</td>
<td>++</td>
<td>-</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>+</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>++</td>
<td>0</td>
<td>+</td>
<td>0</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>0</td>
<td>++</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>+</td>
<td>++</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>+</td>
</tr>
</tbody>
</table>

**Construction materials**
- Material:
  - Seed, topsoil, fill

**Additional Construction Elements**
- Breakwater:
  - 9.8 x 10 m
- Dike:
  - 162 x 10 m
- Flood gate:
  - 7.9 x 10 m
- Managed Nature:
  - 13.1 x 10 m

**Construction aspects**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Seaward Solutions</th>
<th>Landward Solutions</th>
<th>Sediment Solutions</th>
<th>Reclaimed Material (dry)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero +/−</td>
<td>Managed Nature</td>
<td>Overcoming Nourishment</td>
<td>Reclaimed Material</td>
<td>Volume of dredged material (dry)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Volume of dredged material (dry)</th>
<th>13.1 x 10 m</th>
</tr>
</thead>
</table>
4.4.3 CONCLUSION OF THE EVALUATION

From the overall score can be concluded that alternatives Foreshore protection and Managed realignment are no good solutions in the St. Bernard area. Main reason for the Managed realignment is that it loses a line of defence, without any saving of construction material or natural benefit. The bad score for the Foreshore protection follows mainly from the high additional costs of the breakwater, this were there is hardly any saving in construction material.

Alternatives Overtopping resistant and Foreshore recharge have the best overall score. The Reinforced alternative scores well as it has a large reduction in construction materials, also the erosion resistive aspect is a big plus, concerning the erosion at the primary dike during Katrina. The Foreshore recharge alternative scores best, as it is the only alternative with real natural benefits. A problem with both alternatives is that there exact performance under “Hurricane conditions” are not known, making them difficult to sell.

The Zero+ alternative scores average and can still be seen as one of the most sensible solution. Its performance is well known and there will be no big surprises during design and construction.

A favourable solution might be a combination of the alternatives; Dike heightening an overtopping resistant revetment and Foreshore recharge. It follows very much the idea of a multiple lines of defence. Further investigation is needed on those two alternatives. A heavy and costly revetment for the Overtopping resistant alternative makes it less favourable then it appears in this evaluation. And also the effect of the Foreshore recharge on wave reduction requires more investigation, to give an appropriate determination of the value of this alternative.
4.5 CONCLUSIONS REGARDING DESIGN OF THE COASTAL DEFENCE ZONE

From the design of the St. Bernard coastal defence zone a few important conclusions can be drawn:

- The extent of the wide spacious coastal terrain makes St. Bernard Parish suitable for the application of the ComCoast concept with multifunctional land use. However the failure calculation showed that difficulties might arise concerning the handling of the hydraulic loads. The performance is very sensitive when the water levels exceed design heights or not. This is the case in the ComCoast solutions, both landward and the seaward solution, which are based on the handling of waves. (energy dissipation due to wave breaking or wave overtopping admittance) This implies that the ComCoast solutions are suitable in environments where the effects of waves are dominant over the surge heights. This is not the case in the New Orleans area, waves are relatively low as they are already reduced in height by the influences land and shallow water along the long stretches of continental flat, which separates the city from the deep ocean water. This is the same continental flat that leads to high surge heights. In the surge dominated St. Bernard’s Parish area only a small exceedance of the surge design height will lead to extensive flooding.

- From the quick scan on environmental impacts, it can be concluded that the ComCoast alternatives for Overtopping resistant (including handling of the overtopped water) and Foreshore recharge are interesting options for the St. Bernard Parish coastal defence zone. Still the Zero+ option remains a reasonable option. A combination of those alternatives might turn out to be a very good solution. Further investigation on the ComCoast solutions is needed to determine which coastal defence strategy fits best in the coastal configuration of St. Bernard Parish.
CHAPTER 5

Wetlands and Storm Surge

In this chapter the influence of wetlands on storm surges is investigated. After a brief introduction on the relevant processes an analytical approach is made to solve this problem in paragraph 5.2. It appears that the problem is too complex to solve analytically, so in paragraph 5.3 a numerical model has been setup and used in order to gain insight in the problem.

5.1 INTRODUCTION

For the influence of wetlands on a storm surge, two main hydraulic processes can be distinguished.

I. Surge propagation
II. Wind set-up

Which process is dominant depends on the time scale of the whole process. Both processes will be explained beneath.

Surge propagation

This process is dominant when the wetlands are still dry and emerged above the water level. The surge arrives at the ocean and starts to travel inland, over the wetlands. (Figure 5.63). Since the average depth velocity is non-zero and in inland direction, one can speak of a dynamic situation. (Figure 5.63 right). In this case, the friction force caused by of the vegetation works in the opposite direction of the flow. The wetlands can delay the surge and if it retards the surge longer then the duration of the storm, it can reduce the eventual surge height at the dike.

\[ \nabla > 0 \text{m/s} \]

\[ \text{Fw = Wind force} \quad \text{Fg = Gravitational force} \quad \text{Ff = Friction force} \]

Figure 5.63

Emerged wetland
Right: Velocity profile
Wind set-up

In this process, the wetlands are already submerged, flooded, and the wind pushes the water against the dike. Eventually an equilibrium state will be reached between the wind drag force and the gravitational force, in which the gravitational force depends on the gradient of the water level. (Figure 5.64) One can speak of a quasi-static situation as the depth averaged velocity will become zero. However, the velocity is not zero for all depths. If one looks at the velocity profile one can see that at the surface the water will flow into the direction of the dike and at the bottom it will flow in the opposite direction. This as if the wind pushes the surfaces water to the dike, where it will disperse downwards and flows back as a return current at the bottom of the water column. (Figure 5.64 right)

In this quasi-static situation an increase of the bottom roughness, due to vegetation, will retard the return current, leading to higher set-up levels at the dike crest. (Figure 5.67). This means that for the wind set-up process, a higher friction has a negative effect on the eventual surge height at the dike.

Wind set-up → submerged situation → Negative effect on eventual surge height

Combination: Surge propagation ↔ Wind set-up

The difficulty arises when there is a combination of both processes, so when there already flooded wetland, where on one side the surge starts to flow over the wetland and on the other side the wind set-up is building up, but still far from the eventual equilibrium state. The velocity profile will converge from the dynamic profile to the quasi-static profile. (Figure 5.66)
Considering the processes as described above, low-lying wetlands can lead either to lower surge heights or to higher surge heights. The timescales related to both processes determine what will happen to the maximum surge elevation. On the one hand, if the propagating surge is held back long enough by the friction of the wetland, there may be not enough time to develop the wind set-up. In this use, the wetland has a positive effect. On the other hand, if the surge floods the wetlands in an early stage, wind set-up can be considerably. High bottom friction will now lead to an even higher wind set-up.

In order to say something about which and when one of these processes is dominant, surge elevations and especially, the involved time scales, are investigated by means of an analytical and a numerical approach, in respectively paragraph 5.2 and 5.3.

The situation of interest is the one with the combined processes. This is the case when the wetlands are already flooded at the time of the storm reaches landfall, due to early arriving of the front of the surge.

The Surge’s build-up inside the model’s boundaries, due to local waves, atmospheric pressure and rainfall is not taken into account. This implies that model is not usable for dike design purposes; the only purpose is to gain insight in the effects of wetlands on a surge.
5.2 ANALYTICAL ANALYSIS

The system with the wetlands is schematized as a horizontal strip of shallow land with length $L$ and a constant initial flood depth $z_0$. At the ocean side, the water elevation is excited by the surge on ocean waters. This disturbance will start to travel through the wetlands towards the dike. On the other side of the dike, the hurricane’s wind force pushes the water onto the dike. The force of the wind on the water will be compensated by the gradient of the water level. This disturbance travels from the dike towards the other boundary of the system, the oceans basin. (Figure 5.67)

The solution to the problem as discussed above will be approached with the equations of continuity and motion:

Equation of continuity:

$$\frac{\partial h}{\partial t} + \frac{\partial q}{\partial x} = 0$$

Equation of motion:

$$\frac{\partial h}{\partial x} + \frac{C_f q^2}{gD^2} = \frac{\tau_w}{\rho_d gD} - \frac{\partial q}{\partial t} \frac{1}{gD}$$

In the equation of motion, the first term on the left side is the gradient of the water surface; the second term is the resistance. On the right side of the equation, the first term is the external forcing of the wind and the second term is the inertia.

In the equation of motion, the drag or stress on the sea surface due to the wind, is measured as the horizontal force per unit area, $\tau_w$, and is given by:

$$\tau_w = C_D \rho_w W^2$$

Herein is $W$ the wind speed in m/s.

As inertia is neglected and the friction coefficient $C_f$ and the depth coefficient $D$ are considered constant; we can try to find a solution for $q$. We replace the wind forcing term in equation of motion (Equation 5.12) by the constant slope $i$, which will develop for $t \to \infty$.

$$\frac{\tau_w}{\rho_d gD} = i$$
Thus, we consider $i$ as being independent of the water depth, which is in fact not true. The set-up leads to higher water level elevations at the dike, implying that $i$ will decrease when the dike is approached.

With the simplifications made above, a solution for $q$ can been found, by substitution of the above in the equation of motion (Equation 5.12) we find after derivation:

$$q = \frac{c_i D}{\sqrt{C_f}} \sqrt{I_{\infty}} \left[ 1 - \frac{1}{i_{\infty}} \frac{\partial h}{\partial x} \right]^{1/2}$$

The root in the last part of the equation makes it hard to use. We can do another simplification by assuming that:

$$\frac{i}{i_{\infty}} \ll 1 \quad i = \frac{\partial h}{\partial x}$$

This leads to:

$$q \approx \frac{c_i D}{\sqrt{C_f}} \sqrt{I_{\infty}} \left[ 1 - \frac{1}{2i_{\infty}} \frac{\partial h}{\partial x} \right]$$

If we substitute the latter equation into the equation of continuity (Equation 5.11) we will find an advection-diffusion equation, which is commonly known in engineering:

$$\frac{\partial h}{\partial t} + c \frac{\partial h}{\partial x} - K \frac{\partial^2 h}{\partial x^2} = 0$$

Herein $c$ is the celerity of the disturbance and $K$ = the diffusion coefficient.

With,

$$c = \frac{3}{2} \frac{\sqrt{\rho_c C_D}}{\sqrt{\rho_w C_f}} \cdot W \quad \text{And,} \quad K = \frac{g D \sqrt{\rho_w / \rho_a}}{2 \sqrt{C_f C_D}} \cdot W$$

This indicates that the behaviour of the system depends mainly on those two parameters; $c$, the advection coefficient, which stands for the flow celerity of the disturbance and $K$, the diffusion coefficient, which stands for the widening/spread of the disturbance.

As noted earlier, time scales are very important in this problem, therefore it would be helpful to determine time scales for both the advection and the diffusion process.

$$\tau_c = \frac{L}{c}$$

$$\tau_K = \frac{L^2}{K}$$
With these two time scales the common Péclet number, \( Pe \) can be found. If the Péclet number is much smaller then 1, i.e. \( Pe \ll 1 \), diffusion is dominant over advection in the system. The Péclet number is given by:

\[
Pe = \frac{\tau_c}{\tau_e} = \frac{cL}{K}
\]

**Katrina case**

Now we’ll do some quick calculations with the analytic results for a Katrina like case. In the Katrina case \( W = 40\, \text{m/s}, L = 50000\, \text{m}, D_0 = 1\, \text{m}, C_f = 0.004, C_d = 0.0015 \) and \( \rho_w/\rho_a = 1000 \).

First relation between resistance and inertia is evaluated;

\[
\frac{R}{I} = \frac{C_f U_{\text{max}}}{D} \cdot T_0
\]

The maximum velocity estimated at \( U_{\text{max}} = 1.0\, \text{m/s} \)

\[
T_0 = 8333\, \text{s} = 2.3\, \text{hrs} \quad \text{and} \quad \frac{R}{I} = \frac{333}{1}
\]

Showing that in this case resistance is dominant over inertia and that it is appropriate to neglect it in this analytical approach.

If we use the Katrina case values to calculate the advection and diffusion coefficients we find

\[
c = 1.16\, \text{m/s} \quad \text{and} \quad K = 2374\, \text{m}^2/\text{s}
\]

\( c_1 = 4.5 \times 10^{-6} \) this leads to time scales

\[
\tau_c = 15.92 \times 10^3 \, \text{s} \quad \tau_K = 3.16 \times 10^6 \, \text{s}
\]

This indicates that a storm of 4.4 hours is needed to fully develop the adjective process, whereas the diffusive process takes a storm of 878 hours. Typically, hurricanes have a duration in the range of 6-36 hours. This show that during the a hurricane advection will fully take place where diffusion does not. The Péclet number can also be calculated;

\[
Pe = 36.7
\]

This indicates that advection is dominant over diffusion.

It appears that the problem is to complex to find an exact solution analytically. Therefore, in the next chapter, a numerical approach is used to find the exact solution for the problem.
5.3 NUMERICAL SOLUTION

In this paragraph, a numerical approach is used to investigate the effect of wetlands on a storm surge. The goal is to determine how variables like wetland length, initial water depth, the roughness of the wetland and the storm duration influence the eventual surge reduction at the dike. To do this a 1-dimensional computer model has been made using Matlab.

The wetland is simplified as a flat shallow foreshore with the oceans water level, $h_{\text{ocean}}$, at the left model boundary and the dike at the right model boundary, $h_{\text{dike}}$. (Figure 5.68). The wetland has a length, $L$, an initial flood depth, $z_0$, and a roughness, $C_f$. On the surface of the water the wind blows with wind speed, $W$. In the paragraph 5.1 is already stated that that are mainly two processes, which influence the system, namely; the surge propagating into the wetland from the ocean boundary and the wind set-up which is built up against the dike and which propagates from that boundary into the wetland.

Table 5.23, gives a summation of the used variables in the simulation and its dimensions.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>$W$</td>
<td>Wind speed</td>
<td>[m/s]</td>
</tr>
<tr>
<td>$z_0$</td>
<td>Water elevation above wetland at t=0</td>
<td>[m]</td>
</tr>
<tr>
<td>$L$</td>
<td>Length of the wetland</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_{\text{max}}$</td>
<td>Max water elevation in ocean</td>
<td>[m]</td>
</tr>
<tr>
<td>$T_{\text{end}}$</td>
<td>Duration of the simulation</td>
<td>[hrs]</td>
</tr>
<tr>
<td>$C_f$</td>
<td>Bed friction factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$C_d$</td>
<td>Drag friction factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$g$</td>
<td>Gravity acceleration</td>
<td>[m/s$^2$]</td>
</tr>
</tbody>
</table>

The model gives the solution of the equations of motion and continuity, as formulated in the previous paragraph. This means that inertia is also taken into account. The model setup and Matlab script can be found in, respectively Appendices A and B.
**Input**

Important input of the model are the forces on the system, there are two, namely the ocean water excitation and the wind speed. They can be chosen as constant or varying in time, like in Figure 5.69. Herein the ocean’s excitation is modelled as a sine, which comes as a peak on top of water elevation, which arrives early before the arrival of the storm. The wind speed is modelled as a parabolic profile. As the wind will increase as the hurricane eye approaches the area. In addition, the wind is also influenced by the angle of the winds with respect to the dike. The wind fields as calculated during Katrina show this effect in Appendix C.

**General results**

In Figure 5.70, an example of the output of the model is given. It shows the water elevations within a wetland after 10 hours. The length of the wetland is 125km. The ocean’s excitation is constant $2.0m + z_0$. Two different runs are plotted with different initial flood depths, for the red line $z_0 = 1.0m$ and for the blue line $z_0 = 0.5m$. One can clearly see the two processes of the wind set-up and the storm surge. In the storm surge the convective and diffusive effects can be recognized, as found in the analytical approach (paragraph 5.2). A propagation speed of $c \approx 1m/s$ was found, which means that the disturbance should travel 36km in 10(hrs), independent of the water depth. From the following analytical approach it is observed that the diffusion does relate to the water depth, which can be seen in the figure as the slope of the surge has decreased.
5.3.1 CRITICAL STORM DURATION

To give an insight on the effect of wetlands on the storm surge a critical storm duration, $\tau_{\text{critical}}$, will be introduced. This is the storm time, which is needed for the water elevation at the location of the dike to reach the same height as the incoming surge from the ocean.

$$\tau_{\text{critical}} = t \quad \text{if} \quad h_{\text{dike}}(\tau_{\text{critical}}) = h_{\text{ocean}}$$

When a storm period shorter than this critical storm duration is considered the wetland has caused reduction of the storm surge. If the critical storm duration is exceeded, the wind set-up will lead to an increase of the storm surge at the location of the dike. Actually, the situation with the wetlands is comparable with a coast without any shallow waters, thus no wind set-up, which can be seen as the complete opposite of the Louisiana coast.

The height at the dike is reached by a combination of the surge from the ocean, which has to pass the wetland, and the wind induced set-up. In Figure 5.71, for an example scenario, the height at the dike is plotted against time for both the set-up and surge effects. (resp. blue and green line) The two lines combined form the resulting red line, which determines the critical storm duration as the point where it exceeds the ocean’s surge level (stripped line).

Figure 5.71
Determination of critical storm duration, $\tau_{\text{critical}}$

How fast both processes develop, depends mainly on the length of the wetland, $L_w$, and the initial water depth, $z_0$, and of course the forcing on the system. In Figure 5.72 the results are given for case with a constant ocean surge elevation, $h_{\text{ocean}}$, of 2.0m and a parabolic wind speed field with a maximum of 40m/s, which are both comparable with circumstances during Katrina. For five different initial flood depths, the development of the critical storm duration is plotted against the length of a wetland.
The longer the wetland the more time is needed for the surge to reach the dike. This effect is shown in Figure 5.72. One can see that the first parts of the lines in the figure are more or less linear increasing, this is the part were the surge height at the dike is mainly determined by the arrival of the surge. As the wetland gets longer, the influence of the set-up gets more important, up-to the point where the surge has not enough time to reach the dike and the 2m of increased height is only caused by the wind set-up. Here the critical surge heights get constant for these long wetland reaches.

The initial water depth of the wetland determines how fast the disturbances travel through the wetland. If the initial depth decreases, the critical storm duration increases significantly. Especially the development of the wind set-up largely depends on the water depth. When the quasi-static end situation, \( t \rightarrow \infty \) is considered, Equation 2.2 points out that a smaller water depth eventually leads to a higher wind set-up. But on the other hand the simulation points out that it takes longer to reach this quasi-static equilibrium. Figure 5.72 shows that if only wind set-up is considered, which is the case for large values of \( L \), the critical storm duration differs from 10hrs to 23hrs (accounting for initial flood depths of respectively 1.0m and 0.1m) which is quite a substantial difference.

The only parameters which were kept constant in above simulation, are the friction coefficients \( C_f \) and \( C_d \), their influences on the system are treated briefly in the next paragraph.
5.3.2 SENSITIVITY OF THE FRICTION FACTORS

**Bottom friction factor Cf**

The roughness of the wetland is an important parameter in the process. To evaluate its sensitivity three different plots are made in Figure 5.73. It shows that an increase in wetland roughness creates a significant decrease in surge height at the dike. It also shows that the peak of the surge is delayed in case of higher roughness.

![Figure 5.73](image-url)  
Influence of roughness Cf on a storm surge

**Wind drag factor Cd**

The other friction parameter in the system is Cd. Figure 5.74 shows the development of the water elevation at the position of the dike for different values ofCd. It shows that it only increases the amount of set-up, which is mainly the first parts of the curves. The surge, which is the second peaked part of the curves, is hardly affected by variations of Cd.

![Figure 5.74](image-url)  
Influence of wind drag force, Cd, on a storm surge
5.4 CONCLUSIONS REGARDING WETLANDS AND STORM SURGE

From the study about the effect of wetlands on the storm surge a few conclusions can be drawn.

**General conclusions:**

- Two processes determine the water elevation at the location of the dike, namely; the propagating surge (reduction) over the wetlands and the wind set-up (increase) against the dike. The timescale related with the development of each process determines if there is a reduction or not.

- High bottom roughness can both have a negative and a positive effect on surge height. In a dynamic (oncoming surge) it reduces the flow leading to higher surge level. In a quasi-static situation, the return current is obstructed, leading to a higher storm surge. It depends on the phase of process if it has a positive or a negative effect on the eventual surge height.

- From the analytical approach it follows that the system behaves conform an advection-diffusion relation, with respectively parameters $c$ and $K$ being the advection and the diffusion coefficient. Were $c$ depends on the friction-coefficients and the wind speed. $K$ also depends on the water depth.

- The eventual effect of a wetland on a storm surge mainly depends on the length of the wetland, the initial water depth, the storm duration and the roughness of the wetland

**Conclusions from the model**

- The critical storm duration, $\tau_{\text{crit}}$, is introduced and it accounts for the storm time which is needed for the water level to rise to the same water level as the surge level of the ocean.

- Long stretches of wetland, 25km or longer, are required to have any chance in reducing eventual surge heights and even then its effectiveness depends on the initial flood depths, vegetation etc.

- High initial water depths are very unfavourable for the eventual surge heights. This as the wind set-up can develop quickly as it has a wide supply.

**Intermezzo: Channels**

A debate is going on about the existing navigation or oil and exploration channels inside the wetlands. Existent channels can act as a guidance for the set-up’s return current, lowering the set-up level at the crest of the dike. But on the other hand channels can also be a guidance to the rising water of the surge at the time the wetlands are still above the water level in the emerged case.

As described earlier in this chapter, the initial flood depth has a primary influence on the critical storm duration. Existing channels inside the wetlands will lead to higher initial water levels as the early rise of the ocean water, even before the real peak of the surge arrives, flows easily inland in the wetlands guided through the channels. This appears to be the most important aspect of the existence of the channels. In this case the wetland looses its effectiveness against storm surges.
This chapter presents the conclusions and recommendations that resulted from this work. The three objectives of this research, as described in chapter 1, are: 1) Analyse the performance of the St. Bernard Parish’s coastal defence zone, 2) Develop spatial integrated solutions for the St. Bernard Parish coastal defence zone, and 3) Investigate the effects of wetlands on a storm surge. The conclusions and recommendations on these research objectives are provided in, respectively Section 6.1 and Section 6.2.

6.1 CONCLUSIONS

This study provides no final designs or exact solutions to the problems in St. Bernard Parish. However, it has provided a lot of deeper insights in the problems and processes concerning storm surge defence systems:

1. How the dike system protected the flooding of St. Bernard Parish and how it failed in doing so.
2. How a ComCoast solution could be implemented in a coastal area and what the possibilities are for such a solution in St. Bernard Parish area.
3. How a storm surge behaves when it passes wetlands and what the processes involved are.

Conclusions concerning the failure analysis

- Observations after the storm showed the dike sections that failed during the storm. The most important conclusions that can be drawn from these observations are:
  a. The primary dike along the MRGO was heavily breached and eroded. The erosion spots made clear that the dike was partly constructed of relatively poor, non-cohesive construction materials.
  b. It appears that before the storm most dike-sections were up to design level. However, it must be said that often additional sheet piling was used to achieve the design height. Observations showed accelerated erosion around transitions between soil and sheet piles of concrete.
  c. The secondary dike showed signs of overtopping, but showed no severe erosion or breaching. It appears that this dike was constructed of much better materials than the primary dike.
The executed calculation of failure confirmed the failure of the dike system. The most important conclusions that can be drawn from this calculation are:

a. If the primary dike was properly constructed (construction materials and crest heights), the catastrophic failure probably could have been prevented. The secondary dike and the storage area would have been highly effective as a buffer.

b. Compared to the wave heights, the relatively high storm surge elevation makes the system with overtopping-admittance highly sensitive for errors in surge height estimates.

Conclusion concerning the design of a coastal defence zone

From the quick scan on environmental impacts, it followed that ComCoast alternatives Foreshore protection and Managed realignment are no suitable solutions in the St. Bernard area. The conventional Zero+ alternatives and the ComCoast alternatives Foreshore recharge and Overtopping resistant are possible coastal defence strategies. A combination of those alternatives might be a very good solution. Further investigation and elaboration on the ComCoast solutions is needed to determine which coastal defence strategy fits best in the coastal configuration of St. Bernard Parish.

The extent of the wide spacious coastal terrain makes St. Bernard Parish suitable for the application of the ComCoast concept with multifunctional land use. However, the failure calculation showed that difficulties might arise concerning the handling of the hydraulic loads. This is as the ComCoast solutions, both the landward as the seaward solutions, are based on energy dissipation of waves. This implies that the ComCoast solutions are suitable for environments where the effect of waves is dominant over the surge height, which is not the case in the New Orleans area.

Conclusions concerning wetlands and storm surge

The analytical and numerical approach has increased the understanding of the hydraulic behaviour of a storm surge over wetlands. It showed that it will lead to an increase of the eventual surge height in case of the dynamic situation with a surge propagating over the wetland and to a decrease in case of the static situation of wind set-up. Time scales were derived, which determine the behaviour of the system.

A critical storm duration was formulated in order to gain usable results from the numerical Matlab model. Results are given for Katrina like conditions, conclusions which can be drawn from these results are:

a. Long stretches of wetland, 25km or longer, are required to have a significant reducing effect on a hurricane’s surge height. Even then its effectiveness depends on the initial flood depths, vegetation etc.

b. High initial water depths are very unfavourable for the eventual surge heights.
6.2 RECOMMENDATIONS

In the field of this research, the following recommendations can be made:

• In order to take full advantage of the ComCoast concept, it is advisable to select case-study areas with more suitable hydraulic conditions. A more ideal ComCoast area should be an area in which:
  a. The normative hydraulic conditions are much closer to the daily wave climate; this means that more stable environmental conditions can be created.
  b. The hydraulic conditions are wave dominated instead of surge dominated as in the St. Bernard parish area.
  c. Salt intrusion can be seen as a benefit instead of an environmental problem as in the coastal Louisiana area.

• It is recommended to study the possibility of an approach, which is more based on damage control then on damage prevention. This is as very high unfeasible crest heights followed from the design study. Which comes with this is:
  a. Investigate the potential time-delaying effect and its benefits of a multiple lines of defence strategy.
  b. Develop innovative housing and infrastructure, which can handle short periods of inundation, caused by overtopping.
  c. Implement a proper dewatering system, in order to drain the residential area after inundation.
  d. Additionally, the type of failure and flooding should be concerned in decision-making. There is a big difference in damage done by instant breaching of a floodwall, or by gradual overtopping of a dike. Flow speeds inside the flood prone area differ extremely for both cases, whether the eventual flood depths might be the same.

• It is recommended to perform further investigation on the effects of wetlands on a storm surge.
  a. When does an increase of bottom roughness lead to higher surge values? In order to do this one should determine how the velocity profile develops in time. Or in other words: how does the dynamic situation converge to the quasi-static situation in time?
  b. Expand the model to a two dimensional model (x,y) and a three dimensional model (x,y,z).
  c. Expand the model and include other hurricane effects like i.e. wave set-up, atmospheric pressure and rainfall.

• A very extensive study is needed to determine the design hydraulic conditions for the whole New Orleans area. The risk-based “Dutch” approach is advisable, instead of using one “standard project hurricane” for the whole area.
References


Appendix A: Model setup

Figure A.1
Calculation grid

Model formulas:

\[
\frac{q^{n+1} - q^n}{\Delta t} + gD \frac{\partial h^n}{\partial x} + \frac{C_f}{D^2} q^{n+1} \big| q^n \big| = \tau_w
\]

\[
\left( \frac{1}{dt} + C_f \left| \frac{q^n}{D^2} \right| \right) \cdot q^{n+1} = \tau_w - gD \frac{dh}{dx} + \frac{q^n}{dt}
\]

\[
q^{n+1} = \frac{\tau_w - gD \frac{dh}{dx} + \frac{q^n}{dt}}{\frac{1}{dt} + C_f \left| \frac{q^n}{D^2} \right|}
\]

\[
h_i^{n+1} = h_i^n + \frac{(q_i^{n+1} + q_{i+1}^{n+1})}{\Delta x} \Delta t
\]
% gegevens
L=20000;
zo=1.0;
%W=50;
Cd=.0015;
Cf=.004;
g=9.81;
h0max=3;
Wmax=40;
Tend=4;%Storm duur in uren

% rekenrooster
nx=200;
dx=L/nx;
dt=0.1*dx;
neind=Tend*3600/dt;

% initialiseer debiet en waterstand
x=[0:nx]*dx;
h=[1:nx+1]*0;
q=[1:nx]*0;

% uitvoer
hdijk=[1:neind]*0;
hzee=[1:neind]*0;
hmax=hdijk;
W=[1:neind]*0;

for itijd=1:neind
    t=itijd*dt;
    % windschuifsprenging
    %W=15+5*sin(2*pi*t/(3600*5.5));
    W=Wmax*(sin((1/6*pi)+.5*pi*t/(neind*dt)));
    tau=Cd*W^2/1000;
    dh=h*0;
    dq=q*0;
for i=1:nx
    i1=i;
    i2=i+1;
    dhdx=(h(i2)-h(i1))/dx;
    D=z0+(h(i1)+h(i2))/2;
    alfa=tau-g*D*dhdx+q(i)/dt;
    beta=1/dt+Cf*abs(q(i))/D^2;
    q(i)=alfa/beta;
    dh(i1)=dh(i1)-dt*q(i);
    dh(i2)=dh(i2)+dt*q(i);
end
h=h+dh/dx;

% uitvoer
hdijk(itijd)=h(nx+1);
hzee(itijd)=h(1)+z0;
W(itijd)=W;
end
k=[neind-1000:neind];
%ampl=(max(hdijk(k))-min(hdijk(k)))/(max(hmax(k))-min(hmax(k)));
%h(1)=(h0max-z0)*((-cos(t/(dt*neind)*pi)+1)/2);
%h(1)=0;
% evt linker rvw
hold off;
plot(x,h+z0);
axis([min(x) max(x) 0 7])
%title([W= ' ,num2str(W), ' m/s'])
title([T= ' ,num2str(itijd*dt/3600), ' uur'])
drawnow
if(itijd==1)
pause
end
Appendix C: Wind fields during Katrina

Figure C.1
Wind field 09:00 UTC

Figure 25. Wind snapshot for 0900 UTC on August 29, just prior to landfall
Figure C.2
Wind field 12:00 UTC

Figure 26: Wind snapshot for 1200 UTC on August 26, just after landfall near Buras, LA.
Figure C.3
Wind field 15:00 UTC

Figure 27. Wind snapshot for 1500 UTC on August 29, near landfall at MS/LA border
\[ \bar{v} > 0 \text{ m/s} \]

Dynamic profile

\[ \bar{v} > 0 \text{ m/s} \]

Converging to static profile

\[ \bar{v} = 0 \text{ m/s} \]

Quasi-Static