Master Project CT4061

Port of Altamira, Mexico

April 2006
General notice to the reader:

In the academic programme for Hydraulic Engineering we have in the 4th year (i.e. in the first year of the Master Programme) the requirement that students should do in a group of four to six persons a so-called "groupwork". It is also called "Master Project". During this groupwork they should make a full design of something. The work should be integral, starting with terms of reference, and ending with the real design. This can be a structure, but also it can be a harbour lay-out, a policy plan design, etc. The total time available for the project is in the order of two months and will provide 10 European Credits. It has to be practical and applied.

It is certainly not an M.Sc. thesis assignment (the thesis work is individual, 6 months and more focussed on research or advanced design work on details). But it is also not an apprenticeship, internship or traineeship where the student has to work together with a group of experienced people. For this groupwork they have to solve the problem on their own (of course with guidance).

This report is the result of such a Master Project. This report has been assessed by staff of TU Delft. It has been provided with a passing mark (i.e. a mark between 6 and 10 on a scale of 10), and consequently considered sufficient for publication.

However, this work has not been fully corrected by TU Delft staff and therefore should be considered as a product made in the framework of education, and not as a consultancy report made by TU Delft.

The opinions presented in this report are neither the opinions of TU Delft, neither of the other sponsoring organisations.

Department of Hydraulic Engineering
Delft University of Technology
Master Project CT4061

Port of Altamira, Mexico

24th of April - 18th of June 2006

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Preface

Four Dutch students together form group 4B for their MSc-project. This project is part of their Master studies at the hydraulic division of the Civil Engineering and Geosciences faculty at Delft University of Technology. The challenge for this group will be to work on a coastal problem for eight weeks and at the end to come up with some possible solutions.

Group 4B came together almost a year before the actual start of the project. Besides they would be in the same stage of their studies, they also had to have the same ambitions. One of these ambitions was to get in contact and to work with different companies. The decision trying to find a challenging project abroad in cooperation with a company was quickly made. Sooner than expected there was an offer from Dutch dredging company Boskalis to go working on a port project in Altamira, Mexico. Host on site would become the Mexican dredging firm Dragamex (partly owned by Boskalis). Because the project seemed to be very interesting and it is located in a country and region that very much suits the wishes of the team an agreement was made.

Well arrived in Mexico a short week before the actual start of the project the team made a tour across the Altamira port site and they got in good contact with the nearby University of Tampico (UAT). The university offers the supply of necessary information and office space. Also the real (coastal) problems of the port became clear and the main goal for the group is to work out an erosion problem along the shore south of the breakwaters.

The multidisciplinary Project Team is supported by Delft University of Technology in the various technical disciplines involved in the project:

Prof ir H.J Verhagen  
Professor Bed, Bank and Shoreline Protection  
Section Hydraulic engineering  
Faculty of Civil Engineering and Geosciences  
Delft University of Technology

Prof.dr.ir. M.J.F. Stive  
Professor Hydraulic and Coastal Engineering  
Section Hydraulic engineering  
Faculty of Civil Engineering and Geosciences  
Delft University of Technology

There was also support from the University of Tampico involved in this project.

Dr. Sergio Jiménez  
Professor Oceanography and Ports and Waterways  
Faculty of Engineering “ Arturo Narro Siller”  
Institution of investigation in engineering  
University of Tampico, Tamaulipas

The navy of Tampico also helped with gathering information about the study area.

This report is written in English because of the international character of the problem. Unfortunately for the people in Mexico Spanish goes way beyond the capability of the group. It is a real honour to every member that the group gets the opportunity to do such a project abroad under very nice circumstances and we hope that people will enjoy reading this report, even when not involved.

Maarten van Bemmel
Kay Croonen
Dirk Froeling
Gerbrand Marbus
Sponsors

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# Table of Contents

## PREFACE

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## TABLE OF CONTENTS

## SUMMARY

### 1 INTRODUCTION

### 2 SITUATION GLOBAL

#### 2.1 INTRODUCTION
#### 2.2 TECHNICAL INFORMATION
#### 2.3 FUTURE OF THE PORT

### 3 PROBLEM DESCRIPTION

#### 3.1 PROBLEM ANALYSES
#### 3.2 DEFINITION OF THE PROBLEM
#### 3.3 GOAL

### 4 DETAILED SITUATION

#### 4.1 GENERAL
#### 4.2 HURRICANES
#### 4.3 WIND REGIME
#### 4.4 SEISMOLOGY
#### 4.5 TOPOGRAPHY
#### 4.6 OCEANOGRAPHY
##### 4.6.1 WAVES
##### 4.6.2 TIDES
##### 4.6.3 CURRENTS
##### 4.6.4 BATHYMETRY
#### 4.7 ON SHORE AND FORE SHORE SITUATION
##### 4.7.1 LITTORAL BARRIER
##### 4.7.2 DUNES
##### 4.7.3 BEACH PROFILE
##### 4.7.4 GRAIN METRIC
#### 4.8 HYDROLOGY OF THE WATER BODIES
#### 4.9 INUNDATIONS
5  SEDIMENT TRANSPORT PROCESSES  

5.1 WAVES CONDITION AT BREAKER DEPTH  
5.2 SEDIMENT TRANSPORT  
5.2.1 SURF ZONE  
5.2.2 LONGSHORE  
5.2.3 CROSS SHORE  
5.3 WAVE SETUP AND WAVE SET DOWN  
5.4 WIND SETUP  

6  APPROACH OF REQUIREMENTS ALTERNATIVES  

6.1 INTRODUCTION  
6.2 WORST CASE SCENARIO  
6.2.1 STORMS  
6.2.2 WAVE FORCES ON COAST  
6.3 FINAL REQUIREMENTS FROM OUR SIDE  
6.4 COASTLINE PROTECTED  

7  PROGRAM OF CONDITIONS  

7.1 ESSENTIAL PRE-CONDITIONS  
7.2 CONDITIONS  
7.3 STAKEHOLDERS  

8  ALTERNATIVES  

8.1 NON-INTERFERENCE  
8.2 SOFT SOLUTIONS  
8.3 HARD SOLUTIONS  
8.3.1 SEAWALLS  
8.3.2 GROINS  
8.3.3 REVETMENTS  
8.3.4 DETACHED SHORE PARALLEL BREAKWATERS  
8.4 ROUGH TEST  

9  INITIAL DREDGING  

9.1 BORROW  
9.1.1 TERRESTRIAL BORROW  
9.1.2 BACK BARRIER BORROW  
9.1.3 NAVIGATION CHANNEL  
9.1.4 OFFSHORE BORROW  
9.2 SAND  
9.3 VOLUME  
9.4 COSTS  
9.5 ENVIRONMENTAL ISSUES
12.10.3 GROIN HEIGHT LAY OUT ALTERNATIVE 1 AND 2 68
12.10.4 GROIN HEIGHT LAY OUT ALTERNATIVE 3 69
12.11 GROIN CONSTRUCTION AND MAINTENANCE 69
12.11.1 ROCK 69
12.11.2 CONSTRUCTION METHOD 69
12.11.3 ORDER OF CONSTRUCTION 70
12.11.4 MAINTENANCE STRATEGY 70
12.12 DETAILED LAY OUT 71
12.12.1 ROCK SIZE 71
12.12.2 CREST WIDTH 72
12.12.3 ROUGH ESTIMATION VOLUME GROIN 72
12.13 COSTS 73
12.13.1 MATERIAL 73
12.13.2 CONSTRUCTION 74
12.14 CONCLUSION GROINS 74

13 DETACHED BREAKWATERS 76

13.1 INTRODUCTION 76
13.2 SHORELINE RESPONSES 76
13.2.1 SALIENTS 76
13.2.2 TOMBOLO 77
13.2.3 LIMITED SHORELINE RESPONSE 78
13.2.4 GENERAL SHORELINE RESPONSE 78
13.3 PHYSICAL PROCESSES 79
13.3.1 NORMAL MORPHOLOGICAL RESPONSES 79
13.3.2 STORM PROCESSES AND RESPONSE 79
13.4 ADVANTAGES AND DISADVANTAGES OF BREAKWATERS 79
13.4.1 ADVANTAGES OF BREAKWATERS 79
13.4.2 DISADVANTAGES OF BREAKWATERS 80
13.5 STRUCTURAL EFFECTS OF BREAKWATERS 80
13.5.1 LENGTH OF THE SHORELINE TO BE PROTECTED 80
13.5.2 TYPES OF CONSTRUCTION 80
13.5.3 CREST ELEVATION 80
13.5.4 WAVE HEIGHTS 81
13.5.5 CIRCULATION AND MODIFICATION OF CURRENTS 82
13.5.6 EFFECT ON LONGSHORE TRANSPORT 82
13.5.7 EFFECT ON ONSHORE-OFFSHORE TRANSPORT 82
13.5.8 MODELLING 82
13.6 CONSTRUCTION OF BREAKWATERS 84
13.6.1 INTRODUCTION 84
13.6.2 JMC METHOD 84
13.6.3 DIMENSIONAL ANALYSIS FOR DETACHED BREAKWATERS 87
13.6.4 CONCLUSIONS OF THESE DESIGN METHODS 89
13.7 MORE DETAILED DESIGN OF BREAKWATER SYSTEM 90
13.7.1 USE OF STONE AND WAY OF BUILDING 90
13.7.2 WAVE DIFFRACTION AND WAVE TRANSMISSION (PERMEABILITY) 90
13.7.3 TRIAL AND ERROR 92
13.7.4 COSTS 92
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>EVALUATING THE ALTERNATIVES</td>
<td>94</td>
</tr>
<tr>
<td>14.1 INTRODUCTION</td>
<td>94</td>
</tr>
<tr>
<td>14.2 MULTI CRITERIA ANALYZE</td>
<td>95</td>
</tr>
<tr>
<td>CONCLUSION</td>
<td>97</td>
</tr>
<tr>
<td>RECOMMENDATIONS</td>
<td>98</td>
</tr>
<tr>
<td>REFERENCE PROJECTS</td>
<td>99</td>
</tr>
<tr>
<td>LIST OF SYMBOLS</td>
<td>100</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>102</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>104</td>
</tr>
<tr>
<td>LIST OF GRAPHS</td>
<td>105</td>
</tr>
<tr>
<td>SOURCES OF INFORMATION</td>
<td>106</td>
</tr>
<tr>
<td>APPENDICES</td>
<td>107</td>
</tr>
<tr>
<td>APPENDIX A: WIND DATA</td>
<td>107</td>
</tr>
<tr>
<td>APPENDIX B: GRAIN DATA</td>
<td>110</td>
</tr>
<tr>
<td>APPENDIX C: LINEAR WAVE THEORY</td>
<td>112</td>
</tr>
<tr>
<td>APPENDIX D: WAVE HEIGHT AND DEPTH AT INCIPIENT BREAKING</td>
<td>114</td>
</tr>
<tr>
<td>APPENDIX E: CALCULATION SURFZONE LONGUET AND HIGGINS</td>
<td>116</td>
</tr>
<tr>
<td>APPENDIX F: CRESS CALCULATION LONGSHORE SEDIMENT TRANSPORT</td>
<td>117</td>
</tr>
<tr>
<td>APPENDIX G: WAVE SET UP CALCULATIONS</td>
<td>118</td>
</tr>
<tr>
<td>APPENDIX H: THE FLUIDIZING PRINCIPLE</td>
<td>119</td>
</tr>
</tbody>
</table>
Summary

A multidisciplinary team of four master students of Delft University of Technology has been working on a project assignment for the Port of Altamira, Mexico. The assignment has been set up by the Mexican – Dutch dredging company Dragamex, which is a part of the Dutch dredging company Boskalis. The project consists out of an erosion problem at the downdrift side of the Port of Altamira.

The Port of Altamira is one of the fast growing ports in the world and is together with Veracruz and Coatzacoalcos one of the biggest ports of Mexico at the Gulf coast. It is an industrial Port with a lot of potential for expanding. However, the breakwaters of the Port extending into the sea caused severe erosion at the downdrift side of the Port (south side). This ongoing erosion is threatening a precious lagoon and some important land. The erosion can also cause a breakthrough of a dune row, which will result in a flooding of the hinterland. There has to be found a solution for this ongoing erosion. The definition of the problem and the goals of this project are identified in chapter 3.

An analysis has been made in chapter 4 to get insight in the environment around the port, which eventually has a big influence on the erosion. Which coastal processes are causing the initial sedimentation transport and the quantification of the erosion is described in chapter 5.

To determine which solution is the best for this problem, first the wanted situation has to be estimated. This is done by looking at storm condition and the actual situation at the beach in chapter 6. This way there is made an estimation of the requirements which the alternatives have to fulfill after they are constructed. The summary of the total of requirements is given in chapter 7.

The possible alternatives for this erosion problem are:

- Soft Solution - Maintenance dredging
- Soft Solution - By passing system
- Hard Solution - Groins
- Hard Solution - Detached breakwaters

All these alternatives are investigated further in the following chapters. The general description is done in chapter 8. First there is looked at the existing situation and what the wanted natural slope is in chapter 9.

Soft solutions are good, because they are free of human influences on the nature. These solutions restore the old alongshore sediment transport so do not replace the erosion problem.

Maintenance dredging is done by boats which are doing beach nourishments once in a while. In chapter 10 there is investigated how fast the erosion takes place and every how many years there has to a beach nourishment and how much.

By passing system is achieved by a pipeline system from the updrift side to the downdrift side of the Port. At the updrift side there will be an inlet and at the downdrift side there will be an outlet. The dimensions depend strongly on the wanted sediment transport along the coast and these are calculated in chapter 11.

Hard solutions are possible to stop the erosion at the directly downdrift side of the port. However these solution only help the most severe erosion place now, but they move the erosion problem to another place. For the positioning of a construction field there are three alternatives different alternatives.
Groins are long and narrow structured perpendicular to the coast. The design process of groins is very difficult and no guidelines or rules are available for this. No models are used in this report and the estimated dimensions of the groin system in chapter 12 in this report can be seen as very rough design. Groins have two main functions:

- Reducing the sediment transport by realigning the beach toward the dominant wave direction and by blocking the alongshore current.
- Retain a sufficient beach as protection against storm

A groin system will, by interfering with the alongshore sediment transport, catch sand in a groin bay. This way a nice beach will be developed. It is however possible that some sand will flow out this bay due to cross shore transport.

Detached breakwaters are long structures parallel to the coast. There main function is to reduce the wave action at the coast. Due to this reduction the suspended sediment will settle down behind the breakwater causing a Tombolo or Salient as beach. Also the determining of the dimensions of the detached breakwater is a very complex process. This design is normally done by models and a trial - error process. Chapter 13 describes only a rough estimation which is made with two different approaches.

For all the possible alternatives we investigated the estimated dimension, costs and other information, so after this a comparison is made between the different alternatives in chapter 14. This analyzes and comparison of the alternatives is based on different criteria, which give a good overview of the advantages and disadvantages of the solutions.

A final conclusion of the whole investigation will be given in chapter 15. This conclusion is based on chapter 14. The final conclusion can be seen as an advice from our side. But as mentioned before the dimensioning of the different alternatives are very rough and modeling is needed. For the further designing process some recommendation are made in chapter 16.
1 Introduction

Worldwide sea ports have a crucial key function in national and international transport of goods and persons. Regarding to total tonnage maritime transport is the most important form of transport. The economy of certain countries depends on the functioning of its ports. The complexity of ports is huge because of the many different activities that take place at the same time. Processes like guidance, berthing, loading and unloading of the ships, storage and distribution of goods, hinterland transportation, custom controls and surrounding industries cause these ports to become very complex systems.

Since economical areas need sea ports and there are little natural ports in the world, they have to be created. This has almost always huge impacts on its surroundings; positive as negative.

This report investigates two negative effects of a new build port in the area of Altamira, Mexico. An erosion problem is caused by the breakwaters of the port entrance, which interrupts the longshore sediment transportation. And a nature problem; a big nature area was lost to the new build port and its industries. This is partly going to be compensated along the shore.
2  Situation global

2.1  Introduction

Altamira is a port city in the state of Tamaulipas, Mexico and is situated at the Gulf of Mexico. Tamaulipas borders on the states of Veracruz to the south and Nuevo León to the west. To the east Tamaulipas fronts the Gulf of Mexico; to the north Tamaulipas stands on the U.S.-Mexico border, adjacent to the U.S. state of Texas. According to the 2000 census, Tamaulipas had a population of some 2,750,000 people. One of the first ports in the Gulf of Mexico was Tampico. This port is situated in a river mouth and the city is attached to this port. When cargo transport increased in this area the port of Tampico had to expand. But further expanding of this port was not possible due to a lack of space. Also LNG terminals are not a welcome neighbour to urban areas.
For this reasons a new port was developed in the Gulf of Mexico, the Port of Altamira. In 1985 they started building the port in a deserted area near the small city of Altamira. The Port of Altamira is the largest port development complex in the country. Nowadays ranks first in petrochemical fluid handling and is one of the top four in Mexico for the total cargo movement. The port is well located: It has enough space for expanding and there are no towns in the neighbourhood. Another advantage of this port is the possibility for a company to set up a business in Park Altamira, the industrial zone around the port of Altamira. Looking at the global position it is in small reach of the US border and the main markets in Mexico. There are good hinterland connections; by rail and road.

2.2 Technical information

The port has nine ship terminals with a total of 12 berths. The access channel is 3,5 km long and 350 meters wide with an official depth of 14,5 meters. The turning basin has a diameter of 650 meters and a draft of 13,7 meters. The South Channel is 2,5 km long and 250 meters wide with a draft of 11.58 to 12.19 meters. The port receives about 1,200 ship calls each year. The largest vessel that can enter has a length of 300 meters and a draft of 11.88 meters. The port has:
- 2 container terminals ( with 5 ship-to-shore cranes)
- 2 dry bulk terminals
- 5 liquid bulk terminals

2.3 Future of the port

A landmark project for the port of Altamira is a new proposed- terminal that will allow Mexico to import natural gas from overseas. It will be located just north of the turning basin and it will be operated by a Mexican affiliate of the Royal Dutch Shell Group. Due to this development Mexico isn’t longer dependent of the pipeline from the US.
The master plan of the port is a big expansion of the port. The South channel will become longer and there will come a new channel in the North direction. This will mean a real bigger port with much more space for facilities.

The Port of Altamira is a fast growing port with a lot of opportunities of expansion. Companies are interested in setting up a business here, because of the good international location and the availability to build a company on the industrial park of the Port.

Figure 3: Future Plans of the Port of Altamira
3 Problem Description

3.1 Problem analyses

To protect the entrance of the port of Altamira two large breakwaters have been constructed. They have to reduce the wave intrusion in the port, so ships can enter the port safely. A second function is to prevent sedimentation in the entrance channel, so the channel and the port keep their necessary depth.

Because of these two breakwaters the sediment transport along the shore of Altamira is interrupted. The main direction of the long shore current is from north to south, along the coast from Altamira. This gives accretion of the sand on the updrift side of the northern breakwater, which results in a beach, which is growing. At the downdrift side there is erosion, because of the interruption of the sediment transport. This erosion is a threat for the area behind the southern breakwater. In this area is a lagoon, which is separated from the Gulf of Mexico by a thin dune row. And if the erosion continues and the dune row disappears, the lagoon will become part of the Gulf of Mexico. This is a problem for the port since the whole hydraulic system of the area will change. The area will become vulnerable for inundations and incoming waves will be threat for the port’s railway, which lies behind the lagoon. The loss of the lagoon will also have consequences on the ecosystem, which will probably lose its original habitation. Birds lost area and there must come some natural buffer with space for the migratory birds. Directly south of the breakwater lies also a military terrain, which already lost 60 meters of land to the sea. They are also not eager to lose more ground.

Figure 4: Sketch of Erosion Problem
At this moment the dune row between the Gulf of Mexico and the lagoon has at the narrowest point a width of barely 60 meters. In the past, before the construction of the breakwaters, the dune row was much broader and the narrowest point was approximately 200 meters. If nothing will be done to this erosion problem the lagoon will be disappeared in several years.

3.2 Definition of the problem

The unacceptable erosion in the south of the breakwaters of the port of Altamira is threatening the wishes of the stakeholders.

3.3 Goal

Find a solution for the ongoing erosion problems at the south side of the port entrance.
4 Detailed situation

4.1 General

The area studied for the project is positioned just under the Tropic of Cancer in eastern Mexico at the Gulf of Mexico. The area has a tropical maritime climate with high temperatures and very humid air. The average temperature is between 24.6° and 26.5° Celsius, with oscillations of 9.3° Celsius in summer and winter. Rarely, but now and then, extreme situations do occur with temperatures below 0° or above 40° Celsius. The air in this area is extremely humid with an average of 80% during a year. The raining season takes place from June till September. In a year the amount of rainfall will be about 1000 mm. Climate waves from the East can cause tropical depressions, tropical storms and even hurricanes.

4.2 Hurricanes

The tropical storms officially occur from June till the first week of November. These systems originate from the coast of Africa, move across the Atlantic Ocean and move with different paths over the Caribbean Sea and the Gulf of Mexico. They can also originate in the Caribbean Sea or the Gulf of Mexico itself.

4.3 Wind regime

The wind data is gathered from the Airport of Tampico. The location of airport is only 20 km in south direction of the Port of Altamira. The data which is available is from the years 2003, 2004 and 2005. In Appendix A the wind data can be found. There are graphs of the wind speed and occurrence for every direction. From these graphs can be seen that most of the year the direction of the wind is SEE or SE. Winds from the south and the west are really rare. Winds from the north occur occasionally. If the total of occurrence of winds from NEE to NWW and the total occurrence of winds from SEE to SWW, you could say that the main direction of the wind is from the North. For the sediment transport the direction is of importance, but also the wind velocity. A higher wind velocity means higher waves and this means stronger longshore currents (See appendix A for graphs velocity). From these graphs can be seen clearly that the stronger winds occur from the direction north. So the direction of the wind in this region is 50 % of the time from the north. But this 50 % in a year this north wind will be strong. The 50 % south wind is also a lot of the time but these wind are weaker and has less influence on the sediment transport. The wind direction and velocity has a great influence on the wave climate. So the waves from the north will occur more often and will be stronger and will cause more sediment transport. Mainly due to this the main direction of the sediment transport is from North to South.

4.4 Seismology

In the area of Altamira are seismologic movements rare and unknown. Most important seismologic activities occur at a big distance of the studied area.
4.5 Topography

The port of Altamira area exists more or less 10,000 hectares. Its coastal plane, which is characteristic for the Gulf of Mexico, has a system of old dunes which are the highest of this territory. And more to the east (in the direction of the sea) are active dunes which form the coastline. Between these two dune rows are several shallow lagoons and estuaries. Some areas are filled with water and some are dry.

4.6 Oceanography

4.6.1 Waves

The wave climate is determined on base of the reports prepared by Instituto Mexicano del Transporte for API-Altamira. These reports are based on the years: 1999, 2000 and 2001. The information about waves and currents are measured for Playa Miramar, more or less 25 km south of the port of Altamira, over a period of three years. Below is a table shown with the wave direction, significant height and period. The direction indicates where the wave is heading to. The wave height and period is calculated from the wave spectrum. The data of the waves and the currents, which are analyzed, correspond with a buoy in front of the port of Altamira.

<table>
<thead>
<tr>
<th>Season</th>
<th>Most frequent</th>
<th>Direction Asociada</th>
<th>Maximum</th>
<th>Direction Asociada</th>
<th>Period Asociado</th>
</tr>
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<tbody>
<tr>
<td>Spring</td>
<td>0.5 - 1.0</td>
<td>South-North</td>
<td>2.5 – 3.0</td>
<td>South</td>
<td>6 – 8 s</td>
</tr>
<tr>
<td></td>
<td>4 – 6</td>
<td>East</td>
<td>6 – 8</td>
<td>S-E-N</td>
<td>-</td>
</tr>
<tr>
<td>Summer</td>
<td>0.5 – 1.0</td>
<td>North</td>
<td>2.0 – 3.5</td>
<td>North</td>
<td>4 – 8 s</td>
</tr>
<tr>
<td></td>
<td>4 - 6</td>
<td>North</td>
<td>6.0 – 10.0</td>
<td>N-NE</td>
<td>-</td>
</tr>
<tr>
<td>Autumn</td>
<td>0.5 – 1.0</td>
<td>N - S</td>
<td>2.5 – 3.5</td>
<td>North</td>
<td>6 – 8 s</td>
</tr>
<tr>
<td></td>
<td>6 - 8</td>
<td>North</td>
<td>8 – 10</td>
<td>South</td>
<td>-</td>
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<tr>
<td>Winter</td>
<td>0.5 – 1.0</td>
<td>North</td>
<td>3.0 - 4.5</td>
<td>South -SE</td>
<td>6 – 8 s</td>
</tr>
<tr>
<td></td>
<td>6 - 8</td>
<td>North -SE</td>
<td>8 – 10</td>
<td>N-SE-S</td>
<td>-</td>
</tr>
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</table>

Table 1: Wave Heights

<table>
<thead>
<tr>
<th>Season</th>
<th>Velocity most frequent [m/s]</th>
<th>Direction Asociada</th>
<th>Velocity Maximum [m/s]</th>
<th>Direction Asociada</th>
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<tr>
<td>Spring</td>
<td>0.0 – 0.05</td>
<td>-</td>
<td>0.30 – 0.50</td>
<td>North</td>
</tr>
<tr>
<td>Summer</td>
<td>0.0 – 0.05</td>
<td>-</td>
<td>0.30 – 0.50</td>
<td>N-WNW-NE</td>
</tr>
<tr>
<td>Autumn</td>
<td>0.0 – 0.15</td>
<td>-</td>
<td>0.30 – 0.50</td>
<td>South</td>
</tr>
<tr>
<td>Winter</td>
<td>0.0 – 0.10</td>
<td>-</td>
<td>0.30 – 0.50</td>
<td>NW-NNW</td>
</tr>
</tbody>
</table>

Table 2: Wave Height in different seasons

From the data the following conditions are concluded:

Normal wave conditions

The waves attacking the coast are simplified to a periodic wave train with:

- A period of $T_0 = 6$ s
- A deep water wave height $H_0 = 0.75$ m
- Main direction of the waves: 20°
The length of the breakwater is assumed to reach so far into the sea that there is no sediment transport around it. Also the breakwater is impermeable for sand, so no sand particles slipping through the pores of the breakwater. This means that the local longshore transport at the breakwater must be zero. The mean wave angle at this particular point along the coastline with respect to the coastline must be zero as well, otherwise there would still be a longshore sediment transport generated by the waves. The only option is that the coastline rotates in such a way that the wave angle relative to the rotated coastline becomes zero.

**Figure 5: Dominant wave direction**

From figure 5 the main direction of the waves attacking the coast of Altamira can be seen: \( \varphi_0 = 20 \) degrees (This is the angle between the wave direction and the shore normal).

**Storm wave conditions**

These conditions are useful design conditions:

- A period of \( T_0 = 8 \) s
  
  For the wave period the maximum average wave period is taken.

- A deep water wave height \( H_0 = 4.5 \) m
  
  For the wave height the maximum average wave height is taken.

- Direction of the waves

  There is no main direction of the highest waves. With the design of sea defense structures different directions should be taken into account.
4.6.2 Tides

<table>
<thead>
<tr>
<th>Range of the Tide:</th>
<th>average 40 to 60 cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Tide:</td>
<td>Diurnal</td>
</tr>
<tr>
<td>Maximum Low Tide:</td>
<td>- 0.270 m</td>
</tr>
<tr>
<td>Low Tide:</td>
<td>0.000 m</td>
</tr>
<tr>
<td>Mean Sea Level:</td>
<td>0.227 m</td>
</tr>
<tr>
<td>High Tide:</td>
<td>0.472 m</td>
</tr>
<tr>
<td>Maximum High Tide:</td>
<td>0.610 m</td>
</tr>
</tbody>
</table>

Table 3: Tides Data

4.6.3 Currents

a. Local Current

<table>
<thead>
<tr>
<th></th>
<th>Velocity</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>In the Channel</td>
<td>0.10 m/s</td>
<td>&lt;---&gt;</td>
</tr>
<tr>
<td>Cross Shore</td>
<td>0.31 m/s</td>
<td>NE, S</td>
</tr>
<tr>
<td>Longshore</td>
<td>0.25 m/s</td>
<td>N --&gt; S</td>
</tr>
</tbody>
</table>

b. Ocean Currents

The main current in the Gulf of Mexico looks like a complete circle along the coast. It starts near the coast of Campeche (on the peninsula of Yucatan) and circles clockwise to Veracruz. From there it goes north to Rio Bravo and further to the Mississippi Delta. It has an average velocity of 0.9 to 2.7 km/h when it passes the coast of Altamira. This current is not at the direct coastline, but at approximately 100 km out of the coast. This is because the first 100 km are very shallow and these kinds of currents need deeper waters. As a reaction on this main current in the deeper waters, there occurs a counter current in the shallow waters closer to the shore. This results in a current in front of the shore which has a direction from north to south.

Figure 6: Gulf of Mexico Current
4.6.4 Bathymetry

The Port of Altamira is characterised by docks and navigation channels, which form an L-shape and are dredged out of the marshes. The out of the port dredged material is used to fill up the lagoons (which will be used for industrial areas) or dumped far from the coastline (50 km). To use the dredged material for beach nourishment south of the breakwaters is not an option because of contamination of the dredged material. The docks, entrance channel and navigation channel maintain a depth of 12 meters.

![Bathymetry of study area](image)

**Figure 7: Bathymetry of study area**

4.7 On shore and fore shore situation

4.7.1 Littoral Barrier

The littoral barrier at the south side of the breakwaters isn’t in the same condition over the whole length of the studied area. Some sections are more affected by the erosion than other sections and some were already much thicker before the breakwater was built. To get a good insight of the vulnerability of the coast at different places, the coast is divided in cross-sections of 250 meter wide.
Graph 1 gives an insight of thickness of the littoral barrier over the first 10,000 meter, starting at the breakwater and going south. For the thickness of the littoral barrier is the distance taken between the shoreline of the Lagoon and the shoreline of the Gulf of Mexico. In the first 1000 meter there is no Lagoon behind the littoral barrier and the land is in use by the military. In this area the beginning of the dunes is used instead of the Lagoon's shoreline.

Graph 1: Littoral Barrier

In the first 1,500 km the thickness decreases from 85 to 65 meter, from where it fluctuates between 55 and 75 over the next 3.5 km. From there the barrier starts to increase slowly till 7.5 km, from where it starts to increase quickly to a thick and strong barrier.

4.7.2 Dunes

The next figure shows the height of the dunes at the moment. It is clearly that the dunes in certain section are in bad shape; especially between 1500 and 2250 and around 5000. After about 6000 meter the dunes get above 6 meters and from this point the dunes are taken as healthy enough (the exact height isn’t shown from there anymore).
4.7.3 Beach Profile

The dynamic of the coastal line in the surrounding of the breakwater is very intense. On the updrift side of the port there is an accretion of the beach of 500 meters seawards and 2000 meters along the shore northwards. On the other side, downdrift of the breakwaters, the erosion caused a loss of 180 meters land inwards on the most eroded point. The erosion has an influence over more then 5500 meters along the shore southward. From September 1994 till May 1995 the coastline was thrown back for approximately 25 meters. At the moment the erosion process is still active but on a smaller scale. The average loss of the beach profile is estimated at 5 to 10 meters a year. This resulted in a withdrawal of the shoreline and decrement of the beach. The following figure gives an impression of today's beach profile of the first 5 km south of the breakwater.

Graph 3: Beach Profile South

The beach profile is very regular in this area, with just slight changes in profile. Therefore it can be assumed that the beach profile is constant over this area. For the simplification of the beach profile the average profile will be used and taken as constant over the area. The next figure shows the average beach profile.
Graph 4: Average beach profile

The beach profile starts with a quite steep beach slope which gets more flat going seawards. The next table shows the approximation of the slope in various sections.

<table>
<thead>
<tr>
<th>section</th>
<th>0-100</th>
<th>100-250</th>
<th>250-500</th>
<th>500-750</th>
<th>750-1000</th>
<th>1000-1500</th>
<th>1500-2000</th>
<th>2000-2500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope (1 to ...)</td>
<td>40</td>
<td>70</td>
<td>120</td>
<td>220</td>
<td>325</td>
<td>200</td>
<td>200</td>
<td>310</td>
</tr>
</tbody>
</table>

Table 4: Beach Slope

4.7.4 Grain metric

Out of 120 samples of the sand sediment, taken from several locations including beach and sea, the following results can be extracted (See Appendix B). The underwater part of the shore on the north side of the breakwaters fine sand dominates the bottom (0.125 to 0.21 mm), as well as very fine sand (0.0625 to 0.105 mm).

On the south side of the breakwaters the beach constellation is different. The samples are taken on three different places: beach, surf zone and fore surf zone. The grain sizes in these three zones vary from medium/coarse sand (0.25 – 0.42 mm / 0.50 – 0.84 mm) on the beach, very coarse sand (1.00 – 1.68 mm) in the surf zone and fine sand ( 0.0625 – 0.105 mm). There is also a high amount of shell fragments and rocks in this area. This all is tabled in Appendix B.

4.8 Hydrology of the water bodies

The area where the port of Altamira is founded, consists a lot of lagoons with an approximately depth of 500 to 1000 mm, which stretch from the north to the south.

The coastal area of southern Tamaulipas, where the port of Altamira is located, consists of a complex system of lagoons.

4.9 Inundations
Inundations of the lower parts of this area are characteristic for the coast of the study area. The soil consists of fine material. These areas of inundation are located near the sea and are made and constructed under the influence of the sea. River delta's, lagoons and other estuaries are the result.
5 Sediment Transport Processes

5.1 Waves condition at breaker depth

The sea is a very complex combination of current, waves, tides and other forces. The combination of waves, currents, tides and wind is so complex, that it is not possible to calculate their exact value. This is why all the developed formulas are based on estimations.

When waves enter shallow water they will change in dimensions due to refraction and shoaling.

- Shoaling

If a wave approach water which is gradually becoming shallower, like the sandy coast of Altamira, the wave will be affected by the bottom. Nearing the breaker line shoaling occurs: The wave celerity and the wave length will decrease, while the wave height will increase. Towards the breaker line this increasing wave height (and decreasing wave length) gives increasing wave steepness. There will be an upper limit for the wave height,

- Due to maximum wave steepness (H/L)
- And due to a maximum wave height water depth ratio (H/h).

Assume the breaking waves have a constant breaker index\(^1\) \(\gamma = (H_b/h_b) = 0.6\)

- Refraction

Also refraction occurs when waves approach water which is becoming shallower and the wave crests make an angle to the depth contours. The part of the wave crest, which is already in shallower water, will have less celerity then the part of the wave crest which is still in deeper water. Therefore the wave crest will bend toward the coast and the wave crest will get the direction of the bottom contours.

With the linear wave theory now the breaker depth and the wave height were calculated. See appendix C.

<table>
<thead>
<tr>
<th>Deep water conditions</th>
<th>Conditions at the breaker line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave angle (degrees)</td>
<td>20</td>
</tr>
<tr>
<td>Wave length (m)</td>
<td>56.21</td>
</tr>
<tr>
<td>Wave period (s)</td>
<td>6</td>
</tr>
<tr>
<td>Wave height (m)</td>
<td>0.75</td>
</tr>
<tr>
<td>Wave celerity (m/s)</td>
<td>9.47</td>
</tr>
<tr>
<td>Wave group celerity (m/s)</td>
<td>4.68</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5: Wave Characteristics under normal conditions

\(^1\) Kamphuis, J.W. [2000] Introduction to coastal engineering and management
Deep water conditions | Conditions at the breaker line |
--- | --- |
Wave angle (degrees) | 0 | Wave angle (degrees) | 0 |
Wave length (m) | 99.92 | Wave length (m) | 62.32 |
Wave period (s) | 8 | Wave period (s) | 8 |
Wave height (m) | 4.5 | Wave height (m) | 4.35 |
Wave celerity (m/s) | 12.49 | Wave group celerity (m/s) | 6.68 |
Wave group celerity (m/s) | 6.25 | Water depth (m) | 7.25 |
Shoaling factor (-) | 0.967 | Refraction coefficient (-) | 1.000 |

Table 6: Wave characteristics under storm conditions

Note that the wave is assumed to approach the coast with an angle of zero, so perpendicular to the coastline. This is because with this angle the maximum wave height at the breaker line is obtained, so also the biggest water depth. This causes the biggest wave set-up (see next paragraph).

5.2 Sediment Transport

Sediment transport can be defined as the movement of sediment particles in a certain area related with time. The force behind the transport is the bed shear stress caused by the water motion on the bed. When a certain critical bed shear stress is exceeded the particles will start to move. The bed shear stress is induced by a combination of waves and currents. The presence of waves can increase the bed shear stress significantly, compared to a situation with only a current.

Initiation of motion caused by this bed shear stress is the main force behind sedimentation transport. For this project it is needed to know, not when the particles start to move, but what the sediment transport is, integrated over time. To compute the sediment transport distribution it is necessary to determine the velocity and the concentration of the particles. The basic formulation for the sediment transport is:

Sedimentation transport rate \( S \) (m³/s/m²) = Particle velocity (m/s) × concentration (-)

However, in a coastal environment where waves play an important role, both the velocity and the concentration exhibit large variations in time. Due to this it is impossible to find a formula for the Sedimentation transport. A lot of simplifications have to be taken and a lot of different approaches exist. The transport can be divided in two different transports: Bed Load and Suspended Load Transport.
Figure 8: Cross Shore and Long shore

There are two directions for the sediment transport: Longshore transport and Cross shore transport. The longshore transport is parallel to the coast. Cross shore transport is perpendicular to the coast.

5.2.1 Surf zone

Surf zone is created by the breaking waves in this area. The dimensions and calculation of this area is done with the help of the theory of Longuet-Higgins. The theory can be found in Appendix E. In this case the surf zone can schematized like the following:

The wave angle at the breaker line $\alpha_b = 7.6^\circ$

The water depth at the breaker line $h_b = 1.42$ m.
\( \overline{v}_l \) is specifically the longshore current measured at the mid-surf position, halfway between the breaker zone and the shoreline. The maximum magnitude of longshore currents is also roughly in the mid-surf position\(^2\)

\[
x_b = \frac{h_b}{m} = \frac{1.42}{(1/40)} = 56.8m.
\]

5.2.2 Longshore

Currents in longshore direction are induced by radiation stress (\( S_{yx} \)), tidal forces and wind forces. When the waves approach the coast under an angle the radiation stress is the main force for the longshore current. Inside the breakerzone there will be a resulting longshore force on a water column due to the gradient of radiation stress. A counterforce is needed to restore equilibrium of forces in longshore direction. The generation of bottom shear stress due to moving water is the solution. This will result in moving water, which is the longshore current.

**Figure 10: Gradient in radiation stress**

To calculate the amount of sediment transport there has been made use of the program CRESS.

The CERC formula gives the total longshore sediment transport over the breaker zone, generated by the action of waves approaching the coast under an angle. (Appendix F)

The distribution of the sediment transport over the surf zone is not described.

**S (Sediment transport): 288318.01 m\(^3\)/year**

The construction of a port with breakwaters leads to a blocking of the longshore sediment transport (\( dS/dx \neq 0 \)). As a result, the coastline to the left of the port will tend to move seawards (accretion), whereas the coastline on the right side will tend to move landward (erosion).

\(^2\) Beach processes and sedimentation / second edition. Paul D. Komar
Figure 11: Erosion around Port

When the accretion is reaching the end of the breakwater it can also be a problem, because sand will flow into the entrance channel. Of course the biggest problem is the erosion.

5.2.3 Cross Shore

When the waves break on the beach, they cause a rise in the water level between the surf zone and the shore due to the imposition of the momentum of this general movement of water. This increase in water level due to a wave is called ‘wave set-up’. Due to the higher water level there will be a return flow away from the shoreline. This will be a uniform distributed flow if all the waves were the same. But due to the different kinds of waves there can occur rip currents, which take sediment to the sea.

Figure 12: Undertow

For a coast problem it is necessary to taken into account longshore and cross shore transport. However, for many other issues a schematization would lead to a simple representation of reality. That calls for integrated computation models.

5.3 Wave setup and Wave set down

The onshore force generated by waves ($S_w$) must be balanced by an equal and opposite force for equilibrium. This manifests itself as a slope in de mean still-water level, given by $d\eta/dx$

The forces that are acting are the pressure forces, the reaction force on the bottom and the radiation stresses (all forces are wave period averaged).
Outside the breaker zone exists a set-down ($\eta_d$), which is:

$$\eta_d = -\frac{1}{8 \sinh(2kh)} kH_s^2$$

Inside the breaker zone is a wave set-up ($\eta_u$), which results due to the equilibrium in:

$$\eta_u = \left(\frac{1}{1 + \frac{8}{3\gamma^2}}\right) (h_b - h) + \eta_d$$

(for an uniform beach slope)

Especially the wave set-up is an important factor. It will contribute to the overtopping of sea defense structures, during storm conditions. (See appendix G)

- Under normal conditions
  - $\eta_d = -0.03m$ (At the breaker point)
  - $\eta_u = 0.14m$. (At the coastline)

- Under storm conditions
  - $\eta_d = -0.12m$ (At the breaker point)
  - $\eta_u = 0.74m$. (At the coastline)

### 5.4 Wind setup

When a wind blows over the water surface there are stresses between the air and the water. The wind causes the water to pile up higher than the ordinary sea level. This phenomenon is called the "wind set-up". The velocity of the wind during a storm from the East is 70 km/h, which corresponds with approximately 20 m/s (a wind force of 8 on the Beaufort wind scale).

The bathymetry of the coast in front of the erosion zone is schematized in Figure 7. Further then 50 km offshore the water depth is so large, that the contribution to the wind set-up is negligible.
Figure 14: Wind set up

The calculation is done in Appendix G.

With the following constants the wind setup is calculated:
- The friction factor $c_w = 2 \times 10^{-2}$
- $\rho_{\text{air}} = 1.21 \text{ kg/m}^3$
- $\rho_{\text{water}} = 1030 \text{ kg/m}^3$
- $g = 9.81 \text{ m/s}^2$

Total wind set up = 0.18 m

\cite{Karels1979}
6 Approach of requirements alternatives

6.1 Introduction

Before the determination and designing of the alternatives, first have to be thought of what to reach with the solution. What is the wanted result of the alternatives? This is needed to determine the final dimensions of the structures. So there has to be thought of a minimal demand from our side, which the solutions have to fulfil. To come to requirements, which the solutions have to fulfil, there has to be looked at the goal. The goal is to protect the lagoon and prevent the dune belt from collapsing. First will be looked at the area, which has to be protected. Then for the determining two approaches are used: Available data what the hurricane IVAN caused in this region and water set up due to different forces on this coast.

6.2 Worst case scenario

6.2.1 Storms

In the past 15 years the biggest impact by a storm was caused by hurricane IVAN, who past the Gulf of Mexico in the summer of 2004. Although IVAN didn’t get close to Altamira, its waves caused big losses at the littoral barrier. At the places where IVAN struck the hardest a coastal loss of 60 meter was measured and even caused a small breach in the littoral barrier, which was closed short after. Momentary the weakest parts of the littoral barrier are not even two meters high and likely wasn’t much higher in 2004. Therefore the dune loss will be of the same size as the beach loss with such a storm. This 60 meter loss of littoral barrier will be taken as an example for a beach loss by extreme heavy storms.

6.2.2 Wave forces on coast

For maintaining a minimum dry beach to prevent collapsing of the dune belt several aspects have to be considered. The water can attack the dune belt when some conditions will occur. The conditions which have influence on this are described below. In this worst case scenario the demand is the dune row will not be touched.

1. The wave set up in this area is calculated in chapter 5.3 and is 74 cm. This means roughly that a distance around 30 meter beach will be under water.
2. Surface elevation due to wind set up is also calculated in chapter 5.4 as 18 cm which means a run up of 7.2 m
3. Tides, from the chapter 4.6.2 is taken that the tidal range is:
   - Range of the Tide: average 40 to 60 cm
   So here the run up is 24 m.

In total this means that the total of beach length what will be under water in the worse case scenario is around 65 m. This is a lot and highly idealised by just calculating with the beach slope and the total set up. No influence of resistance during the water flowing on the beach has been taken into account. However, in this project, it will be taken into account as a worst case scenario in which the dunes will not be touched and the beach is the protection factor.

6.3 Final requirements from our side

From the above results we can conclude the following things. When the existing littoral barrier is less the 100 m actions have to be taken to protect it. To protect a dune row there has to be a sufficient beach to reduce the forces on this weakened dune row. From the wave forces results
here above it is calculated that with a beach 65 meters the dune are not touched. The littoral barrier just once had a small breach, but is nowadays able to block the sea. We think that with a 50 meter beach in front of the dune belt, the protective ability of the littoral barrier should be big enough. The main thing is that the erosion should not start eating this minimum shoreline any further. So in the end there are two main demands:

- The littoral barrier should be more then 100 m
- If this is not true a beach of 50 meters should be in front of it to protect it

6.4 Coastline protected

Here below is the thickness of the littoral barrier shown, it’s clearly seen which sections got damaged by the erosion and which sections are in good health. So not the entire coastline needs to be protected, just in the unhealthy sections needs to be interfered and improvements made. The section which needs to be improved starts at the breakwaters and ends 5.5 km south of the breakwaters. This section has a littoral barrier thinner than 100 meter and has in most places low dunes. Further south from this section the littoral barrier gets quickly thicker (over 200 hundred meters at 8.5 km) and has high dunes (higher than 6 meter). The 100 meter barrier requirement comes forth out of the analyze on the waves and storm damages. Because the barrier does not fit the minimum shoreline over the first 5.5 km, the littoral barrier must be broadened. The distance of the broadening is shown in the next figure. Orange gives the littoral barrier thickness and yellow the required new beach. The beach in front of the weak littoral beach has to be 50 meters to protect the dune row.

Graph 5: Littoral barrier requirements
7 Program of Conditions

7.1 Essential pre-conditions

- The dune row between the Gulf of Mexico and the lagoon must be preserved to guarantee that the lagoon won’t get lost.
- The loss of land of the military area must be stopped.
- The solutions must be combined with creations of a natural area for birds.
- The new alternative must fit in the nature requirements
- The construction mustn’t have any negative influence on the ships
- Construction shouldn’t have a negative influence at his downdrift side.

7.2 Conditions

- The nature must be kept in account. Next to the creation of natural areas for birds, the other wildlife mustn’t be chased a way.
- Prevent losing the military building
- Prevent collapsing of the dune row
- No influence of solution on beach downdrift.

7.3 Stakeholders

With this erosion and nature problem with its possible solutions several parties are concerned:

- Port authorities

It is the port authority who caused the problem, with the construction of the breakwaters two decades ago. For this they have a certain responsibility for the negative consequences. But the erosion also affects the port facilities.

- Government

The port is important for the economical development of the area. Problems with the port should be tackled. The government has also responsibilities for the wildlife and the control on the environmental consequences.

- Wildlife (groups)

The wildlife in the lagoon could be endangered. Sea turtles breed on the beaches in this area and the bird’s living space has shrunk due to the development of the port.

- Military

The erosion is also affecting the military terrain. There is a building which is nearby the erosion zone and they don’t want to lose more ground.

- Population

The people who live in the neighbourhood of the port and have stakes in the area.
8 Alternatives

8.1 Non-interference

An alternative is to do nothing. This of course causes more erosion and a possible break through of the dune row. When this is not a big damage to the nature it is a possible solution. How fast this erosion goes will be calculated later on. For this project the goal is to find a solution. So this is no option for us. It will be investigated, but won’t be a solution for us.

8.2 Soft solutions

Artificial nourishments do basically not interfere in the occurring sediment transport processes. Therefore this method has to be applied for ever. Material losses have to be refilled from time to time. Seen over a number of years the average coastline will be stable. A big disadvantage of this method is that the refilled material continuously will be taken away; therefore the coastal erosion process remains and new nourishments have to be done after a certain time. The source of the fill material is either outside the nearshore coastal system or inside the coastal system. In the latter case a perfect source of borrow material would be the accumulated sediments at the updrift side of the northern breakwater (the breakwater interrupts the former longshore sediment transport). In that case a sand bypass system is applied.

Figure 15: By pass

Bypass plants, or sand transfer plants as they are sometimes called, are fixed dredges placed on one side of navigational inlets or breakwaters. As sand moves along the shoreline, it is collected by the plant and transferred across harbor structures to down coast beaches.

8.3 Hard solutions

8.3.1 Seawalls

A seawall is a shore parallel structure at the transition between the low-lying beach and the higher mainland or dune. The walls can be vertical or curved to reflect wave power. Modern concrete sea walls tend to be curved to deflect the wave energy back out to sea, reducing the force. Seawalls may be constructed from a variety of materials: most commonly, they are constructed of reinforced concrete, large pieces of rock, steel, or wire cages filled with pebbles. Seawalls require constant maintenance, as the waves will constantly attack the base of the walls and
cause them to be weakened. In case of structural erosion like this problem seawalls are no solution, because volumes of sediment are lost just in front of the seawall so it can become unstable and maybe collapse with possible disastrous consequences.

Figure 16: Seawall

8.3.2 Groins

Groins are rather short structures extending perpendicular to the coast. They interfere in coastal processes close to the shore. Groins are usually made of wood, concrete or, most commonly, piles of large rocks. These can form ‘closed’ structures or ‘open’ structures. The effect of a groin is to accumulate sand on the updrift side. They are effective at causing the deposition of beach material on the one side, but there is a corresponding loss of beach material on the downdrift side, requiring that another groin be built there. Groins are one of the most common coastal defense structures. The big problem with groins is how to design them. Each situation has its own specifics and to translate these into a good design of the groins is very difficult. And when the problem is solved at a certain stretch of coast, what’s happening to another stretch of coast further down drift, because with the groins up drift there will be less sand transport, so maybe an erosion problem occurs there and the problem only has been replaced. All this needs to be researched very carefully in a next part of this study.

Figure 17: Groins

8.3.3 Revetments

Revetments are structures placed on the slopes of the shore in such a way as to absorb the energy of incoming waves. They preserve the existing uses of the shoreline and protect the slope. Like seawalls, revetments armor and protect the land behind them. They may be either watertight, covering the slope completely, or porous, to allow water to filter through after the
wave energy has been dissipated. Also, they are not necessarily filling total height between beach and the surface level of mainland.

Most revetments do not significantly interfere with transport of littoral drift. They do not redirect wave energy to vulnerable unprotected areas, although beaches in front of steep revetments are exposed to erosion. Same as with seawalls, revetments are not optional for this kind of structural erosion problems.

Figure 18: Revetment

8.3.4 Detached shore parallel breakwaters

These are structures built essentially parallel to the coast at a certain distance from the position of the initial coastline, often series of breakwater segments with gaps in between. The crest of the breakwaters can lay above MSL (Mean Sea Level) called emerging breakwaters or the crest can lie below MSL called submerged. Sand will accrete behind these breakwaters in an ellipse form called Tombolos.

Figure 19: Detached Breakwater

8.4 Rough Test

However, groins are increasingly viewed as detrimental to the aesthetics of the coastline, and face strong opposition in many coastal communities.
Seawalls are effective defenses in the short term, but may cause erosion in the long run. They cause the energy of the backwash to be reflected to the beach material beneath and in front of them, so the beach materials are gradually eroded. This problem may be reduced if coupled with beach nourishment (replacement of the eroded material) or rock aprons which reduce wave power by percolating the water slowly through gaps.

![Erosion in front of Seawall](image)

**Figure 20: Erosion in front of Seawall**

Materials eroded from the slope before construction of a revetment may have nourished a neighboring area, however. Accelerated erosion there after the revetment is built can be controlled with a beach-building or beach-protecting structure such as a groin or a breakwater.
9 Initial dredging

To achieve the minimum requirements of the barrier thickness, the dry beach has to increase over the first 5.5 km south of the breakwater. This has to be done by beach nourishment, which has to add such an amount of sand to the shore that the required dry beach is formed. For a beach nourishment there needs to be a sand borrow where the sand is taken from, transportation of the sand and dumping the sand at the disposal area.

9.1 Borrow

There are several source options: terrestrial, back barrier, the navigation channel and offshore. For each borrow there are several ways to transport the sand to the beach and dump it on the shore.

9.1.1 Terrestrial borrow

A terrestrial borrow means that the sand is gained from the land. There needs to be a mine or sand pit where the material can be taken from. Sometimes a pit can be created newly, but there are also commercial companies who can provide the sand. Terrestrial sand is most of the times more expensive than borrowes from the sea; therefore this option is only recommendable if there are no other, say cheaper, options.

For transport there are two options. The first option is to install a pipeline from the borrow to the coast, in which liquefied sand is transported by a big pump. This is an option if the borrow is relatively close to the beach. The other option is to transport the sand by trucks, who are dependent of roads. The problem of the coast of Altamira is that most parts are not reachable by road. Therefore it’s required by this option that or a road is build or an extra transport is used to reach the remote sections (for instance a pipeline). Next to the prices of the borrow, this will be a relatively expensive option because of the expensive transport over land and these infrastructure features for the 5.5 km coast.

9.1.2 Back barrier borrow

Behind the dune belt there are no suitable borrows. There is just the lagoon, which may consist of sand, but it’s not wise to use this. The port’s master plan appoints this lagoon as future land for industrial use. It will take several years to decades before this will be realized, but when this land will be installed as industrial zone, the lagoon has to be filled up to get a dry land which isn’t vulnerable for inundations. Taking first material out of the lagoon and later filling it up again will be double work and costs. This makes the lagoon an un-recommendable borrow.

9.1.3 Navigation Channel

The port of Altamira exits of a navigation channel, turning basin and an entrance channel, which all three needs to maintain a certain depth to let ships navigate safely. Because of several kinds of reasons sediment material enters these areas and decreases the depth. Especially in the port entrance enter a lot of sediment, because of the alongshore transport and the wide port entrance. These areas get dredged every now and than to maintain the necessary depth and to combine this dredging with the beach nourishment looks like a good and cheap solution. The problem is that the sediment material in the port is contaminated by the industry and port activities and should be taken to a deposit point more or less 50 km out of the coast. Therefore this won’t be a suitable solution.
9.1.4 Offshore Borrow

The most common option for this situation would be dredging offshore. The coast in front of the port of Altamira is quite shallow over the first 50 km seaward and consists of sand. It must be quite likely to find a suitable borrow here, which will be big enough and will have the right grain sizes. This dredging should have some distance at minimum of the coast to don’t create another drain for the sediment transport. Except of this it is preferable to have the borrow as close as possible to its dumping place to keep the transportation costs low.

An exception on the non-interference requirement is the accretion section on the updrift side of the breakwaters. This is where the sediment transport is blocked and stored. If the sand will be dredged here and dumped at south side, the beach nourishment will function as a bypass. The consequence is that the accreted beach will disappear and the restoring of the equilibrium in the sediment transport will be disturbed. A positive consequence could be that less sediment would enter the port entrance because there is more stocking place at the north side. The problems of the sedimentation of the port entrance should be taken in account with the suitability of the borrow.

To transport the material from the borrow to the beach again can be done in two ways; by a floating/submerged pipeline or by barges. Depending on the distance between the borrow and beach and the roughness of the sea one of both can be used. The advantage of a pipeline is that the transport to the beach is continues, which enables all the equipment to keep all running. Using a pipeline depends on the location of the winning area in reference to the shipping lanes. Blocking a shipping lane by a pipeline is of course no option. A barge has to be loaded, moved, dumped and moved back again. Most dredging vessels have a barging capacity themselves and can take care of the transport from borrow to beach.

9.2 Sand

The sand material, which will form the supplement, needs to suite the native beach. Especially the grain size is very important to create a stable layer. The median grain sizes of the borrow must get as close to the median grain sizes of the native beach, to secure that the supplement will endure longer under slope and sea conditions of the native beach. If the same median size cannot be found, it’s preferable to take a coarser grain size above a finer.

In appendix B is shown that there are various grain sizes in the different zones of the beach of Altamira. The beach, surf zone and the zone before the surf zone differ a lot. The representative grain size in such case is that of the surf zone. It has the biggest diameter and the surf zone has the most turbulent conditions because of the breaking waves. The grain size of the surf zone at the south side of the breakwaters is that of very coarse sand, which has a median diameter between 1.00 and 1.68 mm.

9.3 Volume

The borrow also needs to contain enough sand for the beach nourishment. The total volume necessary for the beach nourishment needs to be calculated.

As is said before, the weak sections of the littoral barrier needs to strengthen to resist future storms. To achieve a 50 meter beach will be placed to reduce the wave energy before they reach the dunes. The weak zone starts just south of the breakwaters and goes till 5.5 km southwards. From there the original barrier is more than 100 meter and this is assumed as strong enough.
The closure depth is determined by evaluating the beach profile. There is seen that the beach profile hasn’t the same slope. It starts rather steep and turns more flat going seawards. This is quite a parabolic line, except for the slope between 750 and 1000 meter. There the slope is much flatter (1:325) compared to it’s neighboring slopes (1:200). This flatter part will be used for the ending of the beach nourishment. If the nourishment will be landed on this slope, the increase in the slope, caused by the nourishment, won’t be bigger than the neighboring slopes and will be assumed stabile. The average depth between 750 and 1000 meter is 8.3 meter.

Graph 6: Closure depth

In paragraph X.X (requirements) is decided that the 50 meter extra beach will have a slope of 1:40. This means that nourishment needs to begin 1.2 meter above sea level and gives a total nourishment height of 9.5 meter.

To estimate the total volume of the nourishment, these dimensions will be multiplied:
5,500 m (length) * 50 m (broad) * 9.5 m (depth) = 2.61 million m³ sand

The new beach profile will be based on the old profile. It is useful to keep more or less the same shape, because the old shape is the equilibrium shape of this coast. The sediment transport formed this shape over the past decennia and will search this shape in the future. To create a beach close to the old shape lowers the sediment losses caused by the coastal sediment transport.

Next figure gives an idea of the supplement on the average beach profile. Orange gives the original beach, and yellow the nourishment.
Graph 7: Initial beach nourishments

There are several ways to dredge the sand from the bottom. Since we are looking for an offshore sand borrow, the most economical way to dredge is by a trailing suction hopper dredger. This kind of dredger wins the sand like a vacuum cleaner; dragging its ‘arms’ over the sea bottom, while pumps suck up the sand. Which size of dredger should be used depends of the sea bottom depth, distance between borrow and beach and size of project.

As mentioned before, transport can be done by pipelines or by sailing the vessel from the borrow to beach and back. This will depend on the distance and the choice of the dredging company.

The placement of the material can best be done by rainbowing. The disposal area is shallow (deepest 8.3 m.) and not easily reached over land. Therefore a movable rainbow station should be placed which can be moved along the shore or directly rainbowing from the dredging vessel.

9.4 Costs

The costs of the dredging works are hard to predict. Costs depend on several factors: competition, distance between win and disposal area, length of submerged/floating pipeline, size of the vessel in relation to the depth of the win area, type of sand and contract, payment of conditions, etc. Best is to ask a dredger for a quotation.

To give a rough price of the dredging works, following indication is made by a rule of thumb:

USD $ 1,750,000, - for a Mob/Installation
USD $ 6, - a square meter

This results in:

\[ 1.75 \times 10^6 + 6 \times 2.61 \times 10^6 = 17.4 \text{ million} \]

Costs Initial Beach Nourishment $ 17.4 million USD
9.5 Environmental issues

The biggest environmental issue is the sea turtle breeding. Sea turtle breed in the summer time by getting on shore and dig there eggs in the beach sand. When the eggs hatch out, the baby sea turtles climb out of the sand and head for the sea. The coast of Altamira is quite popular by the turtles for breeding and the beach south of the breakwaters is very suitable for it; there are no tourists or other activities that disturb the breeding process.

The beach nourishment creates a good beach for the sea turtles to nest their eggs. But from the other hand, if beach nourishment is done in the breeding season the eggs will get damaged and will not survive it. Therefore it should be investigated if there are already sea turtles breeding in the area or that the beach is too small, steep or turbulent for turtle breeding at the moment. If there are no sea turtles at the moment, the season of the beach nourishment is not restricted. But if there is already sea turtle breeding, it’s better to dredge and nourish outside the sea turtle breeding season.
10 Maintenance dredging

10.1 Introduction

A way to prevent the erosion of the minimum shoreline is creating a buffer in front of it. This will consist of an extra sand supplement which will be placed in the same way as the first ‘Initial Beach Nourishment’. This will not stop the erosion process, because the alongshore sediment transport is still interrupted by the breakwaters and there is still nothing that keeps the sediment on its place. But because the erosion will first start eating the extra supplement, there is no problem until it touches the minimum shoreline. When the erosion reaches the minimum shoreline, a new beach nourishment is needed. So this solution works temporary and needs to be repeated till the erosion is stops because of other reasons.

10.2 Volume

How often beach nourishment has to be repeated, depends on the amount of sand used for the nourishments and the speed of the erosion.

Estimations on the erosion speed can be best done by looking at the blocking of today’s alongshore sediment transport and comparing that with the erosion in the past.

The average yearly alongshore sediment transport in front of the port is 300,000 m³ going from north to south. The assumption is made that there will be a complete blocking of this transport, which leads to zero sediment transport entering the area and 300,000 m³ leaving the area. The total length of the shoreline which needs to be nourished is 5.5 km and the nourishment will have a closure depth of 8.3 m. (the same as the basic nourishment). To calculate how broad the supplement (Bsupplement) needs to be on yearly base can be done by:

\[ 0 \text{ (Sed}_{\text{entering}}) + 5500 \times 8.3 \times B_{\text{supplement}} \text{ (Nourishment)} = 300,000 \text{ (Sed}_{\text{leaving}}) \]

Results in: \( B_{\text{supplement}} = 6.6 \text{ m} \).

For each year the littoral barrier should be broadened by 6.6 m. to keep the amount of sediment on the same level.

The coastal withdrawal over the past 5 years (2000 till 2005) was 5 to 10 m a year. This corresponds with 6.6 m.

It will be likely that the erosion won’t have exactly the same impact over the whole area. There will be slight differences between the sections, although they won’t be that big; the coastal profile is kept quite smooth by the sediment transport over the problem area. Because of this unequal erosion a factor is putted over these 6.6 m to ensure the longevity of the weaker points. The factor is 1.15, which results in a beach of broadening of 7.5 meters a year.

Because erosion is a very complicated process and depends on a lot of factors it is hard to predict. There for monitoring of the behavior of the beach is necessary and will increase the effectiveness or future nourishments.

For the longevity of the first ‘maintenance’ beach nourishment a 7.5 years period is advised, based on the calculation above. This period will give the right balance between working in bigger amounts and not risking the misplacement of to much sand. Working in bigger amounts is cheaper because of scale enlargement; dredging once in 7.5 years will only take once starting costs and transport costs of equipment, instead of moving the equipment once a year. If sand isn’t put in the right places, the erosion will be bigger than expected. To make good use of the information out of the surveys, the first period shouldn’t be too long to reduce the chance of spilled nourishment.

Based on a yearly withdrawal of 7.5 m over 7.5 years the extra beach nourishment will broaden the barrier with 56 m. The total volume of the nourishment will be:

\[ 5500 \text{ (length beach)} \times 8.3 \text{ (closure depth)} \times 56 \text{ (broad)} = 2,570,000 \text{ m}^3 \]
Next figure gives an idea of the Maintenance Beach Nourishment on the average beach profile. Orange gives the original beach, yellow the initial beach nourishment and pink the maintenance beach nourishment.

Graph 8: Maintenance Beach Nourishments

10.3 Costs

Costs can be estimated on the same base as in the Initial Beach Nourishment:

USD  $ 1,750,000, - for a Mob/Installation
USD  $ 6, - a square meter

This results in:

$1.75 \times 10^6 + 6 \times 2.57 \times 10^6 = 17.16 \text{ million}

Costs Initial Beach Nourishment  $ 17.16 million USD
Yearly costs Initial Beach Nourishment  $ 2.29 million USD

10.4 Environmental issues

As said before, the breeding season of the sea turtles should be taken into account for the beach nourishment. If the southern shore of the port’s area is a sea turtle breeding place, the dredging works should be done outside the season. If the dredging period is longer than the non-breeding season, a higher capacity vessel or shorter maintenance dredging period should be considered.
11 Sand Bypassing system

11.1 Introduction

The sand bypassing system has to intercept sand that naturally moves along the coast in a southwards direction towards the entrance of the port of Altamira and move it past the entrance.

A sand bypassing system basically consists of the following main elements:
- The inlet or the sand collecting system: This construction at the north of the port entrance will dredge the sediment and guide it into a pipeline.
- A pipeline: It is transporting the sediment from North to South, running under the entrance channel
- An outlet: The outlet is located on the south side of the entrance channel, where the sediment is deposited onto the beach.

The goal of the sand bypass system is to restore the southern beaches and supply them with sand at rates consistent with natural coastal drift rates.

11.1.1 Capacity of the sand bypassing system

The mean net annual longshore transport rate at the port of Altamira is 300,000 m$^3$. However, there is a seasonal variation in the littoral transport.

The estimations of the annual sediment transport are 800,000 m$^3$/year southwards and 500,000 m$^3$/year northwards. This is because of the changing wind directions in the different seasons. The yearly average becomes therefore 300,000 m$^3$/year southwards. This is only based on estimations$^4$.

\[\text{Figure 21: By-pass system across port}\]

$^4$ Source: survey of Dr. Sergio B. Jimenez Hernandez
Assuming the average transport from North to South has the same intensity as the average transport from South to North. This results in the conclusion that approximately 62% of the time the transport is southwards and 38% of the time the transport is northwards. Assume further that the sand bypass system only is functioning correctly when the current is southwards. When the current is northwards the sand will be transported back in the direction of the port entrance by the longshore current. Especially when the current is directing northwards for a long time this is unwanted, because unnecessarily many sand will flow around the southern breakwater into the entrance channel. (see figure 21)

Because of these reasons the construction of a sand bypass system (and then especially the capacity of it) includes many difficulties:

- A problem complicating the design for a sand bypass plant is to determine the amount of sand actually is transported along the coastline.
  - There are big uncertainties in the actual annual longshore transport
  - It is difficult to measure the rate of the transport
  - There is an annual variation in the net longshore transport rate
  - Because of this the calculation above is based on inaccurate assumptions and estimations.
- Also the differences in the longshore transport on a small time scale are a problem. During a storm the transport rate is much larger then during quite weather conditions
- Another aspect which has to be taken into account is the maintenance of the sand bypass system. The system cannot work continuous because of this maintenance.

Because of these reasons the capacity must be over-dimensioned.

11.1.2 Benefits of a sand bypassing system

- A new equilibrium situation will be created with no further accretion in the north
- The erosion at the southern beaches can be controlled
- Less sediment will pass the northern breakwater into the entrance channel

11.1.3 Some critical notes

- A general problem of a fixed bypass system is that the system could not catch 100% of the longshore sediment transport.
- There must be waited till the sand trap refills with sand supplied by littoral processes before pumping once again.
- As mentioned above, there are many uncertainties concerning the required capacity of the whole system.

11.2 Inlet

As sand moves along the shoreline, it is collected by the inlet and transferred into a pipeline. A look is taken at 3 different possibilities to collect the sand. Assumption: Both the possibilities need an electric powered system. Assume suitable power is located within the port.
11.2.1 Alternative 1: The Submarine Sand Shifter\textsuperscript{5}

One way to collect sand is The Submarine Sand Shifter. The Sand Shifter is based on a fluidizing principle (described in App. H), which allows sand to be dredged from below the sea floor over a horizontal distance of 20 to 40 meters.

The Sand Shifter unit (see figure 22) is buried within the sand accumulations on the up drift side of the port entrance of Altamira (at the North). The Sand Shifter creates a sand trap that captures littoral drift sand.

The captured sand is discharged to an onshore pumping station where the sand/water mixture (or slurry) is pumped through a transfer pipeline to discharge on the South side of the entrance. (see figure 23)

The Sand Shifter units are self burying.

Advantages
- The system could be automated with unattended start and stop of the pumping
- Because the Sand Shifter units are self burying, they don’t need an offshore jetty
- The units are buried, so they
  - are protected from storms
  - don’t obstruct navigation, public access or interrupt the aesthetic lines of the beach

Disadvantages
- It is a system designed by the Australian company “Slurry Systems Marine”. It is probably difficult to install the system in Altamira and it will take extra costs
- Limited capacity
- A pump and pipe system is needed that provides clear water to the fluidizer pipes. The water has to be clear to avoid clogging of the holes in the fluidizer pipe.

A good option is to supply the unit with clear water from within the harbor.

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\textsuperscript{5} http://www.ssm.com.au
11.2.2 Alternative 2: A jetty with fixed dredges

Another possibility is fixed dredges placed on a jetty at the north of the entrance channel of the Port of Altamira.

To collect the sand a sand trap will be developed under the jetty by the operation of a series of submerged jet pumps, which run on water pumped to a high pressure. The pumps will draw-down a series of cones in the beach. Sand that is moved along the surf zone can fall into these cones to be picked up by the jet pumps. See figure 25.

The sand slurry produced will be transported onshore through a pipeline.
Figure 25: of the sand collecting system of the Tweed River Sand Bypassing System

Onshore, a slurry pit will receive the sand slurry from the jetty and concentrate the sand slurry to the required density. From the pit the slurry is pumped through the pipeline to the outlet.

Advantages
- The system can be built with a large capacity
- A large sand trap can be created, so a big part of the longshore sediment transport will be trapped

Disadvantages
- An expensive offshore jetty has to be built
- The jetty
  - is exposed to storms
  - obstructs navigation, public access and interrupts the aesthetic lines of the beach
- A pump and pipe system is needed that provides clear water to the jet pumps. Also with this alternative a good option is to supply the jet pumps with clear water from within the harbor.

11.2.3 Alternative 3: A semi-mobile, land-based equipped system

This system consists of a crane that holds an eductor pump that removes sand from the beach face and two pumps to move the sand through the pipeline.

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6 Sand Transport System at Indian River Inlet, Delaware
Figure 26: A semi-mobile, land-based equipped system

Figure 26 shows a schematization of the system. Some notes:
- The fluidizing water for slurry production is supplied from a water pump near the up-drift side of the port entrance
- A flexible pipe connects the pump house and the crane supported eductor pump
- From a booster pump, the sand slurry is pumped through a fixed pipeline to the other side of the port entrance to the placement area (where the outlet is)

Advantages
- It is a flexible dredging system thanks to the mobile crane
- It is difficult to determine the exact location where the sedimentation will take place. Because of the mobility of this system it is not hindered by this uncertainty
- It is a relatively cheap construction

Disadvantages
- A full-time operator is needed to operate the crane
- A pump and pipe system is needed that provides clear water to the jet pumps Also with this alternative a good option is to supply the jet pumps with clear water from within the harbor
- Because the crane has to stand on the dry beach, a limited distance into the surf zone can be reached
- The system obstructs public access and interrupts the aesthetic lines of the beach
- Limited capacity
11.3 **The pipelines**

Constructing the sand bypassing system the sand has to be transported from the north of the port entrance to the south. A pipeline will have to cross the entrance channel. It is very important that the depth of the entrance channel of the port of Altamira is maintained at all times. The distance from the inlet to the outlet will be approximately 2.5 km.

![Pipeline layout diagram](image)

**Figure 27: Pipeline layout**

11.4 **Outlet**

The slurry existing out of the sand and water mixture will appear very much like ordinary seawater in the surf zone. A strong flow will be seen out of the end of the pipe. The water then flows away leaving the sand behind for the beaches. The outlet will be located on the shoreline/beach. Figure 28 shows an example of an outlet.
11.5 Costs indication of other locations

Tweed Sand Bypass Pumping System:
- **Construction costs**
The system does have a jetty of 450 m. with 10 jet pumps. It makes use of an on shore slurry pit. Totally there are 4 different outlets.
- **Costs when working**
On average it will move about 500,000 m\(^3\) per year and will cost about 3.75 million US$ per year over its life of 25 years (including all costs, administration etc).

The Submarine Sand Shifter of SSM:
- **Construction**
About 3.750.000 US$ (estimated by SSM)
- **Costs when working**
Energy approximately 4 kW/m3 sand (rude estimation, done by SSM: pumping rate of 150 m3/hour, Power consumption in the order of 600 kW \(\rightarrow\) 4 kW/m3)
300.000 m3 / 150 m3/h = 2.000 hours/year the pumps must work
The costs for one kilowatt-hour = approx. 0.12 US$
(1 kW=1000 J/s \(\rightarrow\) 1 kWh = 3.600.000 Joule)
600 kW * 2000 hours = 1.200.000 kWh \(\rightarrow\) 0.12 US$ * 1.200.000 = 144.000 US$ per year (so this is only the cost of the use of energy!)

11.6 Conclusion

A Sand Bypassing System can be a good option to solve the erosion problem of the port of Altamira, however, to make a choice between the different systems further research is needed.
To get a good idea of the construction and maintenance costs the different systems have to be dimensioned.
A well constructed and dimensioned Bypassing System will be a solution for a very long time.
There will be no further accretion in the north and no further erosion in the south: A new equilibrium situation!
All over the world there are more and more bypassing systems operating, so the knowledge about these systems is still growing!
12 Groins

12.1 Introduction

A groin is a relatively long and narrow coastal defense structure, orientated perpendicular to the shoreline. Alongshore sediment transport inside the groin system is reduced due to several aspects. The strength of the alongshore current will be less, because the groin will block a part of this current. A significant effect of introducing a groin system into a length of coastline is to enable the beach within each groin to realign itself towards the dominant wave direction and so reduce the alongshore sediment transport. The direction in which the beach will realign itself in the groin bay is assumed to be the same as the angle of the beach on the north side of the Port of Altamira. Besides the reducing of the alongshore erosion, another function of the groin is to deflect strong tidal currents away from the shoreline and dune row.

A groin system should retain only sufficient material to achieve a beach profile, which is in dynamic equilibrium with the wave climate and provides a reasonable reserve of beach material. A potential problem of the groin system is that the sand that is used for filling up the groin bays is missed at the downdrift side of the groin system. So at the downdrift side of the terminal groin the erosion can increase.

![Figure 29: Alignment of beach in groin bay](image)

This will not be the case when the natural or artificial supply of material is such that the groin bays are full and bypassing of sand along the groins takes place from the beginning.

The overall flow pattern is strongly dependent on the level of the groin crests in relation to still water level and their length in relation to the alongshore current profile.

![Figure 30: Flow pattern in groin field](image)

If groins are impermeable and surface piercing, then these circulation flow patterns are a little bit different. If the groins are submerged, or permeable, then only a proportion of the alongshore
current is diverted offshore. Groin height is the most important in the change of the flow pattern. Higher groins create weaker circulatory flows within the bays, and take care of that the most of the alongshore current passes the head of the groins. The lower groins lead to stratified flows and diverted less of the alongshore current offshore. High local currents may cause a net loss of material from the groin bay which adds to the offshore movement of beach material during storms.

12.2 Assessment of feasibility/suitability of groins

This stage is to determine if a groin system is suitable for the erosion problem of the Port of Altamira. Because of the unique nature and different coastal environments it is not possible to describe some coastal conditions in which groins can provide as a solution. However from the “Guide on the uses of groin in coastal engineering” there are several conditions where it is appropriate to apply a groin system:

- The dominant sediment transport direction of sand is alongshore
- The beach material is bigger than silt of very fine silt sand.
- An insufficient updrift source of material for alongshore drift may be compensated by providing beach nourishment.
- A renourished beach needs to be stabilized and long-term losses reduced.
- Stabilization of the beach is required downdrift of an inlet or harbor

A groin system is not appropriate if the following conditions are true:

- The beach head is erodible and there is a risk of the groin system being outflanked at the groin roots.
- The supply of mobile beach material only provides a thin cover over a solid bed.
- There is no updrift source and the groin bays have to be filled up

In the case of the Port of Altamira it can be said that a groin system is a good solution. The dominant transport direction is alongshore, the beach material is medium sand. There isn’t an updrift supply of sediment, because of the blocking of the breakwaters of the Port of Altamira. But there are several solutions for helping this problem. The groin system will create a beach which is good for attacks of storm. Especially at places where the dune row is very thin, the groins can create new beaches and the lagoon can be preserved.

12.3 Advantages and disadvantages

Advantages:
- Erosion behind the port will stop due to the main functions of the groin
- A wide beach is retained by a groin bay which will give a good protection against storm conditions
- Reduce rip currents which takes sediment offshore

Disadvantages:
- Construction at this side will be difficult
- Erosion moves behind groin field, lee side groin field
- Hazard for recreation
- Expensive

12.4 Erosion lee side groin field

A groin field at the south side of the port will hold the sand at his place and so stop the erosion here. Where the groin field is placed the erosion will stop, but at the downdrift side of the groin
field there is the same problem again. No sediment from updrift side (groin field) and sediment needed downdrift which causes erosion. So when placing a groin field there has to be looked at where this lee side erosion can occur. A solution can be a updrift source of sediment in the dimensions of a by pass system across the Port. This way there is no erosion downdrift of the groin field, because sediment will also by pass the groin field.

12.4.1 Initial filling up groin bays

Another important detail is that the bays have to be filled up artificially from the beginning. Doing this, the bays between the groins will retain there artificially beach profile and don’t have to get the sand from the sediment transport which will reduce the erosion after the terminal groin.

In this case of groins there has to be beach nourishments to restore the amount of beach material which is already lost due to the erosion. The particle size which is used for the beach nourishment should not be finer than the indigenous beach material. So in this case medium sand has to be used. The filling up of the bays will differ every bay, because at every place the existing beach will differ. The filling up process and restoring the natural beach slope is already mentioned in chapter 9.

12.5 Global dimensions

12.5.1 Groin type

At exposed ocean sites, groins are most often of rubble – mound construction, because these structures can withstand wave conditions exceeding original design level, they have relatively low wave reflection coefficients and the ability to reduce the appearance of rip currents in the groin bay. Normally the groins are straight and perpendicular on the coast. There are new developments of T, Y and L shaped groynes to reduce the sediment transport onshore and offshore. But at the port of Altamira the main transport is directed alongshore so there is no need for these different shapes. Groyenes can be divided into two big groups: impermeable and permeable structures. Impermeable structures are constructed of sheet piling or concrete. These kinds of structures will reflect the wave which can have a negative influence on the coast. Permeable groynes can be used on beaches which have sufficient of littoral material for same proportion to pass through them. This will result in a more uniform distributed beach. When the updrift source is constructed with a by pass there will be enough sediment to go trough the permeable breakwaters. Rubble mound groins can be permeable by adjusting the size of the stone and will be used in this case.

12.5.2 Direction of the groins

The angle of the groins with the coastline can be perpendicular, but it is also possible under an angle. For situation where one wave direction is really dominant this is a good solution, but for the situation of Altamira it is not a good solution. In the study area the waves directions differ in a year from north to south, so there will be chosen for groins perpendicular to the coastline.

12.5.3 Length land inward

To obtain a certain security, another goal is to extend each groin into the dune row land inward. This goal is there, because not a lot of wave data is available and there are some uncertainties.
12.6 Alternatives positioning

The construction of groins can be obtained in different forms. There are several alternatives for the erosion problem of the Port of Altamira.

12.6.1 Alternative 1

There can be chosen for a system without a by pass. The system will be constructed from the port till somewhere halfway the beach to Tampico (Figure 31). In this system the groins will extend the entire surf zone so all the sediment will be trapped and will stay in the groin bay. This way the groin can retain a beach. But due to a storm some sediment can be lost, because of the development of cross shore sediment transport. These losses can’t be corrected, because there is no updrift source.

12.6.2 Alternative 2

Another option is to make a groin system, which will be stretched from the Port till the breakwater at the Tampico River. (Figure 31, the yellow lines are the two breakwaters, of Altamira and Tampico) This way the groins have to extend also the entire surf zone to retain a beach. Because the system will extend till the Tampico River there’s no problem at the beach from the port till Tampico. Now the breakwater at Tampico blocks the sediment transport already, so building this solution won’t change anything at this situation. Some offshore sediment transport has to be nourished artificially, but there won’t be great erosion at the beach. This total distance is 25 km.

Figure 31: Groin field from Port Altamira till Tampico Breakwater alternative 2
12.6.3 Alternative 3

This alternative will make use of a bypass system over the Port of Altamira. Sediment will be transported from updrift side to downdrift side with the help of a bypass system. This way there will be an updrift source of sediment. The groins only have to retain a beach length which is wanted. If the groin bays are filled up artificially to there equilibrium all the sand will pass the groin system and continue after the groin system. So this way no erosion behind the groin field will occur due to a lack of sediment. It is not a must that the groin extends the entire surf zone, because when they are filled up they shouldn't interfere with the sediment transport. However they should occupy some of the surf zone, because when due to a storm some sediment is lost, they should have the availability to fill themselves so no artificially filling up is needed.

![Diagram of Groins in combination with bypassing](image)

**Figure 32: Groins in combination with bypassing**

12.6.4 Conclusion Alternatives

A summarizing of the advantages and disadvantages of these alternatives is here below. There is looked at several things:

- How long the groin has to extend into the surf zone to achieve the wanted interference with the sediment transport?
- Till what distance from the port has to be the groin field?
- Is there erosion at lee side of the groin field?
- Is there maintenance filling up needed, because of loss of sediment from groin bay and no filling up by updrift source?

<table>
<thead>
<tr>
<th></th>
<th>Alternative 1</th>
<th>Alternative 2</th>
<th>Alternative 3</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Length groin - surf zone</strong></td>
<td>Entire surf zone</td>
<td>Entire surf zone</td>
<td>30%</td>
</tr>
<tr>
<td><strong>Length groin field</strong></td>
<td>Where erosion is possible?</td>
<td>Tampico breakwater 25 km</td>
<td>Where dune row is strong enough 5500 m</td>
</tr>
<tr>
<td><strong>Erosion lee side groin field?</strong></td>
<td>yes</td>
<td>yes</td>
<td>no</td>
</tr>
<tr>
<td><strong>Maintenance Filling up bays</strong></td>
<td>yes</td>
<td>yes</td>
<td>no</td>
</tr>
</tbody>
</table>

**Table 7: Advantages - Disadvantages alternatives**
Alternative 1 only gives the erosion another place. The erosion isn’t anymore just behind the Port, but is now positioned and heavier behind the groin field. This happens, because there is no updrift source of sediment and after the groin field is no sediment transport. This is very dangerous, because there is no place between Altamira and Tampico where such erosion is convenient. Especially not in the neighborhood of the small dune row which is protecting the lagoon. The groin will extend into the entire surf zone to get as many sand as possible.

Alternative 2 protects the coast between the Port and Tampico is, but the costs to protect the whole coast are enormous. Some offshore sediment transport losses have to be nourished artificially, but there won’t be great erosion at the beach.

Alternative 3 protects only the wanted part where the dune row is not strong enough at this moment. It will only interfere for 30 % with the surf zone, because the sediment transport of the updrift source has to by pass the groin field so no erosion will occur at the lee side of the groin field. They interfere for 30 % to still have the capacity to fill up the groin by themselves so no maintenance filling up is needed.

12.7  Groin Spacing

12.7.1 Ratio Spacing / Length

For the ratio spacing/length the rule of thumb is to have this ratio between 1 and 3. Because of the relative steep slope and fine sediment in this area the ratio should be chosen small. So for this project the ratio is chosen around 1.5.

12.7.2 Groin Spacing

On a coast where the wave direction differs a lot the spacing will be less than a coast where the wave direction is always from one direction. In this case the waves approach the coast of Altamira under several directions so the spacing will be small regarding other applications. Another fact is that steeper beaches require more closely spaced groins. However the mean wave direction is 20 degrees, looking at the updrift side of the existing breakwater, so the beach in the groin bays will become perpendicular in this direction. The spacing should be selected so that, at periods of maximum sediment transport, the angle of the beach head allow the length of the groin to be constructed economically and avoids excessive loading on the groin due to variations in beach level on either side.

Figure 33: Spacing requirements
12.8 Groin length demands

The determining of the length and spacing is judged by two factors:

- To maintain a minimum, dry beach width for specified storm conditions for protection beyond a reference baseline (See chapter 6).
- To interfere with the sediment transport so the groin will block the wanted amount of sediment.

12.8.1 Groin length versus minimum dry beach

As is seen in the above picture (figure 33) you can finally dimension your groin with the help of the requirements. A big requirement in the design of the groins is the wanted minimum dry beach. We set it at 50 m, but maybe more beach is wanted for recreation or other purposes. For the alternatives the 50 m is taken into account. So with the requirement 50 m minimum beach, the estimation that the beach will realign itself toward the dominant wave direction and the estimated ratio spacing / length 1.5 the groin dimensions can be calculated with a following sketch (figure 34) of the situation. This situation is not likely to happen in real life, but it is an estimation of the situation to get a first approach of the length which is needed.

| Ratio Spacing/Length = | 1.5 |
| Dominant wave direction = | 20 |

| Alternative 1 |
| Minimum dry beach | 50 |

<table>
<thead>
<tr>
<th>Length</th>
<th>Spacing</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>687</td>
<td>2.29</td>
</tr>
<tr>
<td>200</td>
<td>412</td>
<td>2.06</td>
</tr>
<tr>
<td>150</td>
<td>275</td>
<td>1.83</td>
</tr>
<tr>
<td><strong>110</strong></td>
<td><strong>165</strong></td>
<td><strong>1.50</strong></td>
</tr>
<tr>
<td>80</td>
<td>82</td>
<td>1.03</td>
</tr>
</tbody>
</table>

Table 8: Groin length with minimum dry beach
12.8.2 Groin length versus surf zone

As mentioned before the sediment transport will occur in the surf zone which is calculated in chapter 5. The groin has to extend into this surf zone to fulfill its function. How far to extend into the surf zone differs every alternative. There are rules of thumbs for this:

- 100% of the longshore transport will be interrupted if the groin extends to $1.67 h_b$.
- 50% of the longshore transport will be interrupted if a high groin extends to $< 0.4 h_b$ or if a low groin extends to $1.67 h_b$.

Groin length versus surf zone alternative 1 and 2

In these alternatives the groin has to extend the entire surf zone to retain and hold as much as possible sediment in the groin bays. To the groin has to extend until:

- 100% of the longshore transport will be interrupted if the groin extends to $1.67 h_b$.

With a breaker depth of 1.42 meters (chapter 5) this means the groins has to extend till a depth of

- Depth seaward tip of groin = $1.42 \text{ m (breaker depth)} \times 1.67 = 2.37 \text{ meters}$.

This means with a beach slope of 1:40 that the groin length has to be:

**Estimation length into surf zone alternative 1 and 2:** 95 meter

**CRESS?**
Groin length versus surf zone alternative 3

When the bays are filled, this amount has to by pass the groin system so no erosion occurs behind the terminal groin. To achieve this there must be looked at the length the groin has to extend into the surf zone. The goal is to fill the bays in advance and that the sediment will pass the groin system and no sediment is needed to fill up the bays which will have an effect on the erosion behind the groin field. But when a storm occurs some sediment can be taken away from the groin bays and then it must have the ability to fill itself. So the goal is the groin must interrupt only for about 30% of the longshore transport.

A Rule of thumb for 50 %:

- 50% of the longshore transport will be interrupted if a high groin extends to < 0.4 \( h_b \) or if a low groin extends to 1.67 \( h_b \)

Off course this is only an estimation, but there can be made a rough first length estimation out of this. Here there is chosen for a relative high groin, so the rule will be that the groin has to extend to about the average of 0.7 times the breaker depth. This result, with the calculated breaker depth of 1.42 (Chapter 5), in a groin that has to extend to:

- Depth seaward tip of groin for 50% = 1.42 m (breaker depth) x 0.7 = 0.994 m
- Length of groin for 50% = 40 (bottom slope) x 0.994 = 40 m

But in this situation only 30% of the longshore transport is wanted to be blocked so the estimated length will be:

**Estimation length into surf zone alternative 3:** 30 meter

12.8.3 Final groin dimensions

As mentioned the groin length have to fulfill two demands

- Interfere with the sediment transport
- Retain a minimum dry beach

This is calculated for the different alternatives. The finally results are potted in the figure here below.

<table>
<thead>
<tr>
<th></th>
<th>Alternative 1</th>
<th>Alternative 2</th>
<th>Alternative 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length groin versus</td>
<td>110 m</td>
<td>110 m</td>
<td>110 m</td>
</tr>
<tr>
<td>minimum dry beach</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length groin versus</td>
<td>95 m</td>
<td>95 m</td>
<td>30 m</td>
</tr>
<tr>
<td>surfzone</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 9: Final demands length alternatives

It is difficult to determine the final groin length with these two demands. We also want that the groin will extend into the existing dune row.

For alternative 1 and 2 the existing beach wide also plays an important role. If there is already a wide beach then extending the beach into the existing dune row and extend 95 meters into the surf zone is resulting in a really big groin. For example if the existing beach is only 10 meters, then the groin will be 110 meters. But if the existing beach is 30 meters then the groin length will be 95 meters together with these 30 meters, which results in a groin length of 125 meters. The existing beach wide is in the study area not more than 40 meters till the accretion area at the breakwater in Tampico, so the final groin length for alternative 1 and 2 is
Length needed into surf zone = 95m
Length beach (groin needs to go into dune row) = 40m
Total needed length groin = 135 meters.

So when there is only a beach now of 10 meters the groin will extend for 125 meters into the sea. But this is not bad and only more sand is retained. The length is kept over the whole groin field the same as well as the spacing.

For alternative 3 the demand for the 50 meter minimum dry beach results in a groin length of 110 which every where in the study are includes the 30 meters into the surf zone, because the beach is not more then 40 meters.

The final dimensions of the groin system per alternative can be seen in the table 10. The estimated final dimensions are the highlighted dimensions per alternative. The other combination is to show what the dimensions would have been, taken another ratio spacing/length.

### Alternative 1

<table>
<thead>
<tr>
<th>Minimum dry beach</th>
<th>Dominant wave direction</th>
<th>Length of groin field</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 m</td>
<td>20°</td>
<td>?</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Length</th>
<th>Spacing</th>
<th>Ratio</th>
<th>How many groins</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>82</td>
<td>1.03</td>
<td>?</td>
</tr>
<tr>
<td>135</td>
<td>203</td>
<td>1.50</td>
<td>?</td>
</tr>
<tr>
<td>300</td>
<td>687</td>
<td>2.29</td>
<td>?</td>
</tr>
</tbody>
</table>

### Alternative 2

<table>
<thead>
<tr>
<th>Minimum dry beach</th>
<th>Dominant wave direction</th>
<th>Length of groin field</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 m</td>
<td>20°</td>
<td>25000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Length</th>
<th>Spacing</th>
<th>Ratio</th>
<th>How many groins</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>82</td>
<td>1.03</td>
<td>305</td>
</tr>
<tr>
<td>135</td>
<td>203</td>
<td>1.50</td>
<td>123</td>
</tr>
<tr>
<td>300</td>
<td>687</td>
<td>2.29</td>
<td>36</td>
</tr>
</tbody>
</table>

### Alternative 3

<table>
<thead>
<tr>
<th>Minimum dry beach</th>
<th>Minimum interference</th>
<th>Dominant wave direction</th>
<th>Length of groin field</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>30</td>
<td>20°</td>
<td>5500</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Length</th>
<th>Spacing</th>
<th>Ratio</th>
<th>How many</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>687</td>
<td>2.29</td>
<td>8</td>
</tr>
<tr>
<td>200</td>
<td>412</td>
<td>2.06</td>
<td>13</td>
</tr>
<tr>
<td><strong>110</strong></td>
<td><strong>165</strong></td>
<td><strong>1.50</strong></td>
<td><strong>33</strong></td>
</tr>
<tr>
<td>90</td>
<td>110</td>
<td>1.22</td>
<td>50</td>
</tr>
<tr>
<td>70</td>
<td>55</td>
<td>0.78</td>
<td>100</td>
</tr>
</tbody>
</table>

**Table 10: Final estimation dimensions alternatives**

To compare this result there will be looked also at already existing groin systems. Here the length in relation with surf zone is compared with database information. In the “Guide on the
uses of the groins in coastal engineering” there is a database of existing groins. From the database of groins there can be made an average of lengths and groin spacing. This database is plotted in the table here below.

<table>
<thead>
<tr>
<th>Beach type</th>
<th>Average length and ratios</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Groine length (m)</td>
<td>Groine spacing (m)</td>
</tr>
<tr>
<td>Shingle</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>Shingle upper/sand lower</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Shingle/sand mixed</td>
<td>70</td>
<td>85</td>
</tr>
<tr>
<td>Sand</td>
<td>95</td>
<td>130</td>
</tr>
</tbody>
</table>

Table 11: Database of groins

So in the case of Port of Altamira the first estimate of dimensions of the groins will be:
Length: 95 m
Spacing: 130 m
Range of Spacing/Length: 0.8 to 2.7

So the length of 110 m and a spacing of 165 m is not a bad estimation. The other estimation of a length of 135 meter and a spacing of 203 is rather big. The spacing is taken at 1.5 but in Table 10 other ratio are also calculated.

12.9 Transition

A transition should be used to tie the groin field into the existing downdrift beach. This will happen at the downdrift end of the groin system of alternative 1 and 3. Alternative 2 will have a big terminal groin in the sense of the breakwater of Tampico. This because there is a certainty that the lagoon is well protected and no inundations will occur.

Figure 35: Transition of a groin field

Kressner (1928) and Bruun (1952) both found that the transition is most effective if a line connecting the seaward ends of the shortened groins and the last full – length groin meets the natural shore alignment at an angle of 6 degrees.
\[ s_n = \frac{R_n}{1 + \frac{R_n}{2} \tan 6^\circ} l_{n-1} \]
\[ l_n = \frac{1 - \frac{R_n}{2} \tan 6^\circ}{1 + \frac{R_n}{2} \tan 6^\circ} l_{n-1} \]

Using these formulas with the estimated ratio and the estimated length and spacing the following transition is calculated:

### Alternative 1

| Length of groins in the groin field (L_n) | 135 m |
| Ratio of groin spacing to groin length in the groin field (R) | 1.5 |
| Coefficient spacing | 1.39 |
| Coefficient length | 0.85386 |

<table>
<thead>
<tr>
<th></th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>First groin in transition</td>
<td>115 m</td>
<td>188 m</td>
</tr>
<tr>
<td>Second groin</td>
<td>98 m</td>
<td>160 m</td>
</tr>
<tr>
<td>Third groin</td>
<td>84 m</td>
<td>136 m</td>
</tr>
<tr>
<td>Fourth groin</td>
<td>72 m</td>
<td>117 m</td>
</tr>
<tr>
<td>Fifth groin</td>
<td>61 m</td>
<td>100 m</td>
</tr>
<tr>
<td>Total length of transition field</td>
<td>701 m</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Length</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>First groin in transition</td>
<td>94 m</td>
<td>153 m</td>
</tr>
<tr>
<td>Second groin</td>
<td>80 m</td>
<td>131 m</td>
</tr>
<tr>
<td>Third groin</td>
<td>68 m</td>
<td>111 m</td>
</tr>
<tr>
<td>Fourth groin</td>
<td>58 m</td>
<td>95 m</td>
</tr>
<tr>
<td>Total length of transition field</td>
<td>490 m</td>
<td></td>
</tr>
</tbody>
</table>

Table 12: Transition alternatives

12.10 Groin Height

12.10.1 Introduction

No analysis of groin height exists. This is due the problem of defining height when the height of groin protruding above the beach profile is changing due to the beach response to the varying
wave climate. The landward end of the groins must extend to a point above the high water line in order to stay beyond the normal zone of beach movement. They must reach into a stable beach back features, so in this case the groins has to extend to the existing dune row. An increase in groin height to improve beach levels could lead to rip currents and erosion gullies. This is of course not convenient for the sand beaches in this project. As a guideline for the height the following is taken into account:

- The underside of the groin should be under the lowest recorded or predicted beach profile.
- The upper side of the groin should be capable of 0.5 meter above the average summer profile for sand beaches.

Typical groin is divided in three sections: (1) the horizontal shore section (HSS), (2) the intermediate sloped section (ISS) and (3) the outer section (OS).

![Groin profile diagram](image)

**Figure 36: Groin profile**

The HSS should extend a sufficient distance landward to prevent groin flanking. The height of the core of the HSS is often set at the height of the natural berm. The ISS should be parallel to the natural foreshore slope. The crest elevation of the OS should be as low as possible, although groins often built from land, which means that the groin must rise high enough out of the water to allow access for construction equipment. Additionally, for safety, the crest elevation should be visible under normal tide conditions. Average water level is 0.270 meter. The height of the seaward end will differ per alternative. For alternative 1 and 2 the goal is to maintain as much as possible sediment in the groin bay so the seaward end will be at mean water level. For alternative 3 the groin can let some sediment pass over the groin so the groin height of the seaward end will be at mean low water level (-0.270 m).

12.10.2 Beach profile first 200 meters

First there is looked at the beach profile of the first 200 meters what is interesting for the groins. The beach profile is figured for 500 meters and 5000 meters southward of the port.
Graph 9: Beach profile

It is seen that the profile at 5000 meters is slightly different from the profile of 500 meters. Another feature that can be seen is that there is not a lot of beach.

12.10.3 Groin Height lay out alternative 1 and 2

Now with the given information there is made an estimation of the groin height lay out for alternative 1 and 2. Groin height seaward end is mean water level and the length is of course 135 meter.
12.10.4 Groin Height lay out alternative 3

The same is done for alternative 3 but now the seaward end groin height will be at low water level of – 0.270 m. So the following lay out is estimated with a length of 110 m.

Graph 11: Final lay out Alternative 3

12.11 Groin construction and maintenance

12.11.1 Rock

Rock used in the groin must be hard and durable to withstand the action of waves and other marine forces. The rock in the study area is gathered 165 km from the port so a lot of transport by trucks is needed.

12.11.2 Construction method

Although the construction of a groin does not look like a difficult construction, the construction activities are complicated by the limitations on access of the working due to the tidal forces. A rubble mound structure has the advantage that the permanent works continue to provide temporary access to maintenance plant. But how the works are done is difficult to say.

Construction from the land is needed, because a construction of a groin head into the dune row is not possible from the sea. However in this area there are no access roads to the beach where the groin field is planned. So if the construction is done by land then access roads also have to be built, because access over the small and weak dune row is impossible. To get access to the beach, there has to be a lot of beach nourishments and strengthen of the beach to finally get access with heavy land based equipment. The rock components in the sea can be done by sea based equipment. The building has to be done by a crane on a pontoon. The problem is however that the depth here is very little.
12.11.3 Order of construction

The sequence in which a groin field is constructed is a practical design consideration and may not be straightforward. To minimize downdrift impacts, beach nourishments and groin construction should be concurrent. Construction of the first groin should be at the downdrift end of the project, preferably the terminal groin adjacent to an inlet. Net drift will combine with the artificial beach nourishment to fill and stabilize the first compartment. The second groin is then constructed and the process repeated. Gradually working updrift, the groin field construction is completed. This process together with tapering the ends will help to minimize the impact to adjacent, downdrift beaches. This method may increase costs, but it also may result in a more practical guide to spacing of the groins than originally designed.

![Order of construction](image)

**Figure 37: Order of construction**

12.11.4 Maintenance strategy

For a groin system it is important that each groin is maintained in a good condition to fulfill his function. The maintenance can be seen as the repair and replacement of the component elements or of complete sections of the groin, which have failed. Adjustments in the lay out can be made if the groin does not fulfill the wanted dry beach or the wanted interference with the sediment transport. The height can also be changed if it is necessary. If this maintenance does not happen there can be a lot of damage done to the groin system. A failure of 1 groin will lead to a lot of sediment losses and malfunctioning of the whole groin system. Little damage of a groin can become really big during storm conditions. Therefore it is necessary to inspect the groins after a severe storm. Some things have to be taken into account which determines the costs of maintenance:

- Because of limitations on access, maintenance work on the seaward end of groins can be time-consuming and costly. So the more expensive durable rock components can pay back in the end with less maintenance.
- Groins are needed to be repaired quickly after a severe storm, but then the weather condition is really bad which results in costly labor work.
- Loose rock groins are very easily maintained as reshaping or topping – up the rock can be carried out during most tides, provided a reserve of rock and handling equipment is available at the site. Maintenance of rock groins can become effective and simple if the top level can be raised to above normal high tide with a sufficient for dumpers and cranes to operate.
• Keeping a good beach profile can reduce the wave forces on the groins. So doing maintenance beach nourishments to retain a sufficient beach will help the groins in their damaging.

12.12 Detailed lay out

12.12.1 Rock size

For the design of a rubble mound groin the design condition are different than for sediment calculations. Now the extreme conditions are taken. From the information which is available the following conditions are taken into account (see chapter 4).

Design wave height: 4.5 meter
Design wave period: 8 s
Number of occurrence of this wave: 1000 times
Slope of construction: 1:3

The global lay out of a rubble mound structure is figured here below. The crest will contain 2 or 3 stone diameters and in general the stones will have a mass of 1 to 3 ton. The armor layer will be 2 stones thick.

![Figure 38: Detailed Lay out Rubble Mound Groin](image)

Here we will use the van der Meer formula which was originally used for determining the armor layer of a breakwater, but we will use it here to get a first estimation of the rock size.

\[
\frac{H_s}{\Delta D_{50}} = 6.2T^{0.18} \left( \frac{S_d}{\sqrt{N}} \right)^{0.2} \xi^{-0.5}
\]

For plunging waves

The criteria to use this formula are that the Iribarren is below a critical value of:

\[
\xi_{mc} = \left[ 6.2T^{0.31} \sqrt{\tan \alpha} \right]^{\frac{1}{T+0.5}}
\]
The Iribarren number is calculated by:

\[ \xi_m = \frac{\tan \alpha}{\sqrt{\frac{H}{L}}} \]

| \(H_s\)  | Wave height       | 4.5 m |
|\(\Delta\) | Relative density  | 1.65  |
|\(P\)     | Notional permeability according vd Meer only rock construction | 0.6   |
|\(\tan \alpha\) | Slope angle       | 0.33333 |
|\(S\)     | Damage level      | 2     |
|\(\xi_m\) | Iribarren number (calculated) | 1.57  |
|\(\xi_{mc}\) | Critical Iribarren number (calculated) | 3.318 |
|\(N\)     | Number of incident waves | 1000  |

Final answer from a calculation is that the needed rock diameter for the groin is

- \(D_{n50} = 1.05 \text{ m}\) Rock diameter

This will result in a mass of:

\[ D_{n50} = \left( \frac{W_{50}}{\rho} \right)^{1/3} \]

- \(W_{50} = 3066 \text{ kg}\) Rock mass

This mass is high in comparison with the normal used 1 – 3 ton in groins. But as a first estimation it will be taken into account. The groins are mostly at the seaward side of the breaker line so the occurring waves can occur. This is why big rock sizes are needed.

12.12.2 Crest width

The crest width of a groin is mostly set on 3 times the rock diameter, which will result in this case:

- \(\text{Crest width} = 3 \times 1.05 \text{ m} \approx 3 \text{ meter}\)

12.12.3 Rough estimation volume groin

The volume of one groin will be a very rough estimation to get a global idea of the amount of rock needed. The groin height starts at the beach with a height of 0.5 meter but will eventually have a groin height of 3.5 meters (see graph 10 and graph 11). We set the average height of the groin at 2.5 meter (see figure below) and calculated this way the total volume with the following cross section lay out.
This results for the different alternatives with a simple calculation:

- Alternative 1 and 2: 3543.75 m³
- Alternative 3: 2887.5 m³

With an estimated average porosity of 40% this means that of this volume the rock volume will be:

- Alternative 1 and 2: 2126.25 m³
- Alternative 3: 1732.5 m³

So with a density of 2650 kg/m³ of the rock the needed mass needed very groin is:

- Alternative 1 and 2: 5634 ton
- Alternative 2: 4591 ton

### 12.13 Costs

#### 12.13.1 Material

The wanted diameter of the rock is available here and the average costs are 16.5 dollar/ton (information from Port Authorities of Altamira). This is pure the costs of the material. The volume which is needed is calculated here above very roughly. These costs are only to indicate what the rough costs are of a groin.

- Alternative 1 and 2: 90144 dollar/groin
- Alternative 3: 75752 dollar/groin

The total amount of groins needed for alternative 1 is not known, but for alternatives 2 and 3 the total amount of groins needed is known. The total costs of the groin field can then be calculated for alternative 2 and 3 and will be:

- Alternative 2: 11.1 million dollar
- Alternative 3: 2.5 million dollar

As mentioned before these costs are only the material and not the equipment, labor and construction costs. For alternative 3 also the costs of the by pass system has to be taken into account.
12.13.2 Construction

For the costs of the construction of a groin there have to be determined how the groin are to be build. The construction from the land is possible but will give a lot of work. There are no access roads to the beach and the existing dune row is too small to work on. So first the costs of making access to the beach have to be taken into account. Then the construction costs of the groin itself have to determine with the help of the costs of labor and the land based equipment.

For the construction of the groin from the water there is not a lot of preparation needed. However the construction process from the sea with labor and sea based equipment is more expensive then from land.

Offshore construction requires a pontoon. A standard size pontoon available in this area is the pontoon with a load-capacity of 2400 tons. To give an estimation of the rental costs for such a pontoon, several things have to be taken in account (all prices are sourced by Dragamex and depend on many more factors). First costs have to be made because of mobilization (and demobilization). For the Gulf of Mexico this will be around $ 100.000 USD, including towing-insurance. The pontoon itself will cost around $ 700 USD per day. This is excluding insurance and protection matters that have to be taken by the user. Also very common is an in- and out survey where the devaluation of the pontoon will be determined over the rental time and this has to be paid by the user. For all these things there will be an additional 10% costs to the daily rental costs. Last thing that must be taken in account is that the pontoon is going to be used to put a big crane on it. So the pontoon has to be stabilized. Either four winches or anchors are needed or a spud-system is necessary. Rough estimated this will cost an additional $ 75 USD per day.

No information is available of the costs of these two construction approaches so no good conclusion can be made.

12.14 Conclusion groins

As can be seen, from the above report about groins, the groin field design process is very difficult. There are not some guidelines and rules to determine the final dimensions. This is mainly caused by all the different environments around the world where groins are constructed. Nowhere the weather conditions are the same. This makes it impossible to give some rules to design. These days a groin system is designed with the help of numerical or physical models. We didn’t have these models available so the final design of this report can be seen as a very rough estimation of the final groin system lay out. After this, defining the groin system is also a trial and error process.

Another feature what makes it difficult is the determining of what the minimum beach has to be. We choose for a minimum dry beach of 50 meters, but maybe the stakeholder wants a 150 minimum dry beach for recreation. So this report can be seen as a design from our side.

Three alternatives are mentioned in the report. Looking at these alternatives it is very hard to see the groins as a future possibility. Alternative 1 only replaces the erosion problem to another location. Alternative 2 gives a very large groin system which results in enormous costs. Alternative 3 gives a good solution, but in this design maybe alone a by pass system is already sufficient. The groin system in combination with a by pass will retain a more beach than only a by pass system so it will be a better protection for the dune row. But the costs are much higher, so there has to be investigated if it is really a profit to construct this combination.
Summarizing dimensions

<table>
<thead>
<tr>
<th>Feature</th>
<th>Alternative 1</th>
<th>Alternative 2</th>
<th>Alternative 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erosion lee side groin system</td>
<td>yes</td>
<td>yes</td>
<td>No</td>
</tr>
<tr>
<td>Initial filling up groin bays</td>
<td>yes</td>
<td>yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Groin type</td>
<td>Rubble mound</td>
<td>Rubble mound</td>
<td>Rubble mound</td>
</tr>
<tr>
<td>Direction groin</td>
<td>Perpendicular</td>
<td>Perpendicular</td>
<td>Perpendicular</td>
</tr>
<tr>
<td>Extending land inward</td>
<td>Into dune row</td>
<td>Into dune row</td>
<td>Into dune row</td>
</tr>
<tr>
<td>Length groin field</td>
<td>5500 m</td>
<td>25000 m</td>
<td>5500 m</td>
</tr>
<tr>
<td>Ratio Spacing / Length</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Estimation length</td>
<td>110 m</td>
<td>135 m</td>
<td>135 m</td>
</tr>
<tr>
<td>Estimation spacing</td>
<td>165 m</td>
<td>203 m</td>
<td>203 m</td>
</tr>
<tr>
<td>Transition</td>
<td>yes</td>
<td>yes</td>
<td>yes</td>
</tr>
<tr>
<td>Volume of rock in 1 groin</td>
<td>1237.5 m³</td>
<td>1237.5 m³</td>
<td>1237.5 m³</td>
</tr>
<tr>
<td>Total costs material</td>
<td>1.4 million euro</td>
<td>6.4 million euro</td>
<td>6.4 million euro</td>
</tr>
</tbody>
</table>

For determining the dimensions of the groin system every alternative a lot of assumptions are made:

- A minimum dry beach of 50 meters is wanted
- The groin bays will be full, so the sand will be at the tip of the downdrift groin. (figure 34) and needed length versus minimum dry beach is calculated according this figure.
- The beach will realign itself toward the dominant wave direction
- The ratio Spacing / Length will be has to be from 1 to 3, it is taken in this case as 1.5
- The rules of thumbs are taken as a guideline to determine the length into the surf zone.

The costs are very rough estimated from a very rough estimated volume per groin. The costs are pure an indication. The construction of the groin is difficult. There is no access to the beach from the land and the dune row is very weak to work on. So access roads have to be build and a lot of beach nourishments together with beach strengthen has to take place before you can access the beach with equipment. Construction from the water is also difficult, because this way the water is to shallow to reach the dune row for construction. The costs for construction are not available.
13 Detached Breakwaters

13.1 Introduction

Shore-parallel, detached breakwaters may be built singly or in series spaced along the shoreline. These beach stabilization structures alone do not provide the sand to maintain a wide protective beach; they simply redistribute available sand. Accretion in one area is balanced by erosion elsewhere unless additional sand is introduced into the project area. Without concomitant beach nourishment, the degree of allowable adverse effects needs to be addressed. If negative impacts cannot be tolerated, beach nourishment must be included in the project. The coastal structures placed in conjunction with beach nourishment can often increase the residence time of the sand, keeping it on the beach within the project area for a longer period of time. If the savings realized by reducing the time between required nourishments exceeds the cost of the structures, their construction can be justified.

Detached breakwaters are constructed close to shore to protect a certain stretch of shoreline against low to moderate wave action and to reduce severe wave action and beach erosion. Sand transported along the beach is carried into the sheltered area behind the breakwater where it is deposited in the lower wave energy region. The effectiveness of a nearshore breakwater or breakwater system depends on the level of wave protection and the length of the shoreline it protects. Important characteristics are the breakwater height, length, wave transmission and distance from shore. Dealing with a system of breakwaters, the width of the gap between adjacent breakwaters and the length of the individual breakwater segments are also important.

The design process is most often one of trial and error. A trial breakwater configuration is assumed based on past experience at existing breakwater systems. Then the trial configuration is evaluated to determine if it will satisfy the project’s objectives. Its effect on the shoreline and on the overall sediment budget of the project area and adjacent beaches is evaluated. The trial configuration is adjusted and the modified project’s effects evaluated. Evaluation tools for proposed breakwater configurations include the interpretation of diffraction analyses, overtopping analyses and other manual computations; physical model tests of the proposed project configuration; and numerical computer simulations of offshore evolution. Because of the limited experience with prototype detached breakwaters, a great deal of engineering judgement and comparison with the few existing breakwater projects is necessary.

13.2 Shoreline responses

13.2.1 Salients

A salient is the preferred shoreline response for a detached breakwater system (see figure 39). This is to allow longshore sediment transport to continue to move through the project area to downdrift beaches.

![Figure 39 Salient formation (source: EM-1110-2-1617)](source: EM-1110-2-1617)
Salients are likely to predominate when the breakwaters are sufficiently far from shore, short relative to incident wavelength, and relatively transmissible (low crested or large gaps with low sediment input). Wave action and longshore currents tend to keep the salient from connecting to the structure. Salient formation provides a recreational swimming environment, but recreational visits in Altamira are not very likely in this area due to the limited (almost impossible) accessibility. Salient formation limits access for maintenance. The most important condition to determine whether or not salients will be formed is the length of a single breakwater \((l)\) derived by its distance offshore \((y)\). Conditions for the formation of salients and references are given in table 13.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Comments</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>(l/y &lt; 1.0)</td>
<td>No tombolo</td>
<td>SPM (1984)</td>
</tr>
<tr>
<td>(l/y &lt; 0.4)</td>
<td>Salient</td>
<td>Gourlay (1981)</td>
</tr>
<tr>
<td>(l/y = 0.5)</td>
<td>Salient</td>
<td>Daily and Pope (1985)</td>
</tr>
<tr>
<td>(l/y &lt; 1.0)</td>
<td>No tombolo (single breakwater)</td>
<td>Suh and Dalrymple (1987)</td>
</tr>
<tr>
<td>(l/y &lt; 2)</td>
<td>No tombolo (multiple breakwaters)</td>
<td>Suh and Dalrymple (1987)</td>
</tr>
<tr>
<td>(l/y &lt; 1.5)</td>
<td>Well-developed salient</td>
<td>Ahrens and Cox (1990)</td>
</tr>
<tr>
<td>(l/y &lt; 0.8)</td>
<td>Subdued salient</td>
<td>Ahrens and Cox (1990)</td>
</tr>
</tbody>
</table>

**Table 13 Conditions for the formation of salients**

13.2.2 Tombolo

Sand will more likely accumulate in the structure lee and form a Tombolo (connect to breakwater) when the breakwater is close to shore, is long relative to incident wavelength, and is relative impermeable (high crest and small gaps with large sediment input), see figure 40.

![Tombolo formation](source: EM-1110-2-1617)

Tombolo formation also provides a recreational beach environment, but like mentioned before, recreation is hardly an option for this area. Tombolo formation does allow direct access to the structure for maintenance. A tombolo-detached breakwater functions like a tee-shaped groin by blocking longshore transport and promoting sediment movements offshore in rip currents through the gaps. Although some longshore transport can occur seaward of the breakwater, the interruption in the littoral system may starve downdrift beaches of their normal sediment supply, causing erosion. Because the erosion problem in that case will only be relocated, a tombolo is a not preferred shoreline response (Unless erosion further downdrift won’t have any negative consequences). Whether or not a tombolo will form is given in table 14.
13.2.3 Limited shoreline response

When the breakwaters are either too far from shore, too short relative to incident wavelength, and/or too much transmissible (too low crested or too large gaps with little sediment input), there will be very limited or no shoreline response. Conditions for minimal or no shoreline response are given in table 15.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Comments</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta y &gt; 2.0$</td>
<td>Double tombolo</td>
<td>Gourley (1981)</td>
</tr>
<tr>
<td>$\delta y &gt; 0.87$ to $1.0$</td>
<td>Tombolo (shallow water)</td>
<td>Gourley (1981)</td>
</tr>
<tr>
<td>$\delta y &gt; 2.5$</td>
<td>Periodic tombolo</td>
<td>Ahrens and Cox (1960)</td>
</tr>
<tr>
<td>$\delta y &gt; 1.5$ to $2.0$</td>
<td>Tombolo</td>
<td>Dally and Pope (1986)</td>
</tr>
<tr>
<td>$\delta y &gt; 1.5$</td>
<td>Tombolo (multiple breakwaters)</td>
<td>Dally and Pope (1986)</td>
</tr>
<tr>
<td>$\delta y &gt; 1.0$</td>
<td>Tombolo (single breakwater)</td>
<td>Sun and Dalrymple (1987)</td>
</tr>
<tr>
<td>$\delta y &gt; 2.0$</td>
<td>Tombolo (multiple breakwaters)</td>
<td>Sun and Dalrymple (1987)</td>
</tr>
</tbody>
</table>

Table 14 Conditions for the formation of tombolos

13.2.4 General shoreline response

Pope and Dean (1986) made the diagram (figure 41) which relates distance offshore, length of single breakwater segment and gap-distance between breakwater segments.
13.3 Physical processes

13.3.1 Normal morphological responses

Waves breaking at an angle to the shore produce time-averaged, longshore (littoral) currents and longshore sediment transport. Physical processes at macro-level scales in the vicinity of the breakwater for normal wave and water level conditions are as follows: The breakwater shelters the coast immediately behind the structure and adjacent areas (diffraction) from the incoming waves. Breaking wave heights are smaller in the sheltered areas. The exposed gap areas have larger breaking wave heights. The wave included, mean water level change (setup) in the exposed gap areas is larger than in the sheltered areas. Longshore variability in the wave setup produces gradients in the mean water surface. Water flows from the elevated levels in the gap area towards the lower, sheltered area to accelerate the longshore current flowing towards the sheltered area behind the structure. These gradients also change the direction of the current which is driven away from the breakwater in the region immediately downdrift of the breakwater. These two current systems (littoral and setup current) merge behind the structure to give rise to complex circulation patterns. The acceleration of the littoral current updrift causes initial erosion of the beach on the updrift side. The same occurs in the area immediately downdrift. These currents carry the eroded material towards the sheltered area, where it deposits.

13.3.2 Storm processes and response

Protection afforded by the breakwater will limit erosion of the salient during significant storms. The exposed gap area will be eroded with sediment dragged offshore during storms.

13.4 Advantages and disadvantages of breakwaters

13.4.1 Advantages of breakwaters

Nearshore breakwaters offer several advantages over other beach stabilization structures. First, if properly designed, they effectively control erosion and retain sand on a beach. Second, they reduce the opportunity for rip currents to form and thus reduce offshore sediment losses. Third, they reduce the steepness of waves in their lee and encourage landward sand transport. Fourth, they reduce wave heights along the beach.
13.4.2 Disadvantages of breakwaters

Because they are located offshore, nearshore breakwaters can be expensive to build and may require the use of temporary trestles or bargemounted construction equipment. Similarly, they may be expensive to maintain because of their offshore location. The gaps between a series of breakwaters can channel flow and sediment offshore if water levels behind the breakwaters build up as a result of wave overtopping. Relatively high offshore velocities through these gaps can scour the bottom unless riprap armouring is provided. Breakwaters can also be a total barrier to longshore sand transport unless care is taken to ensure that some wave energy is available behind them to transport sand. Thus they can totally halt the flow of sand to downdrift beaches and cause erosion there. Breakwaters can also be hazardous to bathers and swimmers if they climb on the structures or get caught in offshore flows, but because there are hardly any touristy visits as mentioned before, this won’t become a major problem that should be dealt with. Finally, breakwaters can reduce the potential for recreational surfing in the project area. This is neither a problem for the Altamira area because surfing possibilities along the entire Mexican Gulf Coast are very much limited.

13.5 Structural effects of breakwaters

13.5.1 Length of the shoreline to be protected

The Altamira coastal area that directly needs to be protected is 5500m (see paragraph 4.7.1 and graph 1). This is where the current beach is less than 100m width. Therefore a system of several breakwaters spaced along the shoreline with gaps between them must be constructed. Building a single long breakwater will not achieve the same result, but will result in the formation of a single tombolo or of two tombolos, one extending seaward from shore to each end of the breakwater. The resulting lagoon enclosed by the breakwater and tombolos is undesired, because no longshore current can start up (slowly) again behind the port-breakwaters. A desired multiple breakwater system with gaps also reduces the amount of material needed for construction.

In Altamira there is a net direction of longshore transport (zie par xx). Tombolos are unwanted because of the downdrift erosion caused by totally interrupting longshore transport. So generally, a system of multiple nearshore breakwaters is needed to protect a long reach of shoreline while still maintaining some longshore transport to minimize erosion along downdrift beaches.

13.5.2 Types of construction

Most nearshore breakwaters built for shore protection have been rubble-mound structures. This type of construction is advantageous since rubble-mound structures dissipate more incident wave energy and are relatively easy to construct in the nearshore zone. Also several patented shore protection devices that function like nearshore breakwaters have been built, mostly in sheltered waters. Some have been built of precast reinforced concrete units; others have been built of concrete blocks and sand-filled geotextile tubes and bags.

13.5.3 Crest elevation

Crest elevation determines the amount of wave energy transmitted over the top of a nearshore breakwater. High crest elevations preclude overtopping by all but the highest waves whereas low crest elevations allow frequent overtopping. Generally, low crests allow more wave energy to penetrate into the lee of the breakwater. Occasional overtopping of a nearshore breakwater by storm waves can prevent tombolo formation or remove a tombolo once it has formed. The crest height needed in this situation is calculated in paragraph 13.7.2.
13.5.4 Wave heights

Wave heights behind a nearshore breakwater can be significantly reduced. Waves in the lee of a breakwater get there by transmission through the structure if it is permeable, regenerating in the lee of the structure by overtopping waves and diffraction around the ends of the breakwater. If the breakwater’s crest elevation is high and it is impermeable, diffraction is the primary source of wave energy in the shadow zone. For a detached breakwater, waves propagate around each end of the breakwater and interact in its lee. Wave heights become smaller farther behind the breakwater.

If the incident waves are nearly monochromatic, they interact constructively or destructively behind the breakwater, depending on whether the crest and trough of the waves coming around each end are in or out of phase with each other. Thus, there are regions behind the breakwater where monochromatic waves nearly cancel each other and other areas where they reinforce each other.

As the direction of incoming waves changes, the salient in the sheltered area behind the breakwater responds by repositioning itself in the region to the structure’s lee.

A diffraction analyses should be used to determine the approximate shoreline configuration behind a breakwater. Studies indicate that if the isolines of the $K' = 0.3$ diffraction coefficients are constructed from each end of the breakwater for a range of incident wave directions and they intersect seaward of the postproject shoreline, a tombolo will not form (see figure 42), (see Walker et al. 1980).

![Figure 42 Estimate of post project shoreline behind a detached nearshore breakwater, isolines of diffraction coefficient, $K'=0.3$](image)

More simply, this is ensured if the breakwater lies more than one half the breakwater’s length seaward of the postproject shoreline, i.e. after placement of beach fill. Waves coming around each end of the breakwater meet each other before the undiffracted incident wave (outside of the breakwater’s shadow) reaches the shoreline. The postproject shoreline can be estimated by drawing the pattern of the diffracted wave crests behind the breakwater and smoothing the crest pattern to balance the amount of sediment available.

Wave conditions seaward of a breakwater are determined by its reflection characteristics. Reflected waves interact with incident waves to cause a partial standing wave pattern seaward of a breakwater. Agitation of bottom sediments by standing waves can cause scour and undermining seaward of the breakwater and contribute to other foundation problems. Reflection
characteristics are in turn determined by breakwater permeability, crest elevation, and type of construction. Permeable, low-crested, rubble-mound breakwaters are the least reflective structures; however, they can allow significant amounts of energy to propagate through them. rubble-mound structures dissipate wave energy by inducing fluid turbulence in their interstices.

13.5.5 Circulation and modification of currents

Construction of offshore breakwaters will result in significant changes in the nearshore current system. On a natural beach, shore-parallel longshore currents are generated by waves approaching the shoreline at an angle. If breakwaters are built, the driving force for the currents is intercepted by the breakwater along part of the shoreline. The prevailing longshore current, unless maintained by its inertia, will slow or stop when it moves into the sheltered area behind the breakwaters. The sand carrying capacity of the current and the wave agitation that suspends sediment so it can be carried by the current are reduced. A breakwater’s length and distance from shore are critical in determining its effect on longshore currents and sediment transport. A long breakwater will cause the longshore current to slow and spread laterally and will shelter a long reach of shoreline from wave agitation.

If the breakwater crest elevation is low enough to allow overtopping, water carried over the breakwater will raise the water level behind it and cause flow around the breakwater. In multiple breakwater systems, overtopping causes a net seaward flow of water through the gaps. Return currents can be reduced by raising the breakwater crest elevation, enlarging the gaps between segments, or increasing structure permeability. For permeable breakwaters, some flow is also carried seaward through the breakwater itself.

13.5.6 Effect on longshore transport

For breakwaters where only salients develop, longshore transport rates can be adjusted to meet desired design objectives. Sediment budget analyses should be made to determine the effect of a transport rate reduction on both updrift and downdrift beaches under postproject conditions. Adjusting the length, distance offshore and crest elevation of a single breakwater will vary the resulting longshore transport rate. For multiple breakwater systems, gap width may also be modified.

13.5.7 Effect on onshore-offshore transport

Nearshore breakwaters can reduce offshore sand transport. Wave heights in a breakwater’s lee are reduced, and their direction is changed. Lower wave heights result in waves with a lower wave steepness (wave-height-to-wavelength ratio) and are therefore more likely to transport sand onshore than offshore. For multiple breakwater systems, offshore sand losses are reduced; however, overtopping can result in a net seaward flow of water and sand through the gaps between breakwater segments. These currents usually occur when the structure is nearly impermeable and low crested so that the water transmitted by overtopping can return only through the gaps or around the end of the structure. The breakwater can also reduce onshore sediment movement. Following breakwater construction, a new equilibrium between onshore and offshore transport will be established.

13.5.8 Modelling

The hydrodynamics around a low crested structure are very complex due to the coexistence of the high number of wave-driven mechanisms already mentioned. An example overview is given in figure 43.
Figure 43 Sediment fluxes for an alongshore uniform beach with an emerged low crested structure

These lead to circulation and sediment transport patterns. Simulations can be performed to evaluate the effect of different wave conditions on the resulting water/sediment fluxes. Example output of such a simulation could be figure 44.

Figure 44 Example output of sediment transport simulation (source: www.citg.tudelft.nl)
13.6 Construction of breakwaters

13.6.1 Introduction

Many investigations have been going on about the construction of breakwaters, resulting in many theories. Different locations have so many different parameters that it appears to be impossible to come up with a step to step design process of offshore breakwaters. Following certain methods, two offshore breakwater systems will be designed:

13.6.2 JMC Method

Rosati and Truitt (1990) have summarized a procedure developed by the Japanese Ministry of Construction (JMC) for the design of a system of nearshore breakwaters. The procedure, developed from observations of the performance of a number of Japanese prototype breakwaters, results in a system of relatively short breakwaters located close to shore. Beach nourishment was not included in most of the prototype projects on which the procedure is based. Because beach nourishment in Altamira will be necessary when offshore breakwaters are going to be built, the final results have to be reconsidered afterwards.

In the JMC Method sufficient data were available only for two types of coastline, type B and type C. Type B is for beaches with well-developed offshore bars, gentle slopes (1:30), moderate weight heights, and mostly shore-normal incident waves. Type C is for relatively steep slopes (1:15), no offshore bar, moderate wave heights, and beaches of coarse sand and pebbles. Both of these beach types do not completely match the situation in Altamira. This coastal area has no or small offshore bars and slopes are even gentler (1:40). Also the sand is more fine than coarse sand to pebbles. Further design will made for beach type B as that suits the Altamira situation most.

The design wave used in the procedure is the average deepwater height of the five highest “nonstorm” waves occurring in a year, \( H_{o5} \), and the wave period associated with that wave height, \( T_5 \). For the Altamira situation, a wave height has to be estimated from the available data: The \( H_{o5} \) is certainly less than the 1-year wave height (the wave height equalled or exceeded at least once every year) but higher than the average daily wave height. For the present problem, the \( H_{o5} \) wave height will be selected as the average of the 1-year wave height and the annual average wave height as given in par 4.6.1; thus,

\[
H_{o5} = \frac{(0.75 + 4.5)}{2} = 2.62m \quad \text{and} \quad T_5 = \frac{(6.0 + 8.0)}{2} = 7.0s.
\]

The deepwater wavelength is given by:

\[
L_{o5} = \frac{gT_5^2}{2\pi} = \frac{(9.81)(7.0)^2}{2(3.14)} = 76.5m
\]

and the deepwater steepness is given by:

\[
\frac{H_{o5}}{L_{o5}} = \frac{2.62}{76.5} = 0.034
\]

After selecting the length of the shoreline reach to be protected (5500m, according to par 13.5.1) and the desired shoreline advancement (salient length, \( y_s \), assumed as 20m), the breaking water depth \( d_{b5} \), of the \( H_{o5} \) wave is calculated using figure 45 with the deepwater values of \( H_{o5} \) and \( L_{o5} \) (the deepwater wavelength associated with \( T_5 \)).
Figure 45 Deepwater wave steepness as a function of nearshore steepness for various beach slopes (source: Goda 1970)

\[ \frac{d_{b5}}{H_{05}} = 1.55 \] and furthermore \( d_{b5} = 4.07 \)

Calculate the ratio \( d'/d_{b5} \), where \( d' \) is the water depth at the offshore breakwater initially estimated using \( d' = \left( d_{b5} + y_s \tan \beta \right)/2 = 2.29m \) where \( \tan \beta \) is the bottom slope (1:40).

With the ratio \( d'/d_{b5} = 2.29/4.07 = 0.56 \), the salient area ratio (SAR) can be found from figure 46 and for this case \( SAR = 0.42 \).

Figure 46 Salient area ration as a function of relative water depth for Type B shoreline

The SAR is given by: \( SAR = \frac{1}{2} \left( \frac{\ell_c y_s}{y \ell} \right) \) in which \( \ell_c \) is the salient length in the longshore direction measured at the original shoreline.

The first approximation of the structure’s distance offshore is given by \( y' = d'/\tan \beta \). The first approximation of the salient extension is then given by \( y_s' = SARy' \). If this value of \( y_s' \) is approximately equal to the value of \( y_s \) originally assumed, the value is adopted. If there is a significant difference, a new estimate of \( y_s' \) is made, and the above procedures are repeated until the two values are approximately equal.
In this first approximation, the structure’s distance offshore is $y' = 91.4m$ and thereby belongs a salient extension of $y'_s = 38.4m$. This is not very close to the desired shoreline advancement; therefore numerous steps above have to be repeated with a second estimate of structure depth, $d'$. 

If for $d'$ the value 1.60 is chosen, the equation will be: $d'/d_h = 1.60 / 4.07 = 0.39$. Then the SAR=0.3. Repeating the steps above now the structure’s distance offshore is $y' = 64.0m$ and there will be a salient extension of $y'_s = 19.2m$. This is more like the desired salient length.

Now calculate the wavelength at the structure: $L_s = T_s \sqrt{gd} = 7.0 \sqrt{9.81(1.6)} = 27.7m$

The range of structure lengths as a function of nearshore wavelength for beach Type B is given by $1.8L_s < \ell < 3.0L_s$ or $49.9m < \ell < 83.1m$. The range of structure lengths as a function of offshore distance for beach Type B is given by $0.8y' < \ell < 2.5y'$ or $51.2m < \ell < 160m$. Applying those two equations give two ranges for the breakwater length. The breakwater length adopted is the average of the highest minimum and the lowest maximum of the two ranges. So the range will be $51.2m < \ell < 83.1m$ and the structure length is calculated as $\ell = \frac{51.2 + 83.1}{2} = 67.2m$.

If the length of the shoreline to be protected exceeds twice the breakwater length (and it does with a coastline protection of 5500m), the gap width can be selected by using the following ranges of gap width that is valid for Type B beaches, $0.7y' < b < 1.8y'$ or $44.8m < b < 115.2m$ and $0.5L_s < b < 1.0L_s$ or $13.9 < b < 27.7$. These two ranges are mutually exclusive; however, an estimate of the gap width is again the average of the minimum maximum, i.e. 27.7m, and the maximum minimum, i.e. 44.8m. Thus, $b = (27.7 + 44.8) / 2 = 36.3m$.

The major sizes of the breakwaters are now set and are given in overview in figure 47.

**Figure 47 Altamira project parameters as designed with JMC method (source: Rosati and Truitt)**

According to the diagram made by Pope and Dean (1986, figure 41), the desired salients will appear.

The calculated breakwater length, gap width, distance from shore and SAR can then be used to develop a final breakwater system design subject to subsequent evaluation using analytical tools such as computer simulations, etc.

Dealing with this JMC method of breakwater design, it’s limitations become clear: Beach fill is not included as a part of the design method; guidance is based on data from sites at which tombolo formation occurred for the majority of cases; wave conditions required for design are the wave height and period corresponding to the “average of the largest five nonstorm waves occurring in a typical year”, and the effects of structural transmissibility and water-level variations are not parameters in the method.
13.6.3 Dimensional analysis for detached breakwaters

Dimensional analyses can provide some insight into the design of single and multiple detached breakwater systems.

\[ \pi_1 = \frac{\ell}{gT^2} \] = Dimensionless breakwater length. This variable can be taken as a scaling factor that can be used to transpose observations of breakwater performance from one location to another. For instance the Altamira situation will be compared with the situation in Holly Beach, Louisiana, USA (also located on the Gulf of Mexico). In Holly Beach segmented breakwaters are placed in 1985 over a length of 555m. Length of segments is 46.50m, gap width 93.89m and distance offshore is 78.61m. Average wave period there is 5.5s, the water depth at the breakwater is 2.5m and salients where formed behind the breakwaters (source: EM 1110-2-1617). So for Holly Beach \( \pi_1 = \frac{46.50}{9.81(5.5)^2} = 0.16 \). Translating this result into the breakwater segment length for Altamira will give \( \ell = \pi_1 gT^2 = (0.16)(9.81)(7.0)^2 = 75.3 \text{m} \).

\[ \pi_2 = \frac{y}{\ell} \] = Dimensionless distance of the detached breakwater from shoreline. For Holly Beach this is \( \pi_2 = \frac{78.61}{46.50} = 1.69 \) so it would be \( y = \pi_2 \ell = (1.69)(75.3) = 127.3 \text{m} \) for Altamira. At this point 127.3m offshore the water depth is 3.0m (derived from graph 4 and table 4 in paragraph 4.7.3) and then according to Pope and Dean (figure 41) with \( \frac{y}{d} = \frac{127.3}{3.0} = 42.4 \) en \( \frac{\ell}{y} = \frac{75.3}{127.3} = 0.59 \), either very small salients will appear or there will be no sinuosity at all.
Though, according to Dally and Pope (1986), there will form salients behind the breakwaters (see table 13).

- \( \pi 3 = \frac{y}{y'} \) = Dimensionless salient length that takes on values between 0 and 1, \( \pi 3 = 1.0 \) for a tombolo. No information is available about salient length in Holly Beach, but regarding the expected shoreline response in Altamira, this value will lie between 0 and 0.5.

- \( \pi 4 = \frac{H}{d} \) = Dimensionless breaking wave height that also determines if the breakwater is inside or outside the surf zone. If \( \pi 4 \) is less than 0.78, the breaker line will be landward of the breakwater. For \( \pi 4 \) greater than 0.78, the breaker line will be seaward of the breakwater; i.e. waves will break before they reach the breakwater and the breakwater will be in the surf zone.

If our design wave is \( H_b = 2.0m \) (assumed in case of a small storm), then
\[ \pi 4 = \frac{H}{d} = \frac{2.0}{3.0} = 0.67 \] and apparently the wave breaker line will be landward of the breakwater and much wave energy will be dissipated by the breakwaters.

- \( \pi 5 = \frac{d}{d_b} \) = Dimensionless water depth at the breakwater. If \( \pi 5 < 1.0 \), the breakwater lies within the surf zone, and waves break seaward of the breakwater. If \( \pi 5 > 1.0 \), the breakwater is seaward of the surf zone, and waves break landward of the breakwater. The product of \( \pi 4 \) and \( \pi 5 \) is the breaking wave height to breaking depth-ratio and is usually about 0.78, although there is some dependence of this ratio on beach slope.

As the breaking depth of the characteristic design wave \( (d_b) \) is unknown for the Altamira situation, the exact value of \( \pi 5 \) cannot be determined. But because it is known that the breakwater will lie outside the breakwater zone, the value of \( \pi 5 \) will be just over 1.0.

- \( \pi 6 = \frac{b}{\ell + b} \) = Dimensionless "exposure ratio". It represents the fraction of the shoreline exposed to waves propagating through the breakwater gaps. Values of \( \pi 6 \) greater than 0.5 indicate relatively large gaps with gaps that are longer than the breakwaters. Values of \( \pi 6 \) less than 0.5 are more typical of prototype installations as indicated in Table 16, which gives "exposure ratios" for several prototype breakwater installations in the USA.

<table>
<thead>
<tr>
<th>Project</th>
<th>Exposure Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Winthrop Beach, MA</td>
<td>0.25</td>
</tr>
<tr>
<td>Lakeview Park, Lorain, OH</td>
<td>0.36</td>
</tr>
<tr>
<td>Castlewood Park, Colonial Beach, VA</td>
<td>0.31 to 0.38</td>
</tr>
<tr>
<td>Central Beach, Colonial Beach, VA</td>
<td>0.39 to 0.45</td>
</tr>
<tr>
<td>East Harbor State Park, OH</td>
<td>0.56</td>
</tr>
<tr>
<td>Presque Isle, Erie, PA (experimental prototype)</td>
<td>0.56 to 0.66</td>
</tr>
<tr>
<td>(hydraulic model)</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Table 16 "exposure ratios" for various prototype multiple breakwater projects in the USA

When the distance between breakwater segments \( (b) \) is determined on the same way as the length of the breakwater by comparing with the Holly Beach situation:
\[ \pi 1 = \frac{b}{gT^2} = \frac{93.89}{9.81(5.5)^2} = 0.32 \quad \text{and} \quad b = \pi 1gT^2 = (0.32)(9.81)(7.0)^2 = 152.0m, \]
"exposure ratio" in the Altamira situation is \[ b = \frac{152.0}{75.3 + 152.0} = 0.67 \], this is quite high compared with the USA projects listed in Table 16. Alternatively, \[ \pi 6' = \frac{\ell}{\ell + b} \] is a "sheltering ratio" that represents the fraction of the shoreline sheltered from incoming waves by the breakwaters. \( \pi 6 \) and \( \pi 6' \) are related by the expression \( (\pi 6 + \pi 6') = 1.0 \) and are thus not independent of each other.

Following this comparison method, a breakwater system should be built with dimensions as given in figure 50.

```
Figure 50 Altamira project parameters as designed in comparison with the Holly Beach segmented breakwater system
```

13.6.4 Conclusions of these design methods

These two methods result in the rough dimensions of the breakwaters. First design, according to the JMC-method, resulted in breakwater segments 67.2m long, 36.3m offshore and gap widths of 64.0m. To cover the stretch of vulnerable coastline (5500m), 43 breakwater segments are needed.

The second design is the comparison with the Holly Beach offshore breakwaters and this resulted in segments 75.3m long, 127.3m offshore and gaps between two segments should be 152.0m. According to these dimensions, 25 breakwater segments are needed to cover the coastline.

To choose between these two methods, at first the costs will be considered. The JMC method requires breakwater segments for a total length of 2900m at a depth of 1.6m and they need to be constructed 64m offshore. The comparison method requires breakwater segments for a total length of 1900m at a depth of 3.0m and they need to be constructed 127.3m offshore. It is easy to see that despite a longer construction length, the construction of breakwaters according to the JMC method is the cheapest option. Depth at place of construction and distance offshore make the second method the more expensive one.

On the other hand it is important which breakwater system will function most appropriate. The first method is based on experience with breakwaters in and around Japan. Also distance offshore is small so the desired salients can change in either tombolos or not any shoreline reaction very easy, regarding unforeseen things or not correctly assumed parameters. Therefore the segmented breakwater system developed in comparison with the Holly Beach breakwater system (Holly Beach is also located on the Gulf of Mexico) can likely handle more changes of unforeseen conditions, while still maintaining the desired salients behind the breakwaters.

Considering the importance of the functional working of the breakwater, the second designed breakwater should be worked out further more.
13.7 More detailed design of breakwater system

13.7.1 Use of stone and way of building

In case a breakwater has the function to establish an area of calm sea where boats can be moored safely during rough weather, it has to be built with a strong core so that wave transmission is (almost) blocked. That results in a multilayer breakwater to protect the relatively light core. It is easy to see that this way of construction is expensive compared with a breakwater that would only exist of one layer. Because (reduced) wave transmission is even a desired effect in this case, the breakwater will be built from one stone-size only. An approximation of the stone-size will be derived in 13.7.2. The one big layer should be placed stone by stone in a sequence that ensures interlocking. The slope should be 3:1 to ensure stability. Any defects in the quality of the rock, in its grading (such as using rocks that are too small) or its placing (on a slope that is uneven or too steep) will seriously put the whole breakwater at risk.

The most reliable source of stone rubble for construction is the quarry. A quarry is usually worked for a whole range of sizes of stone and the yield of the right sizes depends on the capability of the person carrying out the blasting as well as on the geological composition of the ground.

For the actual building of the construction are two options available.
- In situ building using an access way
- In situ building using pontoons

For the first option it must be noticed that for each single breakwater an access road has to be built that can hold heavy loaded trucks. Every access road means stabilizing part of the (nourished) beach, building the access way 127m long up to more than 3m of height and finally the core of the breakwater should be accessible for these heavy loaded trucks. This demands an extra treatment of the core and possibly it is not feasible anymore to stay with the one layer breakwater. If so there must be switched to the more expensive multilayer breakwater that could also have consequences for the wave transmission. If more layers applied, the wave transmission has to be reconsidered.

For the second option the needed materials should be loaded on pontoons with low draught. Also necessary are floating cranes. Neither of them should exceed a draught of 3m, or it will get stuck.

As with most things, the final price tag will be the deciding factor whether to choose one alternative or another. Although building offshore without any direct access to the shore is usually expensive, it still is by far the cheapest option in this situation. This because each access road only holds almost as much cubic metres of rock as each breakwater segment itself. Besides, the access road has to be removed afterwards which will bring costs with it as well (roughly 25% of the initial placement costs). It is easy to see that renting pontoons instead will be far cheaper and so the far better option. Further costs will be worked out in paragraph 13.7.4.

13.7.2 Wave diffraction and wave transmission (permeability)

As mentioned in paragraph 13.5.4, a diffraction coefficient \( K' = 0.3 \) is desired to prevent the construction from tombolo forming. The diffraction coefficient is defined as the ratio of the diffracted wave height to the height of the incident wave which is not disturbed by the breakwater. As the incident wave height is settled as 0.75m, corresponding 0.23m high waves will appear in the lee of the breakwater because of this diffraction.

Also the transmission coefficient \( tK \) is defined as the ratio of the transmitted wave height to the incoming significant wave height. Note that the transmission coefficient can never be smaller than 0 or larger than 1. In practice, limits of about 0.1 and 0.9 are found (figure 52). Also note that small values of \( tK \) (close to 0) almost block the wave energy completely, so a tombolo can
form easily (as mentioned before). On the other hand, in case of large values of $K_r$ (close to 1), the detached breakwater will not/hardly function and the wave energy will all reach the shoreline.

![Diagram of Wave Transmission for Low Crested Structures](image)

**Figure 51: Wave transmission for low crested structures**

The desired diffraction coefficient is settled at 0.3 and the desired transmission coefficient will also be settled at 0.3. Derived from figure 52 it can be said that $R_c/H_s = 0.5$ in which $R_c$ = the crest freeboard relative to still-water level (SWL = the elevation that the surface of the water would assume if all wave action were absent) in m. $H_s$ = the significant wave height (0.75m).

From this equation it appears that $R_c = 0.38$, so the crest of the breakwater should be elevated 0.38m above SWL. Daemen (1991) was able to produce an acceptable formula that relates the transmission coefficient to a number of structural parameters of the breakwater:

$$K_t = a \frac{R_c}{D_{n50}} + b$$

with

$$a = 0.031 \frac{H_i}{D_{n50}} - 0.24$$

$$b = -5.42 s_{op} + 0.0323 \frac{H_i}{D_{n50}} - 0.0017 \left( \frac{B}{D_{n50}} \right)^{1.84} + 0.51$$

In this case this formula will be used to determine the necessary diameter of the armour stone. That this should be done iteratively, becomes clear if the equation is written at once:

$$K_t = 0.031 \frac{H_i R_c}{D_{n50}} - 0.24 \frac{R_c}{D_{n50}} - 5.42 s_{op} + 0.0323 \frac{H_i}{D_{n50}} - 0.0017 \left( \frac{B}{D_{n50}} \right)^{1.84} + 0.51$$

Where $D_{n50}$ is the only unknown variable as we take crest width $B = 2.0m$. 
- For D is 1000mm K is 0.32.
- For D is 900mm K is 0.314.
- For D is 800mm K is 0.306.
- For D is 700mm K is 0.296.
- For D is 750mm K is 0.3014.

From this iterative approach it appears that $D_{n50} = 750\text{mm}$.

The crest width is usually taken around $3xD_{n50}$ and in this case the earlier assumed $B = 2.0\text{m}$ does pretty much match $3xD_{n50}$.

13.7.3 Trial and error

As mentioned before, above results can form a base in a further design stage. Different kinds of models can be used to model the situation after placing the breakwaters. Because of possible wrong assumptions, not accurate information or other factors that could make model-results unreliable, the use of these models could be discussed.

A very common way of placing detached breakwaters is trial and error. It means that after all the possible calculations are done, the detached breakwaters will be placed. After placement there will be kept a close eye on what happens. Obviously, when everything goes as desired, nothing has to be done furthermore. Else, adjustments should be made (like adding or removing a layer of stone, extend or shorten single or multiple breakwaters) and again these breakwaters should be monitored.

13.7.4 Costs

When quantifying the amount of needed rock, an easy calculation can be made from figure 52.

![Diagram](image)

Figure 52 Profile of detached breakwater

Roughly \(2 \times \left( \frac{1}{2} \times 3.38^2 \times 3 + 3.38 \times 2 \right) \times 75.3 = 3090\text{m}^3\). Slopes at both ends are not in this calculation, but it is reasonable to say that $3200\text{m}^3$ of rock material including porosity is needed for each breakwater segment. So for 25 breakwater segments in total $80000\text{m}^3$ of stones is needed. Translating this into a price, take in account a porosity of 40% and a stone density of $2650\text{kg/m}^3$. That means that the real amount of stones needed is $48000\text{m}^3$ or $127,2 \times 10^6 \text{kg}$. Stones can be delivered in the port area from a quarry 165km inland of Altamira.
at a rate of $16.5 USD per ton. So only the material and its delivery will cost about $2.1 million USD.
Like mentioned before, offshore construction needs the use of a pontoon. General costs to hire a 2400T pontoon are given in paragraph 12.13.2.
The 25 breakwater segments to protect the 5500m shoreline will take approximately one year to build. The rent of the pontoon will cost $410,000 USD for a period of one year.
The rent of the crane itself will cost about the same as the rent of the pontoon for this one-year period. The mobilization costs of the crane will be less, but the actual working of the crane will be more expensive because its operation requires more man-hours. Altogether the total construction costs of the complete system of segmented breakwaters will be estimated at just a little less than $3 million USD.
Further costs of maintenance, monitoring and possible adjustments in a further stadium are not (yet) mentioned.
14 Evaluating the Alternatives

14.1 Introduction

The creation of a stable shore is essential for the project. The 50 meters dry beach in front of the littoral barrier will ensure this requirement and has to be accomplished by initial beach nourishment. After the new beach is created one of the alternatives must prevent this beach from eroding again.

The alternatives are evaluated on the following criteria:

Solving the problem
This is obviously the main goal and therefore it is given the maximum weighing factor of 5. All alternatives give a solution for the direct erosion problem, but with the possible hard solutions the erosion problem will be relocated downdrift of the constructions. Therefore the soft solutions will get the better scores at this point.

Coastal dynamics
Some alternatives influence the coastal dynamics more than others. Because these are only side-effects, the weighing factor is settled at 2. With the hard solutions there will appear different water/sediment-fluxes that are hard to quantify and its consequences are hard to qualify. For this reason the alternative that influences coastal dynamics the least will get the better score.

Flexibility
All possible solutions are designed with the use of certain assumptions and possible inaccuracy. Therefore it is preferable that constructions are flexible to adapt when they malfunction or processes differ from prognoses; this weighing factor is settled at 3.

Maintenance
Some alternatives are more labour-intensive for maintenance than others. Because maintenance has a direct link with the costs, its weighing factor is settled at 1.

Monitoring
Monitoring of the shoreline evaluation is essential for the alternatives, but because it is only labour intensive and also directly linked to the costs, it is given a weighing factor of 1 as well.

Environment
The environment is an important issue for this project. The coastal zone is dedicated to become a natural area for birds and therefore needs to have a stable coast strip. It is also preferable to create a quite beach for turtle breeding. The weighing factor is settled at 3.

Construction Costs
The construction costs of the solution are obviously very important, since it is a commercial port with a limited budget and has many more things to invest in. Therefore the weighing factor of the costs is settled at 4.
### 14.2 Multi Criteria Analyze

The multi criteria analyze gives a quantification of the suitability of the various alternatives. All the advantages and disadvantages are quantified with a mark between -2 and 2. Negative marks imply a disadvantage, while positive marks an advantage. Because not all criteria have the same importance, they all have a weighing factor (1 to 5), which has to be multiplied by the mark to get the final weigh. The sum of the final weighs gives an indication of the overall suitability of the alternative.

The next table gives an overview of the Multi Criteria Analyze.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>weigh factor</td>
<td>mark</td>
<td>tot. weigh</td>
<td>mark</td>
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<tr>
<td>Solving the problem</td>
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<td>2</td>
<td>10</td>
<td>2</td>
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<td>Coastal dynamics</td>
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<td>2</td>
<td>2</td>
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<td>Flexibility</td>
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<td>2</td>
<td>6</td>
<td>-1</td>
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<tr>
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<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Monitoring</td>
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<td>2</td>
<td>6</td>
<td>1</td>
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<tr>
<td>Environment</td>
<td>4</td>
<td>-2</td>
<td>-8</td>
<td>1</td>
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<tr>
<td>Construction Costs</td>
<td>15</td>
<td>18</td>
<td>3</td>
<td>11</td>
</tr>
</tbody>
</table>

The various alternatives have the next abbreviation:
- Maintenance Beach Nourishment: MBN
- Bypass System: Byp.
- Groins: Gro
- Detached Breakwaters: DeBr

#### Explanation of the given marks

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Mark</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solving the Problem</td>
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<td></td>
</tr>
<tr>
<td>MBN</td>
<td>2</td>
<td>Solves problem without moving it to other areas</td>
</tr>
<tr>
<td>Byp</td>
<td>2</td>
<td>Solves problem without moving it to other areas</td>
</tr>
<tr>
<td>Gro</td>
<td>1</td>
<td>Moves problem south of groin area</td>
</tr>
<tr>
<td>DeBr</td>
<td>1</td>
<td>Moves problem south of detached breakwater area</td>
</tr>
</tbody>
</table>

| Coastal Dynamics |
| MBN            | 1    | No predicted changes, because the layout is hardly changed |
| Byp            | 2    | No predicted changes, because the layout is not changed |
| Gro            | -1   | Moves sea currents outside groin zone and creates rip currents |
| DeBr           | -1   | Moves sea currents outside det. bre. zone and creates rip currents |

| Flexibility |
| MBN         | 2    | Just adjusting amounts, but costs are adjusted with it |
| Byp         | -1   | Flexible until maximum capacity |
| Gro         | 0    | Replacing construction parts |
| DeBr        | 1    | Replacing construction parts of coast |

<p>| Maintenance |
| MBN         | 0    | Periodically re-nourishment |</p>
<table>
<thead>
<tr>
<th>Category</th>
<th>Byp</th>
<th>Gro</th>
<th>DeBr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maintenance</td>
<td>-1</td>
<td>1</td>
<td>1</td>
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<tr>
<td>Monitoring</td>
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<tr>
<td>Gro</td>
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<td></td>
</tr>
<tr>
<td>DeBr</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Monitoring**
- MBN: Regular monitoring to predict re-nourishment time
- Byp: Monitoring profile for bypass quantity
- Gro: Monitoring function and downdrift erosion
- DeBr: Monitoring function and downdrift erosion

**Environment**
- MBN: Creation of extra beach and secure barrier
- Byp: Maintaining beach and secure barrier
- Gro: Big extra beach and secure barrier
- DeBr: Small extra beach and secure barrier

**Construction Costs**
- MBN: $17.2 million USD
- Byp: $5 million USD (supposing the submarine sand shifter)
- Gro: $10 million USD (extra construction costs for access from land)
- DeBr: $3 million USD
15 Conclusion

A conclusion is very difficult to make after the investigation. As already mentioned in the report several times, the design process of the different alternatives is very rough and inaccurate. For a final design of the alternatives numerical and physical models are needed. With some guidelines and assumptions there is made a rough estimation of the lay out of the different alternatives. These estimated lay outs of the possible solutions are compared with each other looking at different criteria.

From this evaluation the conclusion can be made that the by pass system is the best solution for the erosion problem at the downdrift side of the Port of Altamira. This is based on our report and investigation. The bypass system is a good natural solution which restores the original sediment transport. Besides that the costs of this system are low. These two main features has caused the by pass system as the final solution for the erosion problem. For final dimensioning of this system further investigation is required.
16 Recommendations

After the completion of this report some recommendations can be made for further investigation in a solution to the erosion problem of the Port of Altamira.

Not a lot of wave information was available and some extra monitoring of the wave action is needed in this area. That way all wave conditions can be taken into account. The alongshore sediment transport in this area is estimated by looking at the erosion place. For determining the exact amount of sediment transport models have be used to include all the water forces.

Because there was no model availability a lot of assumptions had to be made. We also did estimation about the wanted minimum dry beach. For the designing process for the alternatives a numerical model or physical model is needed. These models can describe the exact influence of the alternatives on the coast. With the wanted situation, the dimension of the construction can be made. In this report a rough estimation of the dimensions is calculated with some guidelines from several information sources.

After determining the best layout for the construction, it is useful as well to use a physical model. Therefore it would be wise to test the final design in a hydraulic laboratory and to see in a scale model how the coast will react.
Reference Projects

Reference projects sand bypass:

- The port of Coega (ngqura)
  South Africa
  http://www.ports.co.za/coega.php

- Tweed river entrance sand bypassing project
  Queensland, Australia

- Queensland, Australia

- Slurry systems marine

Reference projects detached shore parallel breakwaters:

- Kertih marine facilities:
  http://mcleon.tripod.com/KertehBreakwater.pdf

- The Giardini-Naxos Bay
  Sicily, Italy
# List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tbody>
<tr>
<td>$H_s$</td>
<td>Significant wave height</td>
<td>(m)</td>
</tr>
<tr>
<td>$T_z$</td>
<td>Wave period</td>
<td>(s)</td>
</tr>
<tr>
<td>$T_0$</td>
<td>Deep water wave period</td>
<td>(s)</td>
</tr>
<tr>
<td>$H_0$</td>
<td>Deep water wave height</td>
<td>(m)</td>
</tr>
<tr>
<td>$\phi_0$</td>
<td>Deep water wave angle</td>
<td>(°)</td>
</tr>
<tr>
<td>$H$</td>
<td>Average wave height</td>
<td>(m)</td>
</tr>
<tr>
<td>$L$</td>
<td>Average wave length</td>
<td>(m)</td>
</tr>
<tr>
<td>$h$</td>
<td>Water depth</td>
<td>(m)</td>
</tr>
<tr>
<td>$H_b$</td>
<td>Wave height at breaker line</td>
<td>(m)</td>
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<tr>
<td>$h_b$</td>
<td>Wave depth at breaker line</td>
<td>(m)</td>
</tr>
<tr>
<td>$S$</td>
<td>Sediment transport</td>
<td>(m³/s/m²)</td>
</tr>
<tr>
<td>$y$</td>
<td>Breaker index</td>
<td>(-)</td>
</tr>
<tr>
<td>$\alpha_b$</td>
<td>Wave angle at breaker line</td>
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<tr>
<td>$x_b$</td>
<td>Distance waterline to breaker line</td>
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<tr>
<td>$m$</td>
<td>Beach slope steepness</td>
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<tr>
<td>$\eta_d$</td>
<td>Wave set down</td>
<td>(m)</td>
</tr>
<tr>
<td>$\eta_u$</td>
<td>Wave set up</td>
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</tr>
<tr>
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<td>Friction coefficient</td>
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</tr>
<tr>
<td>$\rho_{air}$</td>
<td>Density of Air</td>
<td>(kg/m³)</td>
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<tr>
<td>$\rho_{water}$</td>
<td>Density of water</td>
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<tr>
<td>$g$</td>
<td>Fall acceleration</td>
<td>(m²/s)</td>
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<tr>
<td>$k$</td>
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<tr>
<td>$c_w$</td>
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<tr>
<td>$B_{supplement}$</td>
<td>Broad of supplement beach nourishments</td>
<td>(m)</td>
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<tr>
<td>$Sed_{entering}$</td>
<td>Sediment entering the nourishment area</td>
<td>(m³/yr)</td>
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<tr>
<td>$Sed_{leaving}$</td>
<td>Sediment leaving the nourishment area</td>
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<tr>
<td>$\phi$</td>
<td>Natural angle of repose</td>
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<tr>
<td>$l_n$</td>
<td>Length of groin</td>
<td>(m)</td>
</tr>
<tr>
<td>$s_n$</td>
<td>Spacing of groin field</td>
<td>(m)</td>
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<td>$R_{sl}$</td>
<td>Ratio spacing / length groin</td>
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<td>$P$</td>
<td>Notional permeability according vd Meer</td>
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<tr>
<td>$\tan \alpha$</td>
<td>Slope angle groin</td>
<td>(-)</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Unit</td>
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<tr>
<td>--------</td>
<td>--------------------------------------------------</td>
<td>-------</td>
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<td>$S_d$</td>
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<td>Nominal grain diameter</td>
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<tr>
<td>$N$</td>
<td>Number of incident waves</td>
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<tr>
<td>$\xi$</td>
<td>Iribarren number</td>
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<td>$\xi_{mc}$</td>
<td>Critical Iribarren number</td>
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<tr>
<td>$W_{50}$</td>
<td>Mass of rock</td>
<td>(kg)</td>
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<tr>
<td>$l$</td>
<td>Length of detached breakwater</td>
<td>(m)</td>
</tr>
<tr>
<td>$y$</td>
<td>Distance offshore detached breakwater</td>
<td>(m)</td>
</tr>
<tr>
<td>$K'$</td>
<td>Diffraction coefficient</td>
<td>(-)</td>
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<tr>
<td>$H_{o5}$</td>
<td>Deepwater wave height of five highest waves / yr</td>
<td>(m)</td>
</tr>
<tr>
<td>$T_{o5}$</td>
<td>Deepwater wave period of five highest waves / yr</td>
<td>(s)</td>
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<tr>
<td>$L_{o5}$</td>
<td>Deepwater wave length of five highest waves / yr</td>
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<tr>
<td>$d_{h5}$</td>
<td>Breaker depth with condition of $H_{o5}$, $T_{o5}$</td>
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<tr>
<td>$d'$</td>
<td>Estimated depth at detached breakwater</td>
<td>(m)</td>
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<tr>
<td>$\tan \beta$</td>
<td>Bottom slope</td>
<td>(-)</td>
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<tr>
<td>$SAR$</td>
<td>Salient area ratio</td>
<td>(-)</td>
</tr>
<tr>
<td>$l_c$</td>
<td>Salient length alongshore direction</td>
<td>(m)</td>
</tr>
<tr>
<td>$y_s$</td>
<td>Salient length offshore direction</td>
<td>(m)</td>
</tr>
<tr>
<td>$y'$</td>
<td>First approximation of $y$</td>
<td>(m)</td>
</tr>
<tr>
<td>$y_s'$</td>
<td>First approximation of $y_s$</td>
<td>(m)</td>
</tr>
<tr>
<td>$b$</td>
<td>Gap width between detached breakwaters</td>
<td>(m)</td>
</tr>
<tr>
<td>$z$</td>
<td>Tidal range</td>
<td>(m)</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Salient area</td>
<td>(m$^2$)</td>
</tr>
<tr>
<td>$K_t$</td>
<td>Transmission coefficient</td>
<td>(-)</td>
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<tr>
<td>$R_c$</td>
<td>Crest freeboard relative to Still Water Level</td>
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</tr>
<tr>
<td>$H_i$</td>
<td>Incoming wave height</td>
<td>(m)</td>
</tr>
<tr>
<td>$s_{op}$</td>
<td>Wave steepness</td>
<td>(-)</td>
</tr>
</tbody>
</table>
List of Figures

Figure 1: Position of Altamira, Mexico ................................................................. 12
Figure 2: Overview of Study Area and Port of Altamira ....................................... 13
Figure 3: Future Plans of the Port of Altamira ..................................................... 14
Figure 4: Sketch of Erosion Problem ................................................................. 15
Figure 5: Dominant wave direction ................................................................. 19
Figure 6: Gulf of Mexico Current ................................................................. 20
Figure 7: Bathymetry of study area ................................................................. 21
Figure 8: Cross Shore and Long shore ............................................................. 28
Figure 9: Surf zone ....................................................................................... 28
Figure 10: Gradient in radiation stress ............................................................ 29
Figure 11: Erosion around Port ...................................................................... 30
Figure 12: Undertow ...................................................................................... 30
Figure 13: Wave setup .................................................................................. 31
Figure 14: Wind set up .................................................................................. 32
Figure 15: By pass ....................................................................................... 36
Figure 16: Seawall ....................................................................................... 37
Figure 17: Groins ......................................................................................... 37
Figure 18: Revetment ................................................................................... 38
Figure 19: Detached Breakwater .................................................................... 38
Figure 20: Erosion in front of Seawall .............................................................. 39
Figure 21: By pass system across port ............................................................... 47
Figure 22: Sand Shifter unit .......................................................................... 49
Figure 23: Schematization of the whole system .................................................. 50
Figure 24: Example of an inlet construction placed on a jetty ......................... 50
Figure 25: of the sand collecting system of the Tweed River Sand Bypassing System ........................................................................ 51
Figure 26: A semi-mobile, land-based equipped system .................................. 52
Figure 27: Pipeline lay out ............................................................................ 53
Figure 28: Example of an outlet ...................................................................... 54
Figure 29: Alignment of beach in groin bay ....................................................... 55
Figure 30: Flow pattern in groin field ............................................................. 55
Figure 31: Groin field from Port Altamira till Tampico Breakwater alternative 2 .................................................................................. 58
Figure 32: Groins in combination with by passing ........................................... 59
Figure 33: Spacing requirements ................................................................... 60
Figure 34: Estimated situation with minimum dry beach .................................. 62
Figure 35: Transition of a groin field .............................................................. 65
Figure 36: Groin profile ............................................................................... 67
Figure 37: Order of construction ................................................................. 70
Figure 38: Detailed Lay out Rubble Mound Groin ............................................ 71
Figure 39: Salient formation (source: EM-1110-2-1617) .................................. 76
Figure 40: Tombolo formation (source: EM-1110-2-1617) .............................. 77
Figure 41: Empirical guidance for offshore segmented breakwaters (source: Pope and Dean 1986) .............................................................. 79
Figure 42 Estimate of post project shoreline behind a detached nearshore breakwater, isolines of diffraction coefficient, $K^*=0.3$ ........................................................................ 81
Figure 43 Sediment fluxes for an alongshore uniform beach with an emerged low crested structure .................. 83
Figure 44 Example output of sediment transport simulation (source: www.citg.tudelft.nl) .................. 83
Figure 45 Deepwater wave steepness as a function of nearshore steepness for various beach slopes (source: Goda 1970) ........................................................................ 85
Figure 46 Salient area ration as a function of relative water depth for Type B shoreline ........................................................................ 85
Figure 47 Altamira project parameters as designed with JMC method (source: Rosati and Truitt) .................. 86
Figure 48 Profile of beach with breakwater ...................................................... 87
Figure 49 Plan of beach with breakwater ......................................................... 87
Figure 50 Altamira project parameters as designed in comparison with the Holly Beach segmented breakwater system. ................................................................. 89
Figure 51 Wave transmission for low crested structures ................................................................. 91
Figure 52 Profile of detached breakwater .................................................................................... 92
Figure 53: Wave ray and crest ......................................................................................................... 112
Figure 54: Refraction Process ......................................................................................................... 112
Figure 55: Fluid streaming out the fluidization pipe ................................................................. 119
Figure 56: Fluidization process ...................................................................................................... 119
Figure 57: Pumping away of the fluidized sediment ................................................................. 120
Figure 58: shows the trench which is formed ............................................................................. 120
Figure 59: Trench ......................................................................................................................... 120
List of Tables

Table 1: Wave Heights .......................................................................................................................... 18
Table 2: Wave Height in different seasons .......................................................................................... 18
Table 3: Tides Data ............................................................................................................................ 20
Table 4: Beach Slope .......................................................................................................................... 24
Table 5: Wave Characteristics under normal conditions ...................................................................... 26
Table 6: Wave characteristics under storm conditions ....................................................................... 27
Table 7: Advantages - Disadvantages alternatives ............................................................................ 59
Table 8: Groin length with minimum dry beach .................................................................................. 61
Table 9: Final demands length alternatives ...................................................................................... 63
Table 10: Final estimation dimensions alternatives .......................................................................... 64
Table 11: Database of groins ............................................................................................................. 65
Table 12: Transition alternatives ....................................................................................................... 66
Table 13 Conditions for the formation of salients ............................................................................ 77
Table 14 Conditions for the formation of tombolos .......................................................................... 78
Table 15 Conditions for minimal shoreline response or none ............................................................ 78
Table 16 “exposure ratios” for various prototype multiple breakwater projects in the USA ................ 88
Table 17: Grain Diameter North Side .............................................................................................. 110
Table 18: Grain Diameter South Side .............................................................................................. 111
Table 19: Wave Characteristics information normal conditions ..................................................... 114
Table 20: Wave characteristics information Storm conditions ....................................................... 115
Table 21: Calculation Sediment transport CRESS .......................................................................... 117
List of Graphs

Graph 1: Littoral Barrier ......................................................................................................................... 22
Graph 2: Height Littoral Barrier .............................................................................................................. 22
Graph 3: Beach Profile South .................................................................................................................. 23
Graph 4: Average beach profile .............................................................................................................. 24
Graph 5: Littoral barrier requirements ................................................................................................... 34
Graph 6: Closure depth ........................................................................................................................... 42
Graph 7: Initial beach nourishments ...................................................................................................... 43
Graph 8: Maintenance Beach Nourishments .......................................................................................... 46
Graph 9: Beach profile ............................................................................................................................ 68
Graph 10: Final lay out Alternative 1 and 2 ......................................................................................... 68
Graph 11: Final lay out Alternative 3 .................................................................................................... 69
Graph 12: Occurrence of Wind direction for the years 2003, 2004, 2005 ............................................ 108
Graph 13: Wind Velocity per direction for the years 2003, 2004, 2005 ............................................... 109
Sources of Information

Groins

- "Guide on the uses of Groynes in coastal engineering".
- "Handbook of Coastal engineering"
- "EM 1110-2-1617, 20 Aug ’92"
- "Manual on the use of Rock in Hydraulic engineering"

under construction
Appendices

Appendix A: Wind Data

Wind direction 2003 in %

Wind direction 2004 in %
Graph 12: Occurrence of Wind direction for the years 2003, 2004, 2005

Wind velocity 2003
Graph 13: Wind Velocity per direction for the years 2003, 2004, 2005
Appendix B: Grain Data

Sand Report

<table>
<thead>
<tr>
<th>Northside</th>
<th>Beach</th>
<th>Surfzone</th>
<th>Before Surfzone</th>
</tr>
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<tbody>
<tr>
<td>1000 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>fine sand</td>
</tr>
<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>900 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>fine sand</td>
</tr>
<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>800 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>fine sand</td>
</tr>
<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>700 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>very fine sand</td>
</tr>
<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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<tr>
<td>600 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>fine sand</td>
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<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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<tr>
<td>500 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>fine sand</td>
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<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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<td>400 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>very fine sand</td>
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<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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<td>300 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>very fine sand</td>
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<tr>
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<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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<td>200 meter</td>
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<td>fine sand</td>
<td>very fine sand</td>
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<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>100 meter</td>
<td>fine sand</td>
<td>fine sand</td>
<td>very fine sand</td>
</tr>
<tr>
<td>diameter</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
<td>0.105 - 0.0625</td>
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Table 17: Grain Diameter North Side
## Breakwater

<table>
<thead>
<tr>
<th>Southside</th>
<th>Beach</th>
<th>Surfzone</th>
<th>Before Surfzone</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 meter</td>
<td>0.84 - 0.50</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>200 meter</td>
<td>0.84 - 0.50</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>300 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>400 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>500 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>600 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.42 - 0.25</td>
</tr>
<tr>
<td>700 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>800 meter</td>
<td>0.84 - 0.50</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>900 meter</td>
<td>0.84 - 0.50</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
<tr>
<td>1000 meter</td>
<td>0.42 - 0.25</td>
<td>1.68 - 1.00</td>
<td>0.105 - 0.0625</td>
</tr>
</tbody>
</table>

Table 18: Grain Diameter South Side
Appendix C: Linear Wave Theory

The wave height at incipient breaking can be calculated with the following approach: calculate the energy balance (without dissipation) for an area in between two wave rays (no exchange of wave energy across the rays). A ray (or orthogonal) is a line perpendicular to the wave crest extending in the direction of the wave propagation.

Now:

\[ P_0 b_0 = P_1 b_1 \]

\[ c_{g,0} = \frac{1}{8} \rho g H_0^2 b_0 = c_{g,1} = \frac{1}{8} \rho g H_1^2 b_1 \rightarrow \frac{H_1}{H_0} = \sqrt{\frac{c_{g,0}}{c_{g,1}}} \frac{b_0}{b_1} = K_s K_r \]

To find the ratio \( b_0/b_1 \), make use of Snell’s Law to find \( \phi_1 \):

\[ \sin \phi_0 = \frac{\sin \phi_1}{c_0} \rightarrow \sin \phi_1 = \sin \phi_0 \frac{c_1}{c_0} = \sin \phi_0 \tanh kh \]

For parallel depth contours the refraction coefficient \( K_r = \sqrt{\frac{b_0}{b_1}} = \sqrt{\frac{\cos \phi_0}{\cos \phi_1}} \).
To calculate the wave height at incipient breaking \( (H_b) \), a water depth \( (h_b) \), where the breaking will occur, is assumed. With the assumed depth the wave height is calculated. After the calculations, the breaker index must be 0.6 \( (\gamma = (H_b/h_b) = 0.6) \).

The wave frequency is \( \omega = \frac{2\pi}{T} \), in which \( T \) is known and is constant for all water depths.

With the dispersion relation \( (\omega^2 = \left(\frac{2\pi}{T}\right)^2 = gk \tanh kh) \), the wave number \( (k = \frac{2\pi}{L}) \) has been determined by iteration.

Now the following equations are used:

\[
L = \frac{gT^2}{2\pi} \tanh kh
\]

\[
c = \frac{L}{T} = \frac{\omega}{k} = \sqrt{\frac{g}{k} \tanh kh} = \frac{gT}{2\pi} \tanh kh
\]

\[
c_g = nc = \frac{1}{2} \left[ 1 + \frac{2kh}{\sinh 2kh} \right] c
\]

For deep water: \( \tanh kh = 1 \)

\[
\text{(Refraction coefficient } = K_r = \sqrt{\frac{b_0}{b_1}} = \sqrt{\frac{\cos \phi_0}{\cos \phi_1}} \text{ and shoaling factor } = K_s = \sqrt{\frac{c_{g,0}}{c_{g,1}}})
\]

The wave height \( H_1 \), and after that, the breaker index, follows from \( \frac{H_1}{H_0} = K_s K_r \).
### Appendix D: Wave height and depth at incipient breaking

<table>
<thead>
<tr>
<th>Input</th>
<th>Value</th>
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<tbody>
<tr>
<td>Wave period ( T ) (s)</td>
<td>6</td>
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<tr>
<td>Breaker index ( \gamma ) (-)</td>
<td>0.6</td>
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<tr>
<td>Deep water wave height ( H_0 ) (m)</td>
<td>0.75</td>
</tr>
<tr>
<td>Deep water wave angle ( \varphi_0 ) (deg)</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Constants</td>
<td></td>
</tr>
<tr>
<td>Acceleration due to gravity( g ) (m/s²)</td>
<td>9.810</td>
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<tr>
<td>( \pi ) (-)</td>
<td>3.142</td>
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<tr>
<td>Water depth ( h_b ) (m)</td>
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</tr>
<tr>
<td>Wave frequency ( \omega ) (1/s)</td>
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<tr>
<td>Wave number ( k ) (1/m)</td>
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<td>Dispersion relation ( 4.79E-08 )</td>
<td>Must be zero</td>
</tr>
<tr>
<td>( \text{tanh} kh )</td>
<td>-</td>
</tr>
<tr>
<td>( \text{sinh} kh )</td>
<td>-</td>
</tr>
<tr>
<td>( \text{cosh} kh )</td>
<td>-</td>
</tr>
<tr>
<td>( n )</td>
<td>-</td>
</tr>
<tr>
<td>Deep water wave length (( \text{tanh} kh=1 )) ( L_0 ) (m)</td>
<td>56.21</td>
</tr>
<tr>
<td>Deep water wave celerity ( c_0 ) (m/s)</td>
<td>9.368</td>
</tr>
<tr>
<td>Deep water wave group celerity (( n=0.5 )) ( c_0 ) (m/s)</td>
<td>4.684</td>
</tr>
<tr>
<td>Wave length ( L ) (m)</td>
<td>21.80</td>
</tr>
<tr>
<td>Wave celerity ( c_1 ) (m/s)</td>
<td>3.633</td>
</tr>
<tr>
<td>Wave group celerity ( c_{g1} ) (m/s)</td>
<td>3.445</td>
</tr>
<tr>
<td>Wave angle breaker point ( \varphi_1 ) (rad)</td>
<td>0.133</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Shoaling factor ( K_s ) (-)</td>
<td>1.166</td>
</tr>
<tr>
<td>Refraction coefficient ( K_r ) (-)</td>
<td>0.974</td>
</tr>
<tr>
<td>Breaking wave height ( H_b ) (m)</td>
<td>0.851</td>
</tr>
<tr>
<td>Breaker index ( \gamma ) (-)</td>
<td><strong>0.600</strong> OK!</td>
</tr>
</tbody>
</table>

Table 19: Wave Characteristics information normal conditions
### Storm conditions

Met angle = 0

<table>
<thead>
<tr>
<th>Input</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Waveperiod</td>
<td>T (s)</td>
<td>8</td>
</tr>
<tr>
<td>Breaker index</td>
<td>γ (-)</td>
<td>0,6</td>
</tr>
<tr>
<td>Deep water wave height</td>
<td>H0 (m)</td>
<td>4,5</td>
</tr>
<tr>
<td>Deep water wave angle</td>
<td>φ0 (deg)</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>φ0 (rad)</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Constants</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Acceleration due to gravity</td>
<td>g (m/s²)</td>
<td>9,81</td>
</tr>
<tr>
<td>PI</td>
<td>π (-)</td>
<td>3,142</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water depth</th>
<th>hb (m)</th>
<th>7,25 Iterate → γ=0.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave frequency</td>
<td>ω (1/s)</td>
<td>0,785398163</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wave number</th>
<th>k (1/m)</th>
<th>0,1008235 Iterate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dispersion relation</td>
<td></td>
<td>2,89E-06 Must be zero</td>
</tr>
<tr>
<td>tanhkh</td>
<td>-</td>
<td>0,623658656</td>
</tr>
<tr>
<td>sinhkh</td>
<td>-</td>
<td>0,797826803</td>
</tr>
<tr>
<td>coshkh</td>
<td>-</td>
<td>1,279268388</td>
</tr>
<tr>
<td>n</td>
<td>-</td>
<td>0,858096016</td>
</tr>
</tbody>
</table>

| Deep water wave length     | L0 (m) | 99,92 |
| Deep water wave celerity   | c0 (m/s) | 12,490 |
| Deep water wave group celerity (n=0.5) | cgo (m/s) | 6,245 |
| Wave length                | L (m)  | 62,318 |
| Wave celerity              | c1 (m/s) | 7,790 |
| Wave group celerity        | cg1 (m/s) | 6,684392854 |

| Wave angle breaker point   | φb (rad) | 0,00000000 |
|                           | φb (deg)  | 0,000    |

| Shoaling factor            | Ks (-) | 0,967   |
| Refraction coefficient     | Kr (-) | 1,000   |

| Breaking wave height       | Hb (m) | 4,350   |
| Breaker index              | γ (-)  | 0,600 OK! |

**Table 20: Wave characteristics information Storm conditions**
Appendix E: Calculation Surfzone Longuet and Higgins

An expression for the mean wave period-averaged longshore velocity $\bar{v}_1$ was derived from different considerations. Outside the surf zone the energy flux towards the coast of a wave travelling at an oblique angle is constant. However, inside the breaker zone this is no longer the case and wave energy flux is rapidly dissipated. Longuet-Higgins derived an expression for the net thrust exerted by the waves. Assumed was that this thrust was balanced by the frictional resistance in the longshore direction he derived an expression for the mean longshore velocity:

$$\bar{v}_1 = \frac{5\pi}{8C} u_{mb} \tan \beta \sin \alpha_b$$, where C was a friction coefficient.

Subsequently Komar found from an analysis of field data that $\frac{\tan \beta}{C}$ was effectively constant, so he proposed a modified formula given by

$$\bar{v}_1 = 2.7 u_{mb} \sin \alpha_b \cos \alpha_b$$

with $u_{mb} = \frac{\gamma}{2} \sqrt{gh_b}$

Longuet-Higgins assumed $\alpha$ small and therefore $\cos \alpha \to 1$
## Appendix F: CRESS calculation longshore sediment transport

<table>
<thead>
<tr>
<th>Report:</th>
<th>Z26.1 Sediment transport due to waves (CERC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Version:</td>
<td>1.0</td>
</tr>
<tr>
<td>Rekenregel:</td>
<td>Rr_Z26_1</td>
</tr>
<tr>
<td>Date:</td>
<td>23-5-2006</td>
</tr>
</tbody>
</table>

- $g$ (Acceleration due to gravity): $9.81 \text{ m/s}^2$
- $H_0$ (Deep water wave height): $0.75 \text{ m.}$
- $T$ (Deep water wave period): $6 \text{ s.}$
- $\theta_0$ (Deep water wave angle): $20 \text{ deg}$
- $\gamma$ (Breaker index): $0.6$
- $\text{Occ}$ (Occurrence of this wave): $100 \%$
- $\text{Cerc}$ (CERC coefficient): $880000$

### $S$ (Sediment transport):

<table>
<thead>
<tr>
<th>$K_r$ (refraction coefficient):</th>
<th>$0.973668492599814$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_s$ (Shoaling coefficient):</td>
<td>$1.166894565819$</td>
</tr>
<tr>
<td>$\theta_1$ (Wave angle at breaker line):</td>
<td>$7.60446708548288 \text{ deg}$</td>
</tr>
<tr>
<td>$D_b$ (Breaker depth):</td>
<td>$1.41378138484227 \text{ m.}$</td>
</tr>
<tr>
<td>$H_b$ (Breaker height):</td>
<td>$0.852129926179229 \text{ m.}$</td>
</tr>
</tbody>
</table>

### Table 21: Calculation Sediment transport CRESS

As you can see the values at the breaker line are the same as calculated in App. D.
Appendix G: Wave set up calculations

Wave set up:

• Under normal conditions

\[ \bar{\eta}_d = - \left( \frac{kH_b^2}{8 \sinh(2kh)} \right) = - \left( \frac{0.2882 \cdot 0.85^2}{8 \sinh(2 \cdot 0.2882 \cdot 1.42)} \right) = -0.03 \text{ (At the breaker point)} \]

\[ \bar{\eta}_u = \left( \frac{1}{1 + \frac{8}{3\gamma^2}} \right) \left( \bar{h}_b - h \right) + \bar{\eta}_d = \left( \frac{1}{1 + \frac{8}{3 \cdot 0.6^2}} \right) \cdot 1.42 - 0.03 = 0.14m. \text{ (At the coastline)} \]

• Under storm conditions

\[ \bar{\eta}_d = - \left( \frac{kH_b^2}{8 \sinh(2kh)} \right) = - \left( \frac{0.1008 \cdot 4.35^2}{8 \sinh(2 \cdot 0.1008 \cdot 7.25)} \right) = -0.12 \text{ (At the breaker point)} \]

\[ \bar{\eta}_u = \left( \frac{1}{1 + \frac{8}{3\gamma^2}} \right) \left( \bar{h}_b - h \right) + \bar{\eta}_d = \left( \frac{1}{1 + \frac{8}{3 \cdot 0.6^2}} \right) \cdot 7.25 - 0.12 = 0.74m. \text{ (At the coastline)} \]

Wind set up:

\[ \kappa = c_w \frac{\rho_{air}}{\rho_{water}} = 2 \cdot 10^{-3} \frac{1.21}{1030} = 2.35 \cdot 10^{-6} \]

\[ dh_i = \sum \kappa \frac{u^2}{gh_i} F_i \cos \varphi = \left( \kappa \frac{u^2}{g \cdot 30} \cdot 47000 \right) + \left( \kappa \frac{u^2}{g \cdot 15} \cdot 1500 \right) + \left( \kappa \frac{u^2}{g \cdot 7} \cdot 1500 \right) = 0.18m \]
Appendix H: The fluidizing principle

Fluidization is a process whereby a granular material is converted from a solid state to a liquid state. This process occurs when a fluid (liquid or gas) is passed up through the granular material.

When a fluid flow is introduced through the bottom of a bed of solid particles, it will move upwards through the bed via the empty spaces between the particles. This can be done by water, pumped into a perforated pipe buried beneath the sand.

At low velocities, the drag on each particle is also low, and thus the bed remains in a fixed state. (fig 55)

![Figure 55: Fluid streaming out the fluidization pipe](image)

Increasing the velocity, the drag forces will begin to counteract the gravitational forces, causing the bed to expand in volume as the particles move away from each other. (fig. 56)

Further increasing the velocity, it will reach a critical value at which the upward drag forces will exactly equal the downward gravitational forces, causing the particles to become suspended within the fluid. At this critical value, the bed is said to be fluidized and will have a fluidic behavior.

![Figure 56: Fluidization process](image)

The fluidic behavior allows the particles to be transported like a fluid, channeled through pipes. When the slurry is pumped away, the fluidized region begins to widen into a trench. The sides of the fluidized region slides inward until an equilibrium is reached in the trench cross-section. (fig 57, 58 & 59).
Figure 57: Pumping away of the fluidized sediment

Figure 58: shows the trench which is formed

Figure 59: Trench