El Castillo del Morro

Maarten Veldhuizen
Mark Schrieks
Raymond van Broekhoven
Jaap Wierenga
Dylan Sipkema
# Table of contents

1 INTRODUCTION ........................................................................................................... 1
  1.1 Cuba ......................................................................................................................... 1
  1.2 El Castillo del Morro .................................................................................................. 1
  1.3 Systematic project approach ..................................................................................... 3
  1.4 Content of the chapters ............................................................................................ 3

2 PRESENT SITUATION .................................................................................................... 4
  2.1 Wind and waves ........................................................................................................ 4
    2.1.1 Introduction ......................................................................................................... 4
    2.1.2 Overview of local conditions .............................................................................. 4
    2.1.3 Extreme wind conditions .................................................................................... 4
    2.1.4 Influence of air pressure on the sea level .......................................................... 5
    2.1.5 Offshore waves .................................................................................................... 5
    2.1.6 Bathymetry .......................................................................................................... 7
    2.1.7 Astronomical tide ................................................................................................. 9
    2.1.8 Wind setup .......................................................................................................... 9
    2.1.9 Wave conditions in shallow water ...................................................................... 9
    2.1.10 Wave setup ....................................................................................................... 11
    2.1.11 Wave forces on the northern wall .................................................................. 11
      2.1.11.1 Decisive wave height ............................................................................ 11
      2.1.11.2 Stability .................................................................................................... 13
      2.1.11.3 Pressure distribution .............................................................................. 16
    2.1.12 Wave loads on the Southside ......................................................................... 17
      2.1.12.1 Introduction ............................................................................................. 17
      2.1.12.2 Wave loads .............................................................................................. 18
    2.1.13 Wave loads on the corner part ......................................................................... 19
      2.1.13.1 Modelling of wave pressures ................................................................. 19
      2.1.13.2 Pressure distribution .............................................................................. 21

  2.2 Rock foundation ...................................................................................................... 21
    2.2.1 Rock material properties .................................................................................... 21
    2.2.2 Actual damage of the rock ................................................................................ 23
    2.2.3 Failure mechanism ............................................................................................. 26

  2.3 Organisations involved ........................................................................................... 31

3 PROBLEM DESCRIPTION .............................................................................................. 32
  3.1 Problem analysis ...................................................................................................... 32
    3.1.1 Current state of the fortress ............................................................................. 32
    3.1.2 Current progress of the project ....................................................................... 32
    3.1.3 Problem definition ............................................................................................ 32

  3.2 Project objective ...................................................................................................... 32

  3.3 Scope of the project ................................................................................................. 32

4 BOUNDARY CONDITIONS AND ASSUMPTIONS ......................................................... 33
  4.1 Boundary conditions ............................................................................................... 33
    4.1.1 Functional conditions ....................................................................................... 33
    4.1.2 Technical conditions ......................................................................................... 33
Report project team Cuba 2005

4.1.3 Meteorological conditions ................................................................. 33
4.1.4 Environmental conditions ................................................................. 33
4.1.5 Execution conditions ........................................................................ 34
4.1.6 Juridical conditions ........................................................................... 34
4.1.7 Economical conditions ...................................................................... 34

4.2 Basic assumptions .................................................................................. 34
4.2.1 Technical / hydrological .................................................................... 34
4.2.2 Currents .............................................................................................. 34
4.2.3 Functional .......................................................................................... 34
4.2.4 Execution ........................................................................................... 34
4.2.5 Economical ........................................................................................ 34

4.3 Further assumptions ............................................................................... 34

5 LIST OF DEMANDS .................................................................................... 36

6 CONCEPTS .................................................................................................. 37

6.1 Introduction ............................................................................................. 37
6.2 Berm ......................................................................................................... 37
6.3 Seawall .................................................................................................... 38
6.4 Low-crested structures ........................................................................... 39
6.5 Caissons .................................................................................................. 41
6.6 Revetment ............................................................................................... 42
6.7 Strengthening of the existing structure ................................................... 44
6.8 Conclusions ............................................................................................ 46

7 STRUCTURAL ELABORATIONS ................................................................. 48

7.1 Introduction ............................................................................................. 48

7.2 Solutions applicable to the north side ...................................................... 48
7.2.1 Breakwater, general aspects ............................................................... 48
7.2.1.1 Stability ......................................................................................... 48
7.2.1.2 Van der Meer ............................................................................... 49
7.2.2 The submerged breakwater ................................................................. 51
7.2.2.1 Location of the breakwater ............................................................ 51
7.2.2.2 Determination crest freeboard ....................................................... 51
7.2.2.3 Armour layer ............................................................................... 51
7.2.2.4 Core ............................................................................................. 53
7.2.2.5 Filter layers .................................................................................. 53
7.2.2.6 Dimensions .................................................................................. 53
7.2.2.7 The reduction of the wave height .................................................. 54
7.2.2.8 Conclusion .................................................................................... 55

7.3 Crack formation in the foundation due to alternating loading ................. 55
7.3.1 Introduction ........................................................................................ 55
7.3.2 Data .................................................................................................... 55
7.3.3 Scaling ................................................................................................ 56
7.3.4 Determination of damage after applying an alternative ....................... 57

7.4 Detailed investigation of the strengthening of the structure ...................... 59
8 METHODS OF REALIZATION AND COSTS ............................................. 63
9 ALTERNATIVES ............................................................................. 64
10 DECISION .................................................................................. 65
11 ELABORATION OF THE CHOSEN ALTERNATIVE ....................... 66
12 CONCLUSIONS .......................................................................... 67
13 RECOMMENDATIONS .................................................................. 68
14 QUALITY ASSURANCE ................................................................. 69
15 LITERATURE .............................................................................. 70
II Summary
1 Introduction

1.1 Cuba

The fortress “el Castillo del Morro” is situated in Cuba. Cuba is a country with a population of 11.2 million people (January 2004) and has an area of 110.86 square km. The fortress was part of the old city wall of Havana, the capital of Cuba. Havana is a very densely populated city. With 2.5 million people it is the biggest city of Cuba.

In 1959 Fidel Castro became the prime minister of the republic Cuba, by liberating Cuba from its American suppression by a great revolution. His policy can be distinguished by a communistic ideology. With the falling apart of the Soviet Union and the liberalization of the Chinese market, Cuba is one of the last truly communist states in the world.

The hostility between Cuba and the US originated from the revolution resulted in a full trade embargo by the US. Up to now the relation with the US is very delicate. As a result from the still present embargo the economical situation of Cuba is not very well. The biggest trading partners of Cuba are China, Venezuela, and Canada. The gross national product of Cuba is 25.9 billion US dollar (January 2004).

The national currencies in Cuba are the peso and the convertible peso (CUC). 24 Pesos amounts to one CUC which equals 0.90 dollar. In the building industry the commonly used currency is CUC, so we will also use the CUC for this project.

1.2 El Castillo del Morro

The fortress of “el Castillo del Morro” has a very rich history. The fortress was founded in 1589 by the Spaniards to protect the harbour of Havana from intruders. During the years of Spanish colonization it was one out of many fortresses to protect Havana and its hinterland against buccaneers and pirates. In 1845 a lighthouse, the first in Cuba, was added to the fortress to serve as an important means of navigation. Nowadays it’s a national monument and contains a museum where every night at nine a big show is put on of a siege on the fortress in historical costumes. At this performance the cannonballs fly in an attempt to save the castle. Recently the fortress has been added to the UNESCO list of monuments.

The fortress is situated at the eastern side of the entrance channel of the harbour of Havana. At the western side of the channel the district ‘Old Havana’ is located. This district, together with the Vedado district, forms the main tourist centre of Havana. Like El Castillo del Morro, the total district Old Havana is part of the UNESCO list of monuments. Nowadays the fortress suffers from erosion. This report points out a solution to this problem. In Figure 1.3 a view of the castle is given and in Figure 1.4 an overview of the entire study area is given.

Figure 1.1, map of Cuba

Figure 1.2, El castillo del Morro
Figure 1.3, overview of study zone
1.3 Systematic project approach

![Diagram of systematic project approach]

1.4 Content of the chapters

In chapter 3 the problem is described in which the current state of the fortress and the current progress of the project are pointed out. Secondly it defines the main problem and objective of the project and in the third place it points out the scope of the project. Finally it takes a closer look at the present situation of Cuba, the fortress, the climate, the coast and the condition of the wall.

In chapter 3 the boundary conditions and assumptions in which the solution must be found are pointed out. The solution has to fulfill the list of demands pointed out in chapter 5. In the next chapter the concepts are defined from this point of view. After this the most realistic concepts are elaborated, both from a hydraulic and structural (Ch 7) as from an executive point of view (Ch 8). In the latter the costs will be taken into account by adding unity prices. The total will result in a table with structural designs and ways of realization, which will be presented in chapter 9. In chapter 10 a decision is made of the most suitable alternative, including the method of construction. In chapter 11 the detailing of the solution is done.

In the chapters 12 and 13 the conclusions and recommendations are pointed out. During the project the quality assurance is of great importance. This is described in chapter 14.
2 Present situation

2.1 Wind and waves

2.1.1 Introduction
In order to find a solution to the problem it is of big importance to have a clear view on the present wind and wave conditions. This data is related to the bathymetry around the fortress. Knowing this, it will be possible to describe the astronomical tide, the wave conditions in shallow water, the wave setup and the wave pressures. First the local wind conditions and the changing of the water level due to air pressure differences will be described, followed by the offshore waves.

2.1.2 Overview of local conditions
The weather in Cuba is dominated by the air pressure at the Atlantic Ocean and the cyclones which are formed there. During the summer the high pressure at the ocean is the source of the wind from the north-eastern and eastern direction. In the afternoon these winds become stronger and at night they lose their velocity. These winds are accompanied by the nice Cuban weather although in the afternoon there are very often thunderstorms due to the high value of the relative humidity in the lower part of the atmosphere.

Systematic circling from warm air on the tropical area of the Atlantic Ocean may develop tropical storms including hurricanes and tropical thunderstorms. These storms, characterized by heavy wind and rainfall are usually heading in the direction of the Caribbean including Cuba due to the eastern-western wind current which exists on this ocean in the months June till November. This explains the fact that the yearly hurricane season in the Caribbean is during the autumn months.

In winter the Atlantic influence is less and the climate is dictated by high-pressure areas which originate from the continent and develop a northern or north-eastern wind. The air pressure of these areas can be as high as 1030 mbar. The passage of this area is preceded by the existence of the “frentes frios”, cold fronts. This front is characterized by many clouds with heavy showers and thunderstorms. Before the passage of such a front the Cuban wind usually comes from the Southern direction. When a cold front as mentioned above is present in combination with a low-pressure area, cyclones can develop from the cold front zones where a large temperature gradient exists.

During the months may and June, very often low-pressure areas are located in the Caribbean. Together with the occurrence of a high-pressure area in the Atlantic region or the eastern part of the US this develops a southern wind direction, which can reach quite large velocities.

When considering the different directions, the following distribution of the different wind directions can be given:

<table>
<thead>
<tr>
<th>Direction</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>7 %</td>
</tr>
<tr>
<td>NE</td>
<td>17 %</td>
</tr>
<tr>
<td>E</td>
<td>31 %</td>
</tr>
<tr>
<td>SE</td>
<td>17 %</td>
</tr>
<tr>
<td>S</td>
<td>11 %</td>
</tr>
<tr>
<td>SW</td>
<td>6 %</td>
</tr>
<tr>
<td>W</td>
<td>5 %</td>
</tr>
<tr>
<td>NW</td>
<td>6 %</td>
</tr>
</tbody>
</table>

2.1.3 Extreme wind conditions
The extreme values are the annual maximum wind velocities. These values are obtained from the Meteor Station Casablanca, which is positioned near Havana, Cuba and are based on measurements of this institute in the period 1906 - 1996. The extremes run from 17.9 m/s up to 80.0 m/s, with an average of 29.5 m/s. When considering the data, it can be concluded that the annual extremes can have different causes. In the next table an overview of these causes is given:
Table 2.2, origins of storms

<table>
<thead>
<tr>
<th>Origin of storm</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local storms during the summer</td>
<td>28.4 %</td>
</tr>
<tr>
<td>Tropical cyclones</td>
<td>22.1 %</td>
</tr>
<tr>
<td>Extra tropical depressions during the winter</td>
<td>41.4 %</td>
</tr>
<tr>
<td>High-pressure winds</td>
<td>4.2 %</td>
</tr>
<tr>
<td>Cause hardly measurable</td>
<td>4.2 %</td>
</tr>
</tbody>
</table>

When the data is sorted using a Frechet distribution the following relation can be made for the intensity of storms and the period of return:

Table 2.3, annual wind velocities

<table>
<thead>
<tr>
<th>Category</th>
<th>Maximum wind velocity (m/s)</th>
<th>Period of return (years)</th>
<th>Storm intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (SS1)</td>
<td>32 - 43</td>
<td>10</td>
<td>Low</td>
</tr>
<tr>
<td>2 (SS2)</td>
<td>43 - 50</td>
<td>25</td>
<td>Average</td>
</tr>
<tr>
<td>3 (SS3)</td>
<td>50 - 58</td>
<td>50</td>
<td>High</td>
</tr>
<tr>
<td>4 (SS4)</td>
<td>58 - 69</td>
<td>100</td>
<td>Extreme</td>
</tr>
<tr>
<td>5 (SS5)</td>
<td>&gt; 69</td>
<td>200</td>
<td>Catastrophic</td>
</tr>
</tbody>
</table>

2.1.4 Influence of air pressure on the sea level

Normally the storms and other depression kind of wind inducing weather are accompanied by a low value of the air pressure. The rise or declination of the air pressure has an influence on the sea level. A reference value of the atmospheric pressure and a related sea level is known, which gives a quite easy relationship between the sea level increment and the air pressure increment:

\[
\Delta h = \frac{\Delta p}{100}
\]

\(\Delta p\): pressure increment (mbar)

\(\Delta h\): sea level increment (m)

From this relation it can be concluded that a rise of the sea level is caused by a lower atmospheric pressure than the reference value. This reference is given by the Casablanca meteorological station and has the following value:

\[P_{\text{average}} = 1016\text{mbar}\]

When considering data from the heaviest storms in the years 1970 to 1994, one can determine that the minimum atmospheric pressure during storms is

\[P_{\text{minimum}} = 990\text{mbar}\]

From which a sea level rise of 0.26 m can be calculated.

2.1.5 Offshore waves

The wind causes two different actions which can be of importance for our project area. Firstly the wind causes waves; the height of the waves has a direct relation with the velocity and the direction of the waves. Because of the topographical situation of our project area (Figure 1.4) the wind oriented to the northern direction does not have a large impact on the wave situation. This is mainly caused by the offshore wind character of this direction. On contrary, the wind originating from the opposite direction, from Northeast to Northwest has a large influence on the wave patterns and wave heights of the study area.

The information on this is acquainted from\[\text{ref. x (geocuba estudios marina)}\] a short over view on the wave records is given in Table 2.4.
All this information is verified by two methods which will be described in the following. The first check has been done by the nomogram of 'Groen and Dorrestein'. In order to do this some information on the geography like water depth, fetch and duration of a storm is needed. Since the Gulf of Mexico is very deep in relation to the wave height, the water depth can be considered as deep water. The fetch and the duration are related to each other at deep seas due to the absence of a nearby shore. In the North-West direction the fetch is over 1.000 kilometres. The duration of the peak of a storm is approximately 9 hours; with a speed of over the 200 km/h. this input in the nomogram given below in Figure 2.1 gives a very good match to the information given by the report.

These parameters can be summarised in one parameter called the dimensionless fetch. This is the normal fetch multiplied by the gravity and divided by the square of the wind speed. This leads to the other method of verifying the information. The given wave height and period can be made dimensionless by a similar computation. The dimensionless wave height and period are compared to the fetch in the graph of Kahma and Calkun, see Figure 2.3. The goal of doing this is the fact that the data can be compared to any other situation, as a result of omitting the dimensions. This is also of good use for future scale model testing. According to this graph the data is also correct. A notable fact is that the fetch length is of such a size that the waves can develop completely.
2.1.6 Bathymetry

To calculate wave height in shallow water, wave run-up and wave set-ups, the bathymetry is a very important factor. This means that the slope of the bottom, the bottom materials and irregularities in the bottom have influence on the wave conditions. The material of the bottom consists totally of rock. In the area the sediment transport is very small, so that it can be neglected. In a following chapter the characteristics of the rock will be described.

The bathymetry around the Morro Fortress is very complicated. There are no straight depth contours due to the rock coast, so the depth differ form meter to meter. There are also plenty of irregularities in the bottom. In order to get a clear view of the bathymetry, the coast around the Morro Castle is divided in three parts. The parts are indicated in Figure 2.3.

Figure 2.3, The division and depth contours of the coast around the fortress

In the first section the depth contours are nearly straight. In case of the determinative North-West wind direction, the waves on the north-side are almost perpendicular to the shore. This gives a frontal impact of the waves against the rocky foundation of the fortress. This is the dominant load on this part of the foundation. In this area the wall is mainly damaged by caves due to the scour below the MSL, this weakens the foundation of the wall. The cross-section near the biggest cave on the north-side is taken as the normative cross-section. This one is reflected in Figure 2.4.
This cross-section makes clear that the slope of the bottom is very steep near the coast. This means that normal waves will break on the foundation, which causes a high impact. At storm surges, the high waves will break earlier and will not break on the foundation of the fortress.

In accordance with section 1 the depth contours are in section 2 also nearly straight and perpendicular to the shore. In this area there are four processes of wave transformation. At first, these are the waves from the NW direction. These waves travel almost parallel to the coast. These waves cause friction at the walls. Second there are the waves that diffract around the western point of the fortress.

Next there is the refraction of the waves due to the bottom’s slope. These cause a frontal impact on the wall, but are small compared to the impacts that occur at the north side. Final there are the wave reflections at the south wall of the approach channel. In a following chapter these processes will be elaborated further. In Figure 2.5 the dominant cross-section of this section is given. This cross-section is taken at the big hole at the part of the 12 apostoles. This is the biggest hole in this section, so it will be the normative cross-section.

The slope of the bottom is in this case not so steep. The depth in front of the walls is not very high. If varies form -1.5 meters till -2 meters. In this case the waves will not break at the foundation, but earlier.

In the third and final section the depth in front of the fortress is very big. Depths till 11 meters are measured here. This means a very steep, almost nearly vertical slope of the rocky coast in front of the fortress. In this section, not only normal waves but also the high waves will break on the shore. This gives a high impact on the foundation of the fortress. Unlike the other two sections, there are no straight depth contours in this area and there are a lot of deep lying holes in the bottom.
2.1.7 Astronomical tide

The astronomical tide which exists in the study area is of course not the result of climatologic influences but does affect the height of the water level. According to the Estacion Mareografica of Siboney, Havana the average value of the tide amplitude is 0.28 m, and an annual maximum sea level value runs up to 0.39 m above mean sea level.

The water level in the basin changes constantly by the tide differences at the ocean, this transport of water induces a current in the channel. High velocities can bring damage to the present banks and to the future revetment. The currents are also of big importance during the construction phase for the waterborne equipment.

To estimate the velocity an approach is used in which the immersion of the channel is neglected and the water level in the basin and at the entrance are in phase. The channel is 220 meter wide and at most 14 meter deep in the middle. By knowing the basin’s bathymetry and the frequency of the M2 tide, the current can easily be determined for a 28 cm excitation. Under normal circumstances this is 0.1 m/s and at storm surges it is at most 0.1 m/s. These low speeds are of no influence and by using a more detailed method they certainly still would not.

2.1.8 Wind setup

Wind setup is a phenomenon which is the result of the friction of the wind flow on the water surface and the friction between the water and the bottom of the sea. Of great importance on this phenomenon is the shape of the coast. In our case the coast is a very steep slope from the bottom of the sea up to the shore. Because of this steep slope, very little friction between the water and the bottom occurs compared to the volume of water. A small calculation can prove the low value of the wind setup. Here the wind setup is equal to a constant divided by the gravity and depth, this times the wind speed squared.

\[
\frac{dS}{dx} = C_2 \frac{u^2}{gd}
\]

So it can be said that the wind setup has very little influence on the sea level, and the values which are used for the calculation are listed below:

<table>
<thead>
<tr>
<th>Return Period [years]</th>
<th>Wind setup [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
</tr>
<tr>
<td>25</td>
<td>0.02</td>
</tr>
<tr>
<td>50</td>
<td>0.04</td>
</tr>
<tr>
<td>100</td>
<td>0.06</td>
</tr>
</tbody>
</table>

2.1.9 Wave conditions in shallow water

The most important feature of shallow water is the breaking of the waves. This is a very complicate process in which a lot of wave energy is dissipated. In this area also wave setup takes place due to a gradient in the radiation stress.

The breaking point is an important factor in the hydrodynamic behaviour of the coast. During normal wave conditions the waves will brake on the shoreline, this means a high impact on the foundation of the fortress. At a storm surge the wave height is bigger and the wave will brake at a larger depth. In order to calculate the wave height the angle of incidence of the waves has to be known. The determinative wind direction is north-west, which gives an angle of incidence of the waves of approximately 5 degrees. This is indicated in Figure 2.6.
The wave motions in the breaker zone can be calculated in different ways. As a first approach the wave will break in constant proportion to the depth. For this purpose we use the breaker index ($\gamma$) which is defined as the ratio between the wave height and water depth at the breaking point. This value is related to the Iribarren number according to the following equation:

$$\xi_0 = \frac{\tan \alpha}{\sqrt{H/L_0}}$$

(2.3)

With the given information of deep water conditions, the Iribarren number can be calculated for return periods of 1, 25 and 50 years which is indicated in the table below.

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>$H_0$ (m)</th>
<th>$L_0$ (m)</th>
<th>$\xi_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.2</td>
<td>60.02</td>
<td>0.59</td>
</tr>
<tr>
<td>25</td>
<td>6.7</td>
<td>165.64</td>
<td>0.57</td>
</tr>
<tr>
<td>50</td>
<td>7.6</td>
<td>195.85</td>
<td>0.58</td>
</tr>
</tbody>
</table>

The Iribarren number is on average 0.58. This means that we deal with plunging waves. Figure 2.7 indicate the relation between $\gamma_b$ and $\xi_0$ and the scatter of the data. This figure points out that the breaker index lies approximately between 0.8 and 1.2

Figure 2.7, relation between Iribarren number and breaker index

It may be clear that the breaker index can be estimated by different methods, which all result in a different value. These different values have a large influence on the depth at the breaking point, as indicated in Figure 2.8. The wave height is not as sensitive to changes in the breaker index as the depth is.
Both Figure 2.7 and Figure 2.8 show that the value of the breaker index is very difficult to calculate precisely. Fortunately the wave setup caused by irregular waves is not very sensitive to this index. Hence an estimation of the breaker index will be sufficient. To estimate $\gamma_b$ in this document an expression based on an energy wave model (ENDEC) in which both shoaling and wave breaking are included is used.

$$\gamma_b = 0.5 + 0.4 \tanh \left( \frac{33 H_b}{L_b} \right)$$  \hspace{1cm} (2.4)

For the determinative wind direction (NW) the breaker index will be 0.84. With this parameter the wave height and depth at the breaking point can be calculated according to the linear wave theory. These figures are indicated in Table 2.7.

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>$H_b$ (m)</th>
<th>$d_b$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.28</td>
<td>2.73</td>
</tr>
<tr>
<td>10</td>
<td>5.61</td>
<td>6.70</td>
</tr>
<tr>
<td>25</td>
<td>6.86</td>
<td>8.20</td>
</tr>
<tr>
<td>50</td>
<td>7.83</td>
<td>9.30</td>
</tr>
<tr>
<td>100</td>
<td>8.92</td>
<td>10.65</td>
</tr>
<tr>
<td>200</td>
<td>9.96</td>
<td>11.90</td>
</tr>
</tbody>
</table>

### 2.1.10 Wave setup

For a first estimation of the wave set-up we use equation (2.5), which is valid for irregular waves with slopes lower than 10%. Like previously shown in Figure 2.8 and as can be determined from the following equation, the wave setup for irregular waves is not sensitive to changes in the breaker index.

$$n = 0.25 H_s \cdot \frac{\gamma_{op}}{\gamma_{op}}$$  \hspace{1cm} (2.5)

The wave set-up in the breaker zone is 1.73 m for waves from the north-west direction with a return period of 100 years.

### 2.1.11 Wave forces on the northern wall

#### 2.1.11.1 Decisive wave height

Regarding the wave forces acting on the rock foundation, there are two representative types of waves that could be decisive for the stability of the structure. First of all standing waves are considered. These waves do not have a big pressure impact on a wall, but can cause a great run up at the seawalls, which could result in overtopping. Since
overtopping at the case of “El Castillo del Morro” is not an issue, standing waves are of minor importance compared to breaking waves. Besides a rise in the hydrostatrical pressure, breaking waves also create a big dynamical pressure on seawalls.

When calculating the forces acting on the foundation, the wave height is of great importance. Under storm conditions occurring once every 100 years, the sea level rise is at most:

<table>
<thead>
<tr>
<th>Cause</th>
<th>Sea level inclination (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air pressure</td>
<td>0.26</td>
</tr>
<tr>
<td>Tide</td>
<td>0.39</td>
</tr>
<tr>
<td>Wind setup</td>
<td>0.06</td>
</tr>
<tr>
<td>Total</td>
<td>0.71</td>
</tr>
</tbody>
</table>

The biggest waves at such a storm are 8.6 m at deep water. Their height at breaking according to chapter 2.1.9 is 8.92 m. With a breaking index of 0.99 the water depth at the breaking point must be 9.01 m. The cross section as described in chapter 2.1.6 is further schematized in Figure 2.9, so computations can be made more easily.

As can be calculated from the cross section, a water depth of 9.01 m lies approximately 70 m offshore at the highest high water. It seems very unlikely that a wave breaking 70 m offshore will cause the biggest impact on the foundation below the seawall. Waves breaking closer to the shoreline will have a direct impact on the foundation, and may cause a much higher pressure. An assumption is made, that waves breaking at the change in the profile, 18.1 meter from the shoreline, will cause the largest impacts.

Although it is merely an assumption it has been made with the following reason. Closer to the shoreline, the water depth is less hence the height of the waves breaking there is lower. So this assumption is a conservative approach. Waves breaking further from the shoreline are indeed bigger. However with a small declination of the sea bottom at the sea side of the chosen point, the error made is only small. If for example the waves break 10 meters before the point assumed, the difference in wave height is only 0.44 m. Secondly, waves that did not break before the change in the profile are likely to break soon after. This implies that a lot of waves break at the same point and are likely to cause erosion. Another argument lies in the Iribarren numbers of waves getting instable on the bottom slopes. Because of the differences in steepness of the bottom, the Iribarren number also differs for an arbitrary wave on both slopes.

\[
\xi = \tan \alpha \left(\frac{H}{L_0}\right)^{1/2}
\]

From the bathymetry and the breaker index the place where a wave with a certain wave height becomes instable can be determined. With the wave characteristics from Table 2.4 and Table 2.7, the Iribarren number can be calculated. The Iribarren numbers for several waves are plotted against their place of breaking in Figure 2.10. For information about the calculations one is referred to appendix XXX.
Figure 2.10, Iribarren number for various distances to the shoreline

Close to the shore waves break with an Iribarren number of about 1.7. According to Battjes the breaker types of waves are plunging as ξ has a value somewhere between 0.5 and 1.5, as can be seen in appendix xxx. Further from the coast values of approximately 0.2 are calculated. This indicates spilling waves. The biggest wave impacts can be measured when the Iribarren is around 3. So this fact also indicates that waves with the highest impact on the foundation do not break before the bending point in the profile. The biggest waves breaking on the point bending point in the profile break at the highest high water. With a depth of 6.71m and a breaker index of 0.99 the wave height results in 6.64m.

2.1.11.2 Stability

For modelling the forces on the foundation of “el Castillo del Morro” several methods of calculating are applicable. Since there are a lot of uncertainties in the modelling, three methods have been used to compare their mutual deviations. These methods are CERC 1984, Minikin and Goda-Takahashi. For more information about these methods one is referred to annex [XXX].

Of these methods, CERC 1984 is the most simplified method. It uses a hydrostatic and a dynamical component to model the wave impact. A constant value is assigned to the dynamical part, depending on the height of the wave, and its wave speed. This is depicted in Figure 2.11; the values of the load components are given in Table 2.10 and in Figure 2.14.

Figure 2.11, schematized wave pressure according to CERC 1984

Minikin also uses a hydrostatic and a dynamical part to describe the wave pressure. Minikin’s method however has a dynamical peak at sea level with a parabolic decrease to zero at the upper and lower bound of the wave. In this report the parabolic shape has been linearized leading to a minor error in the estimation. A scheme of this modelling is indicated in Figure 2.12, the values are given in Table 2.10 and in Figure 2.15.
Unlike the other methods Goda-Takahashi is not formed by a hydrostatical and a dynamical component. This method has more parameters that can be taken into account. However it does not give much insight in the origin of the forces. A scheme of the loads is given below in Figure 2.13, the values are again given in Table 2.10 and in Figure 2.16.

Using these methods with the given parameters led to the results listed in Table 2.10 and given in Figure 2.14, Figure 2.15 and Figure 2.16. More information about the results is shown in annex XX.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>CERC 1984</th>
<th>Minikin</th>
<th>Goda-Takahashi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum wave pressure (kN/m²)</td>
<td>89.82</td>
<td>69.87</td>
<td>106.95</td>
</tr>
<tr>
<td>Maximum pressure (kN/m²)</td>
<td>123.55</td>
<td>100.85</td>
<td>146.97</td>
</tr>
<tr>
<td>Wave force (kN/m)</td>
<td>727.26</td>
<td>403.80</td>
<td>965.49</td>
</tr>
<tr>
<td>Total force (kN/m)</td>
<td>960.42</td>
<td>636.96</td>
<td>1059.28</td>
</tr>
</tbody>
</table>

Table 2.9, wave parameters at the northern part

Table 2.10, values of the wave loads, calculated with the differed methods
Figure 2.14, load components according to CERC 1984

Figure 2.15, load components according to Minikin

Figure 2.16, Load components according to Goda-Takahashi
Although quite a big difference in values is obtained by the three different methods, they are of the same order of magnitude. The outcome resembles the graph given in Figure 2.17 valid at the values \( HD = 6.0 \text{ m}, h = 9.0 \text{ m}, \ h' = 7.0 \text{ m}, \ d = 5.0 \text{ m} \) and \( h_c = \infty \).

**Figure 2.17, comparison of the total wave forces from the different wave models**

The Goda-Takahashi method can be seen as an upper bound at the given parameters and the solution by Minikin can be seen as a lower bound. The wave force is acting as solicitation on the construction, so taking the upper bound leads to the most save design. In addition, more parameters can be taken into account in this method suggesting to be more suitable for the situation. For example, the Goda-Takahashi distinguishes between the water depth at breaking and the water depth at the place of impact. In this case the point of impact is assumed in the middle between the breaking point and the shoreline. This place seems reasonable considering the wave speed and the time needed to fall the wave height of 6.64 m. At this point the water depth is half the water depth at the bending point plus half the wave setup, resulting in 4.22 m. In the Goda-Takahashi this difference is taken into account by taking a berm with a water head of 4.22 m.

Even though the Goda-Takahashi method is widely used to calculate the force of breaking waves on a seawall it is not completely satisfying for this situation. This is mainly caused by the sloping shore on which the waves break in stead of a vertical wall. Especially because of the empirical character of the methods, applying it here will give a certain error with respect to the occurring situation. To correct for this error the given values will be multiplied by the cosine of the angle of the sloping shore. With a slope of 42° this results in a multiplier of approximately 0.70. The reduced values are given in Table 2.11.

**Table 2.11, reduced values according to Goda-Takahashi**

<table>
<thead>
<tr>
<th>Loads</th>
<th>Goda-Takahashi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum wave pressure (kN/m²)</td>
<td>73.7</td>
</tr>
<tr>
<td>Maximum pressure (kN/m³)</td>
<td>112.1</td>
</tr>
<tr>
<td>Wave force (kN/m)</td>
<td>669.7</td>
</tr>
<tr>
<td>Total force (kN/m)</td>
<td>757.2</td>
</tr>
</tbody>
</table>

**2.1.11.3 Pressure distribution**

The wave pressures given in the previous paragraph can be considered as an average pressure during a wave attack. These pressures are important for calculating the stability of the structure, but are not the highest pressures that occur during a wave attack. This is due to a great peak in the pressure distribution of a breaking wave. This distribution is schematically given in Figure 2.18. Note that the time scale is not linear.
Figure 2.18, pressure distribution during wave impact

The horizontal line in this graph can be regarded as the pressure calculated with the Goda-Takahashi method. Calculating the height of the peak is still a field of research. The following approximation has been made by TAW:

\[ p_{\text{max,50\%}} \approx 8 \rho_w g H_s \tan \alpha \]  

\[ p_{\text{max,0.1\%}} \approx 16 \rho_w g H_s \tan \alpha \]  

With \( H_s = 6.64\text{m} \) and \( \alpha = 42^\circ \), this leads to:

\[ p_{\text{max,50\%}} \approx 480.94 \text{kN}/\text{m}^2 \]  

\[ p_{\text{max,0.1\%}} \approx 961.87 \text{kN}/\text{m}^2 \]

The 0.1\% upper boundary of the peak pressures according to TAW are just over 13 times as high as the maximum wave pressures given by the Goda-Takahashi method, reference XX, the “Coastal structures 2003” the article “Dynamic wave loads on coastal structures: Analysis of impulsive and pulsating wave loads” refers to an example wave load where the measured peak value of a breaking wave is about 20-30 times as high as the value found by the formula of Goda-Takahashi. So although 13 times as high as the value found by the Goda-Takahashi method is even rather small, it seems like a reasonable value.

2.1.12 Wave loads on the Southside

2.1.12.1 Introduction

The problems occurring at the Southside are localized at the 12 Apostoles. This paragraph will deal with the waves which reach the shore at this part of the area. The shore line at the 12 Apostoles is curved, and differs for a big part from the average direction of the channel. The depth contours just outside of the coastline are also curved. At the entrance of the channel the contours are almost perpendicular to the shoreline. In front of the 12 Apostoles these contours are parallel with the shoreline. Four processes of wave transformation are active at this area. First, the waves from the NW direction, these waves travel almost parallel to the coast. Next are the ones that diffract around the western point of the fortress. Third there is the refraction of the waves due to the bottom’s slope, which cause a frontal impact. Final there are the waves from the west, which are not obstructed by anything, induct perpendicular to the shore and can reach a height up to 3.5 meters. These four types will be discussed in this paragraph.
Figure 2.19, four different wave types at the south side

2.1.2.2 Wave loads

The first type of waves to be discussed is the one occurring during the heaviest storms, these come from the north-west. They do not directly hit the coast; this because the offshore wave direction is parallel to the coast line. They cause friction along the walls and scour along the shore line. The maximum intern wave velocity of these waves is two meter per second. This is rather low, so it only causes a small amount of erosion over many years.

The second type is about diffraction around the west point of the fortress. When the wind comes out of the northern or north-eastern direction the south side turns into a shadow zone. The coastline can be seen as a single breakwater, which the waves diffract around. Even though the diffraction, they can not be the most important waves in the area. Because these waves lose a lot of energy before reaching the southern wall they will be inferior to the direct incident waves from the west, described in the fourth case. Although the load is not determined, they do cause scour in time.

The next type to be discussed is the refraction. This is a special case since it can be related to waves of all directions. The wave height and angle of induction are two important parameters which are opposite in this case. The biggest waves from the north-west have an induction of 90 degrees, parallel to the shore line, which reduces them to an insignificant height at the southern shore line. Up to an induction of 60 degrees these waves are divided less than a half in height. This height is still less than that of the waves direct of the west. Waves from a more western direction have less height but a more ‘favourable’ induction and they are less reduced in height during the refraction process. These waves do not loose much of their energy and height, but again compared to waves perpendicular to the shoreline they are of minor influence. The perpendicular waves give a full impact on the shore, as waves from other directions are reduced. Therefore these waves do not give the determining load as well, but they cannot be neglected since they cause some minor loads from different directions.

Waves which reflect on the southern bank of the approach channel are not a load on the south side of the fortress. This is simply not possible because the bank is more land inwards, out of the project zone and the reflecting waves travel even further away of the fortress. Therefore the waves which reflect are not of importance to the situation.

The fourth and last type of waves are the waves from the west, they have been mentioned in all sections before. They travel from the Gulf of Mexico just around the most northern part of Havana, Punta Brava. And they also just pass the
southern head of the approach channel. This free passage allows these waves to hit the coastline directly at the 12 Apostoles. The angle of wave induction is 45 degrees with the average shoreline of the channel, but the approach is perpendicular to a large part of the 12 Apostoles. This point is most damaged and therefore the determining cross section, an illustration is shown in Figure 2.20 and Figure 2.19, and is fully described in chapter 2.1.6. At the shore is a vertical wall, which has to withstand the wave load. Therefore the load at this wall is described.

Figure 2.20, the determining cross section at the 12 Apostoles just south of the cave

According to ref xx geocuba and Table 2.4 the waves from the west can reach a offshore height of 4.7 meter once every 50 years. At the shore this height will be reduced due to shallower waters. The same Goda and TAW approach is used for this coast as for the northern side.

The mean water depth just in front of the shore line is 2.5 meter, an additional wind setup, tide and other factors increase the depth to a maximum of 4 meters. Only the extreme case is calculated since that will give the determining load. By the breaker index of 0.99 as described in chapter 2.1.9, this results in waves of 3.96 meter. The results of the average load on the bank and peak loads on the wall are given in Table 2.12. Overviews of the components of these loads are shown for Goda in Figure 2.21, these loads are the upper boundary loads and are save to use for further protection methods.

Table 2.12, wave loads on the southern wall

<table>
<thead>
<tr>
<th>Method</th>
<th>Load</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Goda</td>
<td>Force on the bank (kN)</td>
<td>329</td>
</tr>
<tr>
<td>Goda</td>
<td>Pressure on the bank (kN/m²)</td>
<td>47</td>
</tr>
<tr>
<td>TAW</td>
<td>Max. pressure by 50 % (kN/m²)</td>
<td>287</td>
</tr>
<tr>
<td>TAW</td>
<td>Max. pressure by 0.1% (kN/m²)</td>
<td>574</td>
</tr>
</tbody>
</table>

Figure 2.21, load components according to Goda-Takahashi

2.1.13 Wave loads on the corner part

2.1.13.1 Modelling of wave pressures

In chapter 2.1.6 a division has been made between three sections. The reason for this, was both for the direction of the shoreline and the bathymetry just in front of the shoreline. For the corner section especially the bathymetry differs
strongly from the other two sections. The bathymetry of this corner section as given in chapter 2.1.6 is further schematized in Figure 2.22 to be able to make computations.

Figure 2.22, cross section of the corner part

As can be seen in this picture the bottom slope just before the shoreline is very steep. Not far out of the shore, there is a sharp angle in the bottom profile at a depth of 6.6m. Another bending in the bottom profile is located 27.0m out of the shoreline.

Waves out of the western and northern direction and all the directions in between (clockwise) are able to reach the corner section. This includes waves from the north-west, which are the biggest in the area. For this situation waves with a $H_0$ of 8.6m occur once every 100 years.

Again the Iribarren numbers of the breaking waves are taken to get an insight on the way of which waves break. The Iribarren numbers for several waves are plotted against their place of breaking in Figure 2.23. For information about the calculations one is revered to annex XXX.

Figure 2.23, Iribarren number for various distances to the shoreline

The Iribarren numbers for small waves which become instable on the steep slope of 71.5° close to the shoreline are relatively high. According to Battjes the breaker types of waves become surging as $\xi$ is bigger than 5 as can be seen in annex xxx. Although the Iribarren parameter is not truly valid for steep slopes, a number higher than 14 indicates that waves which become instable on the steep slope don’t break on the foundation.

The waves which become instable on the slope of 11.25° have an Iribarren number of approximately 1. This means that the breaker type is plunging. Although the highest loads are present at an Iribarren number of about 3, the impacts of these waves will be huge. The wave heights breaking on this part in the profile are bigger than 6.53m. The biggest wave breaking once every 100 years is 9.15m. In accordance with the list of demands this is taken as the decisive wave height.

Because the pressures due to standing waves are much smaller than those caused by the breaking waves, only the breaking waves are considered. Again the method of Goda-Takahashi is used to calculate the forces on the foundation. A table with results of this calculation is given in Table 2.13, and its accompanying graph can be found in Figure 2.24.
Table 2.13, wave loads acting on the corner part

<table>
<thead>
<tr>
<th>Loads</th>
<th>Goda-Takahashi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum wave pressure</td>
<td>123.20 kN/m²</td>
</tr>
<tr>
<td>Maximum pressure</td>
<td>186.88</td>
</tr>
<tr>
<td>Wave force (kN/m)</td>
<td>1660.15</td>
</tr>
<tr>
<td>Total force (kN/m)</td>
<td>1885.79</td>
</tr>
</tbody>
</table>

Figure 2.24, pressure on the corner wall according to Goda-Takahashi

Again, a sine of the angle of the sloping shore is used as a multiplier for the wave pressures to correct for the difference between the vertical wall as modeled and reality. With an angle of 71.5°, the multiplier amounts to 0.95. A multiplication of the wave pressures of the previous table leads to the figures as given in Table 2.14.

Table 2.14, modified wave loads

<table>
<thead>
<tr>
<th>Loads</th>
<th>Goda-Takahashi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum wave pressure</td>
<td>115.44</td>
</tr>
<tr>
<td>Maximum pressure</td>
<td>180.65</td>
</tr>
<tr>
<td>Wave force (kN/m)</td>
<td>1574.36</td>
</tr>
<tr>
<td>Total force (kN/m)</td>
<td>1800.00</td>
</tr>
</tbody>
</table>

2.1.13.2 Pressure distribution

The TAW formula is not really applicable for the situation at the corner section, because of the angle in the bottom profile just in front of the shoreline. In this case the waves break due to a slope of the bottom of 11.25°, but have their impact on a wall of 71.5°. The TAW formula is only applicable for slopes with one angle. Taking 11.25° would lead to a largely underestimated pressure produced by the wave impact. At the other hand, 71.5° lies outside of the domain for which the formula is valid. With the value of $p_{max.1%}$ growing to an asymptote at $\alpha$ is equal to 90°, the formula is already out of its domain at an angle of 71.5°. As a rough estimation a peak pressure of 20 times the value calculated with the Goda-Takahashi method is taken, which is equal to a pressure of 2308.8 kN/m.

2.2 Rock foundation

2.2.1 Rock material properties

In this paragraph the main material properties, such as strength and deformation characteristics, of the rock beneath ‘El Castillo del Morro’ are described. These properties can be determined from a few material classifications, and together with a small computer program named ‘Roclab’ several failure criteria can be defined. The material classifications are mainly dependent on the type of rock and its quality, considering number of joints and irregularities. Classifications
needed to develop a failure criteria are: General Strength Index (GSI), intact rock parameter \((m_i)\), and the intact compressive strength \((\sigma_{ci})\).

The rock bottom of 70 % of the Cuban area is formed of corral in the magasonic period, from which one can conclude that the age of the rock is about 5 – 23 million years. The rock at the coast of Havana is made of limestone with in this case a fine texture. Usually many discontinuities can be found in the Cuban rock, which gives rise to many difficulties in determining the rock strength parameters and the reliability of research data.

From the main material and the number of discontinuities the quality of the rock can be defined, resulting in a quite low value of 30 for the GSI. To determine the \(\sigma_{ci}\) some fieldwork has been done. It seemed that peeling a small part of the rock could easily be done, which indicates a low strength. Together with information gathered at the soil mechanics department of CUJAE an estimation of the \(\sigma_{ci}\) of 12 MPa is made. Finally the intact rock parameter is to be determined, which value is entirely based on the type of rock and in this case results in a value of 8. The resulting failure envelope is together with the input data given in Table 2.15 and Table 2.16.

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>GSI</td>
<td>30</td>
</tr>
<tr>
<td>(\sigma_{ci})</td>
<td>12 MPa</td>
</tr>
<tr>
<td>(m_i)</td>
<td>8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>rock mass tensile strength (\sigma_t)</td>
<td>0.0 MPa</td>
</tr>
<tr>
<td>Uniaxial rock compression strength (\sigma_c)</td>
<td>-0.2 MPa</td>
</tr>
<tr>
<td>Global rock mass compressive strength (\sigma_{cm})</td>
<td>-1.2 MPa</td>
</tr>
<tr>
<td>Youngs modulus (E)</td>
<td>1.1 Gpa</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0.4 MPa</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>23°</td>
</tr>
</tbody>
</table>

Rock experts of the CUJAE gave an estimation of the Young’s modulus which is of the same order as the results from the computer program. The output of the computer program is a translation from estimated test data to ‘real’ in situ rock mass parameters. The large difference in the values of the intact rock strength and the rock mass strength is mainly caused by the existence of discontinuities in and the surface conditions of the in situ rock.

The applied failure criterion is Hoek-Brown, which is mostly used in case of rock investigations. From this criterion the Mohr-Coulomb criterion is derived by curve fitting. Both criteria are displayed in Figure 2.25, where the overestimation of the Mohr Coulomb at very low stresses can be seen.

---

**Figure 2.25, failure envelope**
In order to get more familiar with the material behaviour a stress-strain diagram for compression is given in Figure 2.26. The material has a brittle fracture behaviour which results in a small strain softening part. Tension strength is very low in these kinds of materials and can therefore be neglected. It should be noted that this stress-strain relation is determined by laboratory tests and therefore gives only an idea of the in situ rock behaviour.

**Figure 2.26, stress-strain diagram**

### 2.2.2 Actual damage of the rock

At the moment of study the rock has several damages. In Table 2.17 this is pointed out for a number of sections. For each of these sections the damage kind, damage level, cause of the damage and the position relative to the mean sea level is depicted. The location of the sections is shown in Figure 2.27.

<table>
<thead>
<tr>
<th>Section</th>
<th>Damage kind</th>
<th>Damage level</th>
<th>Cause</th>
<th>Position (msl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>shear</td>
<td>high</td>
<td>wave impact</td>
<td>0 - +5</td>
</tr>
<tr>
<td>B</td>
<td>wear</td>
<td>medium</td>
<td>scouring</td>
<td>-1 - +0.2</td>
</tr>
<tr>
<td>C</td>
<td>cave</td>
<td>high</td>
<td>wave impact</td>
<td>+0 - +4.0</td>
</tr>
<tr>
<td>D</td>
<td>cave + cracks</td>
<td>high</td>
<td>scouring</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>shear</td>
<td>low</td>
<td>wave impact</td>
<td>+6 - +15</td>
</tr>
<tr>
<td>4</td>
<td>shear</td>
<td>low</td>
<td>wave impact</td>
<td>+8 - +9</td>
</tr>
<tr>
<td>5</td>
<td>shear</td>
<td>low</td>
<td>wave impact</td>
<td>+7 - +13</td>
</tr>
<tr>
<td>6</td>
<td>shear</td>
<td>low</td>
<td>wave impact</td>
<td>+8.4 - +13</td>
</tr>
<tr>
<td>7</td>
<td>cantilevering rock</td>
<td>high</td>
<td>wave impact</td>
<td>+7 - +13</td>
</tr>
<tr>
<td>8</td>
<td>cantilevering rock</td>
<td>high</td>
<td>wave impact</td>
<td>+7 - +13</td>
</tr>
<tr>
<td>9</td>
<td>cantilevering rock</td>
<td>high</td>
<td>wave impact</td>
<td>+7 - +13</td>
</tr>
<tr>
<td>10</td>
<td>cantilevering rock</td>
<td>high</td>
<td>wave impact</td>
<td>+7 - +13</td>
</tr>
<tr>
<td>11</td>
<td>shear</td>
<td>high</td>
<td>wave impact</td>
<td>0 - +13</td>
</tr>
<tr>
<td>12</td>
<td>shear</td>
<td>high</td>
<td>wave impact</td>
<td>+3 - +14</td>
</tr>
<tr>
<td>13</td>
<td>crack + cracks</td>
<td>high</td>
<td>Scouring + wave impact</td>
<td>0 - +10</td>
</tr>
<tr>
<td>14</td>
<td>cracks/shear</td>
<td>high</td>
<td>wave impact</td>
<td>-11 - +12</td>
</tr>
<tr>
<td>15</td>
<td>shear</td>
<td>high</td>
<td>wave impact</td>
<td>0 - +12</td>
</tr>
<tr>
<td>16</td>
<td>shear</td>
<td>low</td>
<td>wave impact</td>
<td>0 - +12</td>
</tr>
<tr>
<td>17</td>
<td>cracks</td>
<td>high</td>
<td>wave impact</td>
<td>0 - +12</td>
</tr>
<tr>
<td>18</td>
<td>cave</td>
<td>low</td>
<td>?</td>
<td>+11</td>
</tr>
<tr>
<td>19</td>
<td>cantilevering rock</td>
<td>high</td>
<td>wave impact</td>
<td>0 - +16</td>
</tr>
</tbody>
</table>
With a view on the different damage kinds, heights where the damage occurs and loads on the fortress wall a division can be made. The fortress wall can be divided in a part at the northern side of the lighthouse, a part at the south of the lighthouse and a part in between.

The part at the north of the lighthouse contains the sections D and 13 to 19. A representative section for this part of the fortress wall is section 15, which is shown in Figure 2.29. In the picture the common type of damages for this part appear. At first a shear block is about to slide off the rock. This is caused by the cracks which occur in the rock foundation. The direction of most of the cracks is parallel to the slope of the rock foundation. Furthermore some erosion has occurred near mean sea level. The regular lines in the rock foundation which look like masonry are a schematization of the bedding planes. The direction determines the orientation of a possible shear plane.

Figure 2.27, Study area with damage cases noted in Table 2.17
Sections 2 to 12 represent the part of the fortress wall at the corner near the lighthouse. A common section for this part of the fortress wall is section 12, which is presented in Figure 2.30. Some similarities when compared to the previous section are the existence of cracks, their main direction, a block which tends to slide off the slope and the little erosion which is found at mean sea level. A major difference when considering both sections is the direction of the bedding planes in the rock, which in this case is less dangerous.

The third part covers the sections B and C, located at the wall of ‘El Cafetaria los 12 apostoles’. A typical damage kind for this part is shown in Figure 2.31 and Figure 2.31, where a large hole in the wall can be seen. Typical for this part is the existence of several gaps just below mean sea level. These gaps, caused by scouring of the seawater along the wall, may result in bigger holes as shown below. Furthermore wave actions can increase the velocity of the described process.
Figure 2.31, damage of 'los 12 apostoles'

However not all damage cases can be described with the previous models. In the northern part of the project area the sections 18 and D do not fit with the expectation. In Figure 2.33 and Figure 2.34 these sections are shown respectively. Section D seems to be the result of scouring, which might be the consequence of its position. The cause of the damage in section 18 is hard to define.

Figure 2.32, section 18

Figure 2.33 section D

Drawings of all sections can be found in annex X.

2.2.3 Failure mechanism

After the determination of the most important rock parameters and investigation of the different damage cases a schematization of the way of failure of the rock mass can be made. In order to investigate the present condition a proper
failure model must be defined. In the present situation cracks have already been developed so the failure model of the rock mass reduces to failure of the cracks caused by a static shear load.

![Failure model diagram](image)

**Figure 2.34, failure model**

In Figure 2.35 the failure model of the rock mass is depicted. The moment when the block will slide of the rock can be determined by a Coulomb model. In this case the cohesion and internal fraction angle of the crack on the shear plane are the input parameters. The cracks are either empty or filled with limestone and sand. In reference [xxx (spaans rapport 1989)] the friction parameters of these cracks have been determined. The results of this are shown in Table 2.18, crack properties, in which it can be seen that the internal friction angle varies between 23° and 40°. A remark on this is that it is very difficult to get an accurate estimation of these values, so they have to be taken into account carefully. It may be clear that there is no cohesion in the cracks as they are either empty or filled with loose material.

<table>
<thead>
<tr>
<th>size of cracks</th>
<th>limestone filling</th>
<th>sandy filling</th>
<th>empty</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 0.5 mm</td>
<td>30°</td>
<td>36°</td>
<td>40°</td>
</tr>
<tr>
<td>0.5 – 3 mm</td>
<td>25°</td>
<td>33°</td>
<td>32°</td>
</tr>
<tr>
<td>&gt; 3 mm</td>
<td>23°</td>
<td>30°</td>
<td>30°</td>
</tr>
</tbody>
</table>

With the known crack material parameters the Coulomb envelope can be drawn (Figure 2.36).

![Coulomb envelope](image)

**Figure 2.35, Coulomb failure envelope**
When a safety factor is defined as the loading divided by the capacity this factor can be computed by combining the Coulomb envelope with the slope of the shear plane. In Figure 2.37 a plot is given of the safety factor for different slopes of the rock wall. In this analysis the shape and roughness of the crack are neglected. To take these characteristics into account a very detailed investigation has to be done which is beyond the scope of this project. Furthermore by not increasing the friction due to irregular shapes of cracks a conservative approach is made.

According to the previously described approach the safety factors for all damage cases with blocks which are about to slide off can be determined. These blocks are numbered and their safety is represented in Table 2.19.

<table>
<thead>
<tr>
<th>block</th>
<th>section</th>
<th>slope</th>
<th>safety 20</th>
<th>safety 30</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>2</td>
<td>63</td>
<td>0.19</td>
<td>0.29</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>47</td>
<td>0.34</td>
<td>0.54</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
<td>76</td>
<td>0.09</td>
<td>0.14</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>76</td>
<td>0.09</td>
<td>0.14</td>
</tr>
<tr>
<td>7</td>
<td>8</td>
<td>67</td>
<td>0.15</td>
<td>0.25</td>
</tr>
<tr>
<td>7</td>
<td>9</td>
<td>62</td>
<td>0.19</td>
<td>0.31</td>
</tr>
<tr>
<td>7</td>
<td>10</td>
<td>60</td>
<td>0.21</td>
<td>0.33</td>
</tr>
<tr>
<td>8</td>
<td>11</td>
<td>56</td>
<td>0.25</td>
<td>0.39</td>
</tr>
<tr>
<td>9</td>
<td>12</td>
<td>62</td>
<td>0.19</td>
<td>0.31</td>
</tr>
<tr>
<td>10a</td>
<td>13</td>
<td>34</td>
<td>0.54</td>
<td>0.86</td>
</tr>
<tr>
<td>10b</td>
<td>13</td>
<td>64</td>
<td>0.18</td>
<td>0.28</td>
</tr>
<tr>
<td>10c</td>
<td>13</td>
<td>87</td>
<td>0.02</td>
<td>0.03</td>
</tr>
<tr>
<td>11</td>
<td>14</td>
<td>26</td>
<td>0.75</td>
<td>1.18</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>17</td>
<td>1.19</td>
<td>1.89</td>
</tr>
<tr>
<td>13</td>
<td>16</td>
<td>21</td>
<td>0.95</td>
<td>1.50</td>
</tr>
<tr>
<td>16</td>
<td>19</td>
<td>39</td>
<td>0.45</td>
<td>0.71</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>85</td>
<td>0.03</td>
<td>0.05</td>
</tr>
</tbody>
</table>

As can be seen in the table, most of the safety factors have unacceptable low values. This is very unrealistic, because it is a description of the present situation. With the knowledge that the considered blocks are still at their position the model is incomplete. To cope with this shortcoming of the model a new phenomenon has to be taken into account. In the new model the shear resistance is not only caused by friction, but is also caused by some other kind of shear capacity known as the apparent cohesion. This may be caused by the shape of the crack and influenced by the crack width. Detailed investigation is made of each separate situation, from which Table 2.20 is formed. It should be noted that the apparent cohesion as been shown in this table is the minimum value of this as it is the difference between the shear load and the friction of existing situations.
Table 2.20, crack data of damage cases

<table>
<thead>
<tr>
<th>block</th>
<th>slope angle</th>
<th>form</th>
<th>width</th>
<th>fill</th>
<th>phi</th>
<th>friction</th>
<th>apparent cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>60</td>
<td>undulating</td>
<td>4 cm</td>
<td>limestone</td>
<td>20</td>
<td>5.48</td>
<td>20.59</td>
</tr>
<tr>
<td>4</td>
<td>47</td>
<td>plane</td>
<td>1 cm</td>
<td>limestone</td>
<td>25</td>
<td>2.95</td>
<td>3.84</td>
</tr>
<tr>
<td>5</td>
<td>76</td>
<td>undulating</td>
<td>&lt; 1 cm</td>
<td>limestone</td>
<td>25</td>
<td>1.72</td>
<td>13.07</td>
</tr>
<tr>
<td>7a</td>
<td>70</td>
<td>semi undulating</td>
<td>0 - 1 cm</td>
<td>limestone</td>
<td>25</td>
<td>2.46</td>
<td>12.02</td>
</tr>
<tr>
<td>7b</td>
<td>70</td>
<td>semi undulating</td>
<td>0 - 1 cm</td>
<td>limestone</td>
<td>25</td>
<td>1.51</td>
<td>7.38</td>
</tr>
<tr>
<td>8</td>
<td>60</td>
<td>semi undulating</td>
<td>&lt; 1 cm</td>
<td>limestone</td>
<td>25</td>
<td>4.75</td>
<td>12.91</td>
</tr>
<tr>
<td>9</td>
<td>62</td>
<td>semi undulating</td>
<td>≤ 1 cm</td>
<td>limestone</td>
<td>25</td>
<td>6.13</td>
<td>18.59</td>
</tr>
<tr>
<td>10</td>
<td>86</td>
<td>plane</td>
<td>0.1 - 0.5 cm</td>
<td>limestone</td>
<td>25</td>
<td>0.76</td>
<td>22.48</td>
</tr>
<tr>
<td>11</td>
<td>30</td>
<td>irregular</td>
<td>1 - 3 cm</td>
<td>limestone</td>
<td>25</td>
<td>4.70</td>
<td>1.12</td>
</tr>
<tr>
<td>12</td>
<td>17</td>
<td></td>
<td></td>
<td></td>
<td>23</td>
<td>21.31</td>
<td>0</td>
</tr>
<tr>
<td>13</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td>6.38</td>
<td>0.35</td>
</tr>
<tr>
<td>16</td>
<td>39</td>
<td>irregular</td>
<td>1 - 3 cm</td>
<td>limestone</td>
<td>25</td>
<td>7.07</td>
<td>5.21</td>
</tr>
<tr>
<td>18</td>
<td>85</td>
<td>irregular</td>
<td>6 - 15 cm</td>
<td>limestone</td>
<td>20</td>
<td>0.49</td>
<td>14.82</td>
</tr>
</tbody>
</table>

The fact that all the considered cracks are filled with limestone can be seen as a result of the low shear resistance of these cracks due to their low angle of internal friction. Furthermore it can be concluded that the apparent cohesion of the cracks is of the same magnitude as the tension strength of the rock. In Figure 2.38 the resulting extra capacity versus the slope angle is plotted for each separate case.

Figure 2.37, measured apparent cohesion

From Figure 2.38 a certain threshold value for the apparent cohesion has to be determined. Due to the fact that the internal friction of the filling is at least 20°, at gentler slopes than 20° the friction capacity is always larger than the acting sliding force. At a slope angle of 90 degrees the shear block has a full dependency on the apparent cohesion due to the absence of friction at this angle. The regression line as drawn in Figure 2.38 shows the mean extra capacity which is in use at different slope angles. This line is about 0 at a slope of 20° which coincides with the expectation. Since at 90° the blocks are fully dependent on the apparent cohesion the threshold value is determined as the value of the regression line at 90° which in this case is 20 kPa.

Due to very large uncertainty of this phenomenon a large safety has to be taken into account. Besides this safety a safety factor of 1.1 has been applied on the acting gravity force and a safety factor of 1.2 on the resistance by friction. In Table 2.21 an overview is given of the dangerous damage cases at different safety levels of the apparent cohesion. The safety level is defined as:

\[
C_{a,d} = \frac{C_{a,rep}}{\gamma_m} \quad (2.9)
\]
Table 2.21, safety factor of the apparent cohesion

<table>
<thead>
<tr>
<th>$\gamma_m$</th>
<th>cases with insufficient strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
</tr>
</tbody>
</table>

From the previous table the sensitivity of the number of cases which should be repaired to the safety factor used can be determined. It seems that a safety factor larger than four has no influence on the number of damage cases to be repaired, so it seems reasonable to include a safety factor of four. It should be noted that in total 13 damage cases have been considered.

Next to the gravity force which is acting on the rock, a consideration has to be made concerning the wave impact forces. In the paragraph 2.1.11 to paragraph 2.1.13 a description has been given of the Goda-Takahashi method which gives the acting pressure of a wave on the wall of different parts of the study area. A calculation has been made to determine which blocks have sufficient resistance against any movement given the Goda-Takahashi wave loads. In this calculation only the wave pressure which is directly acting on a block is taken into account, which results in a force parallel to the slope of the rock. This force has to be in equilibrium with the acting gravity force, the friction resistance and the extra shearing capacity to ensure sufficient stability. A scheme of the calculation is given in Figure 2.38.

Table 2.22, influence of impact factor on damage cases

<table>
<thead>
<tr>
<th>impact factor</th>
<th>cases with insufficient strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>12</td>
<td>10</td>
</tr>
</tbody>
</table>

From Table 2.22 can be concluded that an impact factor of 8 gives sufficient safety considering the damage cases of the project area. This factor is considerably lower than the factor 13 determined by the TAW model. Due to the fact that this...
wave impact has a very short duration, it will only be taken into account when considering the crack formation. When calculating a certain reinforcement of the structure the only the gravity and quasi-static wave loads.

2.3 Organisations involved

Because of the many social and cultural values of the fortress and its position in the city of Havana several different organizations have influence on the project area. Therefore it’s of use to gain insight in the structure and the different demands of these organizations in order to get an appropriate design. First of all the owner of the fortress area is of great importance. In this case this concerns the local authorities of Havana which are represented by the “Historiador de la Ciudad”. The second organization involved is the maritime museum that is located in the fortress. Due to the fact that tourism is one of the highest sources of income in Cuba, it is of great importance to the tourist industry that the museum and the surroundings of the fortress remain attractive. Another association connected with the cultural value of the fortress is UNESCO, which has put the fortress on its World heritage list.

Now the rock foundation of the fortress has to be protected the museum has contacted a contractor. This company was already involved in an earlier stage when they had filled the existing cracks and holes with mortar. The contractor at his turn asked the CUIAE University to design a sustainable solution for the problem mentioned.

Finally the port authorities are of importance when considering the hindrance in the channel caused by construction works. Besides they have several demands concerning the size of and wave conditions in the channel. The last and minor user is the coastguard, which runs the lighthouse.

Figure 2.39, scheme of organisations involved
3 Problem description

3.1 Problem analysis

3.1.1 Current state of the fortress
In the course of time the rock foundation underneath the fortress wall suffers from erosion and cracks have developed. Without effective measures this would cause irreparable damage to the fortress and could even lead to collapse of the lighthouse. The present plans to protect the rock foundation are focused on filling the cracks with mortar. One could ask whether this is a durable solution.

3.1.2 Current progress of the project
To make the reparation on the fortress durable the first preparations have been made by GeoCuba to analyze the situation. Subjects that already have been investigated are; the bathymetry along the fortress, the wind and wave data at the fortress, the cracks and caves in the rock foundation. A number of cracks has already been filled with mortar. Before the start of the realization of the solution formulated in this study all the cracks will be filled.

3.1.3 Problem definition
Erosion and development of cracks weaken the rock foundation of “El Castillo del Morro”.

3.2 Project objective
Find a sustainable solution in order to minimize the erosion of the foundation of “El Castillo del Morro”.

3.3 Scope of the project

- Check earlier studies
- Elaborate on earlier investigations
- Analysis of the source of erosion
- Create concepts on basis of the hydraulic system
- Carry out a study of wave transformation
- Elaborate concepts on hydraulic and structural behaviour
- Work out some ways of realization
- Choice and detailing of the most appropriate alternative
- Advice on repair and maintenance
4 Boundary conditions and assumptions

4.1 Boundary conditions

4.1.1 Functional conditions

- The entrance to the bay of Havana should have sufficient room for ships to enter, manoeuvre and exit this bay, which gives a limitation to the size and position of a solution. Furthermore, the structure should not develop reflective or refractive waves which could affect the easy trespassing of ships.
- A solution to the problem has to be designed such that it should not harm the cultural value of the fortress, which means that under normal conditions it remains invisible and is no aesthetic obstacle when looking at the fortress.

4.1.2 Technical conditions

- The natural material of the bottom is solid rock; the basement of the fortress consists of this solid rock together with a protection wall which was built in the 16th century. The protection wall consists of two masonry walls with rocky filling in between.
- The altitudes of the project area's surroundings is given by the bathymetrical map which is depicted in annex X.
- It is not possible to obtain US patented structures, like breakwater parts.

4.1.3 Meteorological conditions

One of the most important groups of boundary conditions concerns the meteorological situation. In the chapter 3, Problem description a brief elaboration of the climate was given, which has been used to describe the local weather and wave conditions. These conditions resulted in a maximum value of the water- and wave height connected with a certain return period. In the following tables respectively the wave heights, sea level inclinations and tidal information are given.

### Table 4.1, wave data

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Wave height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.2 (T = 6.2 s)</td>
</tr>
<tr>
<td>25</td>
<td>6.7 (T = 9.2 s)</td>
</tr>
<tr>
<td>50</td>
<td>7.6 (T = 11.2 s)</td>
</tr>
<tr>
<td>100</td>
<td>8.6 (T = 12.2 s)</td>
</tr>
<tr>
<td>200</td>
<td>9.6 (T = 12.9 s)</td>
</tr>
</tbody>
</table>

### Table 4.2, sea level inclinations

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>Wind setup (m)</th>
<th>Wave setup (m)</th>
<th>Pressure rise (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.35</td>
<td>0.26</td>
</tr>
<tr>
<td>25</td>
<td>0.02</td>
<td>1.04</td>
<td>0.26</td>
</tr>
<tr>
<td>50</td>
<td>0.04</td>
<td>1.20</td>
<td>0.26</td>
</tr>
<tr>
<td>100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4.3, tidal information

<table>
<thead>
<tr>
<th>Phenomenon</th>
<th>Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>average amplitude</td>
<td>0.28</td>
</tr>
<tr>
<td>maximum rise above mean sea level</td>
<td>0.39</td>
</tr>
</tbody>
</table>

4.1.4 Environmental conditions

- The harm of the solution to the local biological system should be minimized.
- Problems regarding the water quality should be avoided by choosing the appropriate materials and execution methods.
4.1.5 Execution conditions
- Due to the political situation in Cuba, very few foreign contractors act in this country. Therefore a solution to the problem should be possible to realize by local companies or companies from countries which have a good political relation with Cuba. This gives a limitation to the available equipment and knowledge of execution processes.
- For similar reasons, solutions made of local available materials are favourable in comparison with those with imported materials.

4.1.6 Juridical conditions
- The fortress is owned by the Cuban authorities, which explains the large influence this institution has on the demands and decision on which solutions eventually will be executed.
- Another important organisation is UNESCO. Because of the fact that the fortress belongs to the world heritage list, this influence is present.
- The harbour authorities have some demands concerning the trespassing of the bay which are mentioned earlier in the requirements regarding the function and execution of the solution.
- The design of the solution should be checked according to Cuban building requirements. Besides, the execution has to be done according to these regulations. When no Cuban requirements are available, the Dutch will be used assuming that they suffice to the Cuban ones.

4.1.7 Economical conditions
- Due to the economic situation in Cuba, the financial means are limited. In order to have a well-executed solution, it has to be cost-extensive. Taking into account the high price and low availability of materials in comparison with labour, a well-designed solution is material-extensive and labour-intensive.

4.2 Basic assumptions

4.2.1 Technical / hydrological
- Data concerning wave heights and sea levels obtained from Global wave statistics and local weather stations are correct and can be used for further calculation.

4.2.2 Currents

4.2.3 Functional
- Although the fortress needs to be repaired, only very small and very few adjustments of the structure are allowed to make because of the UNESCO world heritage regulations.
- The solution has to concentrate on the places of the wall where a lot of damage is present.
- During storms no shipment is present in the entrance of the bay and its surroundings.

4.2.4 Execution
- During construction the ships heading to or from the harbour in the Havana bay should be capable of passing the construction area without any hindrance. If any hindrance cannot be prevented it should be minimized and an agreement with the port authorities should be made in order to prevent any problems or financial claims.

4.2.5 Economical
- The integral costs of a solution should be minimized. The final solution has to be chosen according to this, which means that the appropriate solution is the one which has the lowest costs concerning both construction and maintenance.
- Funds will be made available by UNESCO, which explains their influence on the project and regulates things like tendering, execution and forms of cooperation between a contractor and the Cuban authorities.

4.3 Further assumptions
- During normal weather conditions, the waves and currents in the Bay’s entrance are such that the construction of a solution can be done without any difficulties.
- The waves developed by ships are of no significant height and can therefore be neglected.
The material from which the foundation of the fortress is formed is rocky, and can be considered as homogeneous.
5 List of demands

- A reduction of the return period of the significant wave height has to be made such that the lifetime costs of the repair of the fortress and its foundation are minimized.
- Scouring of the rock foundation must be negligible.
- The lifetime of the reparation of the fortress wall and the rock foundation has to be 100 years.
- In the figure below the minimum maneuvering space is shown. This picture is based on the actual size of the ships that enter the port of Havana, and takes into account a future growth of the ship sizes.
- The approach cannal is 220 meters wide; a ship of 30 m must still be able to enter the port. Since the buoys which indicate the save depth of the channel are 50 m out of the shoreline the construction may reach at most 40 meters out of the bank.
- Offshore constructions have to be of an unnoticeable size.
- Adaptations to the fortress and its foundation should not harm its historical view.
- The construction activities must leave the fortress free of damage.
- Design and execution have to be done according to Cuban building and construction regulations.
6 Concepts

6.1 Introduction

To solve the erosion problems there are five types of actions that can be taken. First a zero-alternative, in which no action is taken, should be considered. This concept is not useful considering the present severe damage to the foundation. The second action is to take away the cause of the problem which is in this case the incoming wave action. Both the storms and waves are natural phenomena, so they can never totally be taken away. Furthermore sediment could be supplied, but when considering this coast line this is not applicable. Finally the loads can be reduced and the strength of the structure can be increased. These two actions are most appropriate for this situation and lead to possible concepts to solve the problem. These are:

- Berm
- Seawall in front of the fortress
- Low-crested structures
- Caissons
- Revetment on the foundation
- Strengthening of the existing construction

In this chapter all the concepts will be treated briefly and finally some are selected for further elaboration.

6.2 Berm

A berm is a massive tableland attached to the shoreline near the water level; this is shown in Figure 6.1. They are often applied at dikes to reduce the wave run-up. Because of the shallow water the waves break on the berm instead of the shoreline itself. The most effective position for a berm is at the design water level. It can be built by different materials like rock, concrete or soil with an armour layer.

![Figure 6.1, scheme of a berm](image)

The influence of a berm mainly depends on the width and the water depth above it. The formulae available are empirical and based on Dutch dikes, therefore they can only be used for global insight and not for a detailed calculation. At first a sensitivity analysis is made for the berm width and the wave run-up at different wave heights, which are plotted in Figure 6.2. Some notes have to be made for this analysis.

These calculations have been made for a horizontal berm just below the water level. The steep slope of the bottom and a vertical wall behind the berm cannot be applied, because of the Dutch character of the formulae. Due to the more gentle slopes and the absence of a wall in the Dutch formulae, the estimated run-up is higher than will occur. Therefore the berm can be of less width. Behind the berm there is hardly any run-up since there is a vertical wall. This analysis only shows that the reduction factor is linear and parallel for different wave heights. Another point is that a berm makes a significant reduction of the wave height at the shoreline.
Another approach for estimating the dimensions is given by the equivalent slope principle [Ref XXX]. The results are set out in Figure 6.3. This model can handle the steeper slopes, which directly results in a smaller berm.

A conclusion which can be drawn from these analyses is that a berm is an effective tool to reduce the wave height at the shoreline. The reduction factor does not depend on the wave height. This leads to a design criterion based on the largest waves occurring at the location. According to Table 2.9, wave parameters at the northern part, this leads to a horizontal berm at the water level has to be 4 meter wide at the northern cross-section. For the southern cross-section according to the data given in paragraph 2.1.12 a berm of 2.3 meter should be sufficient. With these dimensions and the shallow water at the north- and south side little material has to be used to construct a massive berm.

Another amenity is the flexibility of the concept. Beside the many building materials which can be used, it can be adjusted to every separate location of the shoreline without affecting each other. This is a big advantage according to the different sections around the fortress. A disadvantage of the concept is the visibility, the design water level is above the mean sea level, and this means the berm can be seen most of the time. This can be taken care of by lowering the berm, but this will lead to an increase of the width. Another disadvantage is the load to which the edge of the berm is exposed, which can lead to maintenance and scouring just in front of the structure. All this can be taken care of by applying quite simple adjustments.

6.3 Seawall

This concept contains a slender wall, erected some distance out the shoreline. It can be compared with a quay wall, only there is no soil at the back. A sketch of a wall in sea is given in Figure 6.4. The foundation of the wall is placed into the rock of the bottom. The wall reaches up to just below the water level, so it cannot be seen from the shore.
Figure 6.4, scheme of an offshore sea wall

The wall is not permeable for the ellipse shaped wave movement, which will cause reduce of the wave movement. The waves either break on the wall or will be highly reduced in height. Behind the wall some space should be left for plunging water actions. Behind this zone there will be hardly any wave action. This part should not be too wide either, in order to avoid new waves to build up. To create these zones the wall has to be positioned approximately 30 m offshore.

This concept has a couple of advantages, which at first is the possible elimination of the total wave action. The next is the little space it needs; a vertical wall covers less than a sloped breakwater. The disadvantages are numerous, some will be treated. A vertical wall will create a standing wave in front, which can reach twice the height of the incoming. This implies a completely new wave action in the surroundings; this has mayor consequences for the shipping and the ecology. Furthermore the foundation has to be strong, not only to handle the waves but also to withstand the pressure gradient between the front and the back of the wall. This pressure gradient develops due to the impermeability and different water levels. Not to mention the difficulties to construct a vertical wall in these wave conditions. This wall will have wave diffraction at the sides, which induces new wave actions.

Due to the large amount of disadvantages this concept is not very suitable for application, although most of these disadvantages can be solved by placing a rumble mount slope in front and making the wall more stable.

6.4 Low-crested structures

These structures can be divided in two different types of breakwaters, namely the reef breakwater and the submerged breakwater. A reef breakwater is a low-crested dynamically stable structure. It consists of a mound of homogeneous stones with no core or filter layer in the structure. After construction the incoming waves will reshape the structure until equilibrium and the final profile is reached. Depending on the stone characteristics the final profile will be submerged or not. A schematization of a reef barrier is indicated in Figure 6.5. The best way to construct a reef breakwater is with dump barges.

Figure 6.5, scheme of a reef breakwater

The submerged breakwater is statically stable. In contrast to the reef breakwater, the submerged breakwater has a core of finer materials and armour layers. Considering the continuing overtopping of the waves, also the lee-side needs a sufficient armour layer. A submerged breakwater is indicated in Figure 6.6. Crane barges are required to construct the breakwater.
Figure 6.6, submerged breakwater

The crest height of both breakwaters is determined by the required wave reduction. Large waves will break on the slope of the breakwater and so there wave energy will be dissipated. Waves which do not break will also lose energy as a result of the transmission capacity of the breakwater. This transmission capacity depends mainly on the structure geometry, crest freeboard ($R_c$), crest width, water depth, permeability and the wave conditions. These breakwater characteristics differ between a reef breakwater and a submerged one. For example, the permeability of a reef breakwater is higher than for a submerged one. A reef barrier also deforms after its construction stage, which results in an increase of the transmission capacity. But to get a first estimation of the crest freeboard and the corresponding wave reduction, both breakwaters (in their final profile) will be considered equally.

From various test results on transmission through and over low-crested breakwaters, van de Meer derived a single prediction method [ref.xx] to calculate the transmission coefficient ($C_t$). With this coefficient the reduced wave height behind the breakwater can be estimated. This only yields for non-breaking waves. High waves which break on the slope of the breakwater or before, already lose their energy due to dissipation. Thus the transmission capacity of the breakwater is only valid for non-breaking waves. In Table 6.1, the sensitivity of the reduced wave height in relation with the crest freeboard is given. The reduced wave height depends on the waves that will not break before arriving at the crest of the breakwater.

<table>
<thead>
<tr>
<th>$R_c$ (m)</th>
<th>$\gamma$</th>
<th>$H_r$ (m)</th>
<th>$R_c/H_r$</th>
<th>$C_t$</th>
<th>$E_{before\ bw}$ (J/m$^2$)</th>
<th>$E_{after\ bw}$ (J/m$^2$)</th>
<th>$H_{reduced}$ (m)</th>
<th>Reduction $H$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1</td>
<td>0.99</td>
<td>0.99</td>
<td>-0.12</td>
<td>0.49</td>
<td>1232</td>
<td>610</td>
<td>0.70</td>
<td>30</td>
</tr>
<tr>
<td>-2</td>
<td>0.99</td>
<td>1.98</td>
<td>-0.23</td>
<td>0.53</td>
<td>4928</td>
<td>2610</td>
<td>1.44</td>
<td>27</td>
</tr>
<tr>
<td>-3</td>
<td>0.99</td>
<td>2.97</td>
<td>-0.35</td>
<td>0.56</td>
<td>11087</td>
<td>6260</td>
<td>2.23</td>
<td>25</td>
</tr>
<tr>
<td>-4</td>
<td>0.99</td>
<td>3.96</td>
<td>-0.47</td>
<td>0.60</td>
<td>19710</td>
<td>11817</td>
<td>3.07</td>
<td>23</td>
</tr>
<tr>
<td>-5</td>
<td>0.99</td>
<td>4.95</td>
<td>-0.58</td>
<td>0.63</td>
<td>30797</td>
<td>19538</td>
<td>3.94</td>
<td>20</td>
</tr>
</tbody>
</table>

A breakwater causes two processes of energy dissipation, namely wave breaking and wave transmission. According to the table, wave breaking is the most effective way to dissipate energy. A larger crest height, or smaller crest freeboard, will lead to a lower wave that can reach the crest without breaking. More waves will break on the slopes of the breakwater and dissipate their energy if the crest height is higher. The percentage of the reduction of the wave height due to transmission through and over the breakwater becomes larger at a smaller crest height.

For a first estimation we assume that the breakwater is situated 40 m offshore. According to the bathymetry (see chapter 2.1.6) the water depth 40 m offshore is around 6.8 m on the north side, 2.8 m on the south side and 11.1m in the corner (section 3).

The crest width of the submerged breakwater must be sufficient to allow at least three stones to fit. As a first approximation a crest width of 3 m is taken. The slope angle of the structure equals the maximum natural slope of dumped material under water, which is 1:1.2. For different values of crest the freeboard, the cross-sectional area of the breakwater can be compared with each other. This may be helpful to get a clear image of the needed materials and so the costs of the structure in relation with the required reduced wave height. This is indicated in Figure 6.7 for a breakwater at the north side, the south side and at the corner. Also the fact that the stability of the submerged breakwater increases as the crest height decreases, will have an influence of the choice of the crest height to be taken.
The dimensions of the reef breakwater in its final shape depend on the deformations after the construction shape and thus depend on the used stones. As a first rough estimation the cross-sectional area of the reef breakwater is supposed to be equal to the cross-sectional area of the submerged breakwater.

The dynamically stable reef breakwater is an effective way to reduce the wave height. It has the advantages that the construction can be accomplished by free dumping of stones from a dump barge. In comparison with the submerged breakwater the weight of the individual stones is smaller. A disadvantage exists when oblique waves attack the structure. Long shore then current might occur, which can affect the stability of the reef breakwater. Predicting the behaviour of the reef breakwater is very complicated considering the very few design formulae and the fact that only empirical data is available. Another disadvantage could be the availability of a large amount of big stones with good quality. In case of the submerged breakwater only the exterior armour layer consists of big stones.

The submerged breakwater also is an effective way to reduce the wave heights. It is always situated below water level, which is not always the case with the reef barrier. The armour layer can be made of stones, although in some cases the required diameter of the stones is too large to ensure a stable armour layer. In these cases artificial elements, such as tetrapods and acropods, can be used to create a more stable armour layer.

### 6.5 Caissons

Caissons are large hollow concrete blocks which have to be placed side by side in order to create a dam. The breakwater types have found wide applications at many locations. They are of good use for closure works in areas where rock is scarce. The Figure 6.8 gives an impression of a caisson which can be applied here. A caisson is placed by shipping equipment to the final spot and filled with soil or is inundated to sink. Normally they reach above the water level, but that is not allowed in this case. The caisson has to be placed on a horizontal bed. This foundation layer is of great importance for the stability and strength of the structure.
This foundation layer can be made of rip-rap stones, which has the best capacity to flatten out the bottom. This plane is needed for an equal distribution of the load and weight to avoid crack formation in the caisson. Also the most failure modes are related to the foundation, namely sliding backward and tilting.

Sliding backwards is caused by horizontal wave loads at the front. This happens when they exceed the friction resistance at the interface between caisson and foundation. Tilting is a backward rotation around a point at the surface. It effectively means that the stones are crushed due to an increase of stress at that point.

The horizontal and upward loads on the caisson can best be determined by the method of Goda, the same as in chapter 2.1.11 where the pressure on the shoreline is calculated. The friction resistance between the caisson and the foundation depends strongly on the mass of the caisson. The shear stress, according to the design formula by Coulomb, is determined by the normal stress multiplied with the tangent of two third of the angle of internal friction. As the area of the interface is kept constant, the friction force increases linearly with the mass of the caisson. After sinking the caisson onto its place extra mass can be added by filling the hollow sections with sand or rubble if necessary. Beside the magnitude of the mass, the position of its resulting force is an important variable to prevent the caisson from tilting. The foundation itself has to be stable too, especially the toe. It must be free of erosion and settlement, terms which can be satisfied without a lot of problems at a rock bottom.

Normally, when a caisson is placed at an intermediate depth, waves are not jet broken before they arrive at the caisson. With the vertical wall, a standing wave can occur in front of the wall instead of a breaking one. Because standing waves do not have a big peak load at impact, this strongly decreases the forces acting on the caisson. In this case an irregular type is applied, because it is submerged. This difference will probably cause waves to break on top of the caisson.

Stones and concrete are both available, so this is not an important reason to prefer the caissons. An advantage of the caisson is the stability of the shape. Presuming the caisson is well designed, a very great lifespan can be expected. Another advantage is the method of construction. The concrete blocks can easily be built in a dry dock. This prefab character also implies that they can be placed quickly. At the open sea favourable working weather can be scarce. Even though aspects of realization are favourable because of the visibility of working onshore, there are also aspects with great difficulty. Creating a foundation out of rip rap stones has to be done very precisely. Special equipment has to be used for making such a construction.

Although the structure will be submerged the waves which break on the caisson will be noticeable onshore. The spectacular sight of wave action at the shore will also belong to the past, and can be considered a disadvantage. The reflecting waves at the seaside of the caisson could cause some hindrance to the shipping. Although the consequences due to this effect are not predicted as very serious.

### 6.6 Revetment

Normally a lot of different types of revetment can be applied. For this case however the bottom profile is very steep, which limits the possibilities. Secondly, a lot of revetments are designed for the protection of fine materials. With a rock bottom in this case, this is not the objective. More options seem to fail because of the irregular shape of the rock foundation. Types of revetments that are adequate should either be plastic when brought into placed and harden after, or be piled in front of the foundation. Both concrete and asphalt can be considered as plastic materials to harden at the surface of the foundation to form the revetment. The list of demands requires an unnoticeable solution. So applying an asphalt layer on the rock foundation is out of the question. The use of concrete on the other hand can blend in with its...
surroundings when using the right compositions and put on properly. That this can be done in a sufficient way has been proved on earlier restorations just south-east of the 12 Apostolos.

Figure 6.9, previously repaired section

Although the natural subsoil does not consist of loose rocks, the use of a piled revetment can have a natural look if the right stones will be used. In order to get a stable solution a revetment may also be built out of more than one component. For example a firm concrete submerged part of the revetment with natural rocks on top. This gives a natural look, so the revetment remains unnoticeable. In Figure 6.10 the both revetments are given.

Figure 6.10, concrete revetment (a), piled revetment (b)

A revetment protects the rock foundation in two different ways. By having an armoured layer, incomings waves do not have a direct impact on the foundation. Secondly the scour on the rock foundation will be reduced by the revetment. With a small concrete cover the pressure reduction on the foundation will only be minimal. The way the pressure interacts on the foundation may give a big difference. Because pressure propagation into a crack may introduce tension stresses, the concrete revetment may prevent the pressure propagation and thereby further development of cracks, as indicated in Figure 6.11a. This effect depends strongly on the permeability of the revetment. Also the crack formation of the concrete is of importance, mainly because it is most likely to crack at the same places as the existing cracks in the foundation. The most important cracks however, are parallel to the rock surface. Therefore, it is uncertain if the wave pressures really propagate all the way into the cracks. In Figure 6.11b the direction of the most important cracks are indicated.
Special attention must be paid on the interaction area between the foundation and the concrete. A big disadvantage is the way in which the concrete has to be brought into place, because a part of the revetment has to be made below sea level.

Both the wave pressures and the scour will reduce with a piled revetment. Due to the piled revetment wave energy will be dissipated and the current velocities will decrease. This leads to a reduction of the wave pressure on the wall and a decrease of the erosion. The stability of the revetment depends on the dimensions, roughness, shape, density and strength of the stones. These stone characteristics can be determined in order to fulfil the requirements.

Piled revetment will reduce the pressure on the walls, because of the big capacity to dissipate the wave energy. For the execution dump barges or cranes from the shore can be used. A disadvantage of the piled revetment is the stability. The maximum possible natural slope of stones is gentler than the steep slopes just in front of the wall. Without extra precautions a lot of stones are needed to get a stable revetment.

### 6.7 Strengthening of the existing structure

This concept concerns the strengthening of the rock foundation. It is not a durable solution of the problem, because it does not take away the load. However the current state of the foundation demands such a measurement. From this point of view this concept can be seen as an extension of the current situation. Furthermore this reparation can be seen as an inevitable addition to the other concepts.

There are three reasonable possibilities to strengthen the rock foundation. These are called reasonable because they mainly suffice one of the main demands: ‘Adaptations to the fortress and its foundation should not harm its historical view’. Each damage case has to be considered separately in order to get an appropriate solution.

First of all the existing cracks could be filled with a cement mortar. This filling enables adhesion forces between the different rock layers on both sides of a crack, which increases the stability of the rock mass. This method of filling cracks requires a large pressure in order to insert the mortar. The pressure is in this case proportional to the filling length, and therefore a restriction to the length of the crack can be defined. This is due to the fact that the rock has a finite strength and therefore can cope with a certain maximum pressure. Hence crack filling according to this method has to be done with great care in order not to damage the rock mass.

Another method of reinforcing the rock mass is pouring small concrete supports on the rock surface (Figure 6.12). This should be done at spots where blocks are about to fall off the rock. This method is not very favourable because of its contradiction with the main demand described above, but due to its very local character it may be acceptable. A negative aspect of this local character is that it results in a minor strengthening of the total rock foundation. Furthermore the solution is very specific, when some erosion occurs elsewhere other supports should be added again.
A final option is to anchor parts of the rock that have a great risk of sliding off, as schematized in Figure 6.13. This anchoring should be carried out in such a way that the loose parts of the foundation are firmly attached to the more solid and lower parts of the rock mass. Although this solution also has a local character, it is not as specific as the previous option. A large disadvantage of this method is the lack of possible maintenance and inspection cannot be done easily. Furthermore the technology needed to execute a method as described is not available in Cuba and therefore foreign contractors have to execute the work.

In the following table the main aspects of the three different methods are summarized.

<table>
<thead>
<tr>
<th></th>
<th>Filling of cracks</th>
<th>Concrete support</th>
<th>Anchoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk during execution</td>
<td>high</td>
<td>low</td>
<td>low</td>
</tr>
<tr>
<td>Costs</td>
<td>high</td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>Visual hindrance</td>
<td>low</td>
<td>high</td>
<td>low</td>
</tr>
<tr>
<td>Functionality</td>
<td>medium</td>
<td>low</td>
<td>high</td>
</tr>
</tbody>
</table>

Filling of cracks and anchoring can be applied on similar situations due to their low visual hindrance and similar effects. However filling of cracks is very risky due to the uncertainty of the strength of the rock material, the shape of the cracks, their present filling and the large pressure involved in the execution. Considering this only the concepts anchoring and concrete supports will be considered.
6.8 Conclusions

In order to get the most appropriate solutions which suffice the most to the required demands a selection has to be made between the concepts. They will be verified according to the demands and from that a conclusion is taken.

As said before the concept of strengthening the existing structure is not a solution, but an addition to it. Only strengthening is not a durable solution to the problem. In order to fulfill the requirement of a lifetime of the reparation for a period of 100 years, strengthening of the foundation and a hydraulic structure are needed. The strengthening will lead to a strong foundation which is able to withstand the loads of the fortress for the required period. In order to maintain the bearing capacity of the foundation, the wave impacts on the structures must be reduced. Hydraulic structures and revetments can cope with this problem. Their main objective is to reducing the wave energy and thus the wave impact.

In order to get the best fitted hydraulic solution, the concepts treated in this chapter will be verified according to the required demands. The unnoticeable size of the structures is one of the most important demands. During normal wave conditions the structure must be invisible. This can be done by submerging the structure or to give the structure a natural look. Another important aspect is the reduction of the wave loads on the structure. This is also dependent on the available amount of money. As indicated with the demands, costs play an important role in the decision making between solutions. Also the available materials and building equipment play a role. Some materials are scarce in Cuba, which leads to unacceptable costs. Materials and building equipment also have an influence on the execution of the project. Finally the construction aspects are important, but here too costs have a major influence. Briefly, the total costs of the project should be guarded permanently. In Table 6.3 all the features of the concepts related to the demands are indicated. For background information see the previous paragraphs concerning the concepts.

Table 6.3, summary of the concepts

<table>
<thead>
<tr>
<th>Features</th>
<th>Reef breakwater</th>
<th>Submerged Breakwater</th>
<th>Seawall</th>
<th>Berm</th>
<th>Concrete revetment</th>
<th>Piled stones revetment</th>
<th>Caissons</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visibility</td>
<td>low</td>
<td>low</td>
<td>low</td>
<td>medium</td>
<td>high</td>
<td>medium</td>
<td>low</td>
</tr>
<tr>
<td>Availability</td>
<td>good</td>
<td>good</td>
<td>good</td>
<td>good</td>
<td>moderate</td>
<td>good</td>
<td>good</td>
</tr>
<tr>
<td>Availability</td>
<td>good</td>
<td>bad</td>
<td>good</td>
<td>moderate</td>
<td>good</td>
<td>good</td>
<td>moderate</td>
</tr>
<tr>
<td>Wave load</td>
<td>good</td>
<td>good</td>
<td>good</td>
<td>low</td>
<td>low</td>
<td>low</td>
<td>good</td>
</tr>
<tr>
<td>Execution</td>
<td>easy</td>
<td>moderate</td>
<td>difficult</td>
<td>easy</td>
<td>difficult</td>
<td>easy</td>
<td>difficult</td>
</tr>
<tr>
<td>Stability</td>
<td>moderate</td>
<td>good</td>
<td>good</td>
<td>medium</td>
<td>moderate</td>
<td>moderate</td>
<td>good</td>
</tr>
<tr>
<td>Costs1</td>
<td>acceptable</td>
<td>acceptable</td>
<td>unacceptable</td>
<td>acceptable</td>
<td>unacceptable</td>
<td>acceptable</td>
<td>moderate</td>
</tr>
</tbody>
</table>

1Global costs of the structure to fulfil all the requirements. It is dependent of all the previous demands, to which the structure must suffice. They are relative costs compared with the benefits of the structure.

For concrete revetment and the seawall the costs are unacceptable for the obtained benefits. These two concepts will not fulfill the financial aspects of the problem. And therefore these two concepts will not be further discussed. This also yields for the caisson-type of breakwaters. The ratio costs against benefit are not optimal and besides this the difficulties that occur during execution can cause severe problems. Also the local lack of experience of caissons is a problem.

A solution must be sought for the north side, south side and for section 3. The different bathymetries and different wave impacts of these sections require different solutions. All the concepts respond different to the different bathymetry and waves. These responses and features are indicated in Table 6.4.

Table 6.4, responses and features of structures to bathymetry and waves

<table>
<thead>
<tr>
<th>Features</th>
<th>Reef breakwater</th>
<th>Submerged breakwater</th>
<th>Berm</th>
<th>Piled stones revetment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Energy dissipation by wave breaking</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>transmission</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>direct protection</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Frontal wave response</td>
<td>stable</td>
<td>stable</td>
<td>stable</td>
<td>stable</td>
</tr>
<tr>
<td>Oblique wave response</td>
<td>stability problems</td>
<td>energy dissipation</td>
<td>energy dissipation</td>
<td>energy dissipation</td>
</tr>
</tbody>
</table>

To find the best solution for the different parts of the study area, these will be treated separately.
The north side is characterized due to deep water and high oblique and frontal waves. To reduce the high wave energy, wave breaking must take place to dissipate energy. A revetment is in this case not suitable, due to the high wave forces and the steep slope in front of the fortress. A large width is necessary to get a stable revetment. This is in fact the same as a berm, which is a suitable solution for the problem and will be further elaborated. A submerged and reef breakwater are also suitable in this case. However the reef breakwater can get problems due to the oblique incoming waves. This problem will be researched further during the project.

Very shallow water and waves from different directions are the characteristics of the south side. Due to the presence of the approach channel to the harbour of Havana, the building area is restricted. These facts make the application of a breakwater less suitable for this section. The required building area reaches up to 40m offshore. According to the bathymetry of the south-side, the depth 40 m offshore varies between 2 and 3m. This is not deep enough for dump barges to construct breakwaters. A possible solution with a breakwater may be realized when constructing attached to the Malecon at the other side of the bay. However this option is not suitable to protect against waves from different directions, because an unacceptable large breakwater is needed to protect the south side of the fortress. The berm and the piled stones revetment will be further elaborated.

The corner section is the transition area between the north and south side. The depth in front of the fortress is very big and varies between 5.5 and 11m. The same reasoning as for the north side can be applied in this section. The slopes are even steeper and the depth is bigger just in front of the foundation. So, the same solutions as in the case of the north side can be applied here, namely a submerged breakwater, reef breakwater and a berm. A disadvantage of a breakwater in this case is the removal of the impressive sight of breaking waves in front of the lighthouse during storm conditions.
7 Structural elaborations

7.1 Introduction

7.2 Solutions applicable to the north side

7.2.1 Breakwater, general aspects

7.2.1.1 Stability

A breakwater can be classified by some keywords as: rip-rap, open and permeable. The first to be determined for the stability of the breakwater is the size of the armour layer. This layer has to protect all the other layers against erosion. Behind the armour layer a filter must be build in order to protect the core.

Two types of methods are used to determine the rock size, Hudson (7.1) and Van der Meer (7.2). Both formulae are given below and are worked out and filled trough the chapter. Hudson (7.1) is highly empirical and based on the momentum of a single particle. It is based on the drag force by the flow versus the weight and the friction between particles. The formula by Van der Meer (7.2) is more suitable because it contains more parameters, is less empirical and it is not based on single grains. The most important extra parameters are the permeability and a factor which can be added for use of submerged breakwaters. Due to the overtopping of a submerged breakwater not all wave energy has to be absorbed which results in smaller diameters of the armour rock. The left-hand term can be interpreted as a stability parameter.

At last before the hard calculations start some boundary conditions have to be determined. In this case these are the wave conditions with a return period of 100 years, with a wave height of 8.6 meter and a wave period of 12.2 seconds. Delta is the relative density, for this situation this is about 1.55.

Hudson:

\[ M = \frac{\rho_s H_s^3}{K_D \Delta^3 \cot \alpha} \quad \text{or} \quad \frac{H_s}{\Delta d} = \sqrt[3]{\frac{K_D \cot \alpha}{\Delta}} \]  

Van der Meer:

\[ \frac{H_s}{\Delta d_{n50}} = 6.2 f^{0.18} \left( \frac{S}{\sqrt{N}} \right)^{0.2} \xi^{-0.5} \]  

As can be seen above the wave is poorly described, neither the Irribarren number nor the wave period is included. Omitting the permeability is even more remarkable, since the method is based on the flow around a particle. The two parameters left are the slope of the breakwater and \( K_D \). \( K_D \) is the ‘dustbin–factor’ this is related to wave breaking, overtopping and the kind of stones, it is 3.5 for rock and 9 for artificial elements.

The diameter of the stones has been set out in Figure 7.1 versus the slope of the armour.
Some comment will be given on this figure. A stone diameter of more than 2 meters is not desirable, because these stones are difficult to transport and place. In that case artificial elements should be used. Because of the other shape the $K_D$ also changes in a positive way. At last the slope at an angle of 12 degrees is twice as long as one of 6 degrees, a reduction in steepness means more material. These calculations are only made to verify the results of the more detailed Van der Meer calculation further on.

### 7.2.1.2 Van der Meer

There are two types of formulae by Van der Meer, one for surging and one for plunging breakers. The last type of waves is more common in this case, due to the steepness of the slope and the wave period. The Iribarren number varies between 1.0 and 2.5 for the waves and slopes considered. The method of Van der Meer is not valid for Iribarren values below 1.0 and has the most severe wave attack for collapsing waves with $\xi=3$.

Second to be worked out is the permeability. This parameter is established by curve fitting. The type of armour, filter and core are of great importance. Clay and even a sand core are impermeable for the wave action. Due to the impermeability of the core a stronger armour layer is needed. The different types of compositions are set out in Figure 7.2 versus the diameter of the armour layer. The more energy is dissipated the stronger the structure has to be. The risk of pressure build up from the inside of the breakwater also increases the diameter of the needed armour layer.
The stones are rather big, but they only have to be placed at the vulnerable spots of the breakwater. The toe for example has less wave attack. Another point which will be tackled later on is the reduction of the stone size for submerging the breakwater.

A clear distinction between the two types of breakwaters is made here, the reef and the submerged one. It might seem favourable to build an open structure, but this gives less reduction to the wave height. A decision can be made depending on the wave reduction requirements, the available material and their quantities. It is clear that the number of waves have influence on the damage, since this is the main attack. By these determining waves damage equilibrium is reached after a day of storm, this is about 7,500 waves. The stones are than ‘shaken’ in place and the structure will be stable. This process is related to the allowable damage, the damage factor S. The integer is more or less equal to the number of removed stones. The different levels are shown in Figure 7.3, a value of two is generally acceptable.

Although good reductions in diameter can be reached by accepting a little more damage, it can be dangerous in this situation. For much smaller stones an extra armour layer is easy to apply, for larger sizes this is more difficult. Another point of caution is the complete damage of the armour at a certain point. Then the filter layers are subsequently exposed to wave impacts. This last point is certainly the case during the construction because the core has to be placed before the armour. Although this can be done in a calm season, waves are still of noticeable height.

The last parameter to be considered is the reduction for overtopping, in this case of a submerged crest. The factor is determined by Van der Meer, based on Pilarczyk and is given in formula (7.3)
As shown in Figure 7.4, the reduction in diameter can be up to 25% for a submerged crest. Lower crests can reduce the diameter even more, but a crest 4 meters below sea level has hardly any function since most waves can pass.

![Reduction Low crests](image)

**Figure 7.4, rock diameter reduction for low or submerged crested breakwaters.**

A marginal note has to be made on the depth below sea level. The structure should never be exposed above water, which means that during calm conditions it is just some decimeters below. In this case the loads are minimal. During a severe storm higher water levels can be reached by different kinds of setup, this implies a deeper submerged breakwater. It can be possible that the reduction during calm conditions is too low, which might result in a larger stone size during severe storm conditions. This will be further elaborated during the design of the breakwater.

### 7.2.2 The submerged breakwater

#### 7.2.2.1 Location of the breakwater

On the north side there are no restrictions to the building area. From an economical point of view it is recommended to build the breakwater near the shore. A steep bottom slope reaches from the shoreline to 20m offshore (Figure 2.9, schematized cross section of the northern part). This very steep slope is not suitable for the breakwater foundation. After 20 meter the bottom slope becomes gentler, which is a better condition to construct the foundation. Between the breakwater and the shoreline an area is needed to ensure calm wave conditions in front of the fortress after wave-breaking during severe storms. To ensure calm wave conditions the breakwater is preliminary situated 100m offshore. The length of the breakwater equals the length of the fortresses foundation that should be protected and is 300m. The depth 100m offshore varies from around 11 to 8m.

#### 7.2.2.2 Determination crest freeboard

The crest freeboard is responsible for the reduction of the wave energy due to wave-breaking. It determines which waves will break on the breakwater. Also as indicated in section 6.4 it influences the transmission coefficient. In order to optimize the wanted wave reduction and the corresponding costs, the stones sizes and layers for different crest freeboard or crest heights will be determined. Finally the reduction of the wave heights can be plotted against the costs of realizing this wave reduction. For the comparison of different breakwaters in the next paragraphs, crest freeboards will be taken from 0 till 4 m below mean sea level.

#### 7.2.2.3 Armour layer

With the above information a stone diameter of the armour layer can be determined. Stones of these sizes do actually not go by diameter but by grading in weight, the one which is in the order of these diameters is three to six ton. There is one
larger grading which goes up to ten tons. Another possibility is the use of artificial elements. The diameter of three types of elements will be calculated, namely cubes, tetrapods and acropods. The Van der Meer formula (7.2) is not suitable to determine dimensions for artificial elements, but with an empirical relationship (7.4) based on the Van der Meer formula the elements dimensions can be calculated. On the left side the same stability parameter can be recognized. The needed coefficients for the cubes, tetrapods and acropods are given in annex xx.

\[
\frac{H_1}{\Delta D_n} = C + (A + BX_d / N^{S_d})^2
\]

(7.4)

For calculating the required stone and elements dimensions, some extra parameters are needed. The number of waves and the relative density are indicated in the general part of this chapter. They are respectively 7500 waves and 1.55. The permeability is 0.3, because the breakwater has a core. Finally the acceptable damage level is 2, to prevent the core suffering from wave exposure. In order to obtain a stable structure the required weight of the elements for different crest freeboards are given in Figure 7.5. For this graph different slopes have been taken for the breakwaters of rock and the other elements. This is done because otherwise either a slope which is too flat must be taken for the elements, or unrealistic diameters of rock stones have got to be used to reach the stability requirement. In appendix xx a table with all the data of the elements is given. The weight is calculated with the equivalent diameter. In the appendix the ratio normal diameter to equivalent diameter for all elements are given. The equivalent diameter is defined as the diameter of a cube with the same volume as the concerning rock/element.

Some remarks have to be made on this figure. The elements consist of concrete with a lower density than rock, namely 2400 kg/m$^3$ instead of 2600 kg/m$^3$. Acropods® and Tetrapods have a non-cube shape, with the property that they can grip on each other. This is in favour of the stability of the armour. Less material is needed but their construction costs however are high. Cubes on the other hand require a lot of concrete, but their construction costs are lower than those of acropods® and tetrapods.

As discussed at the beginning of the paragraph the slope is of influence at the size of the armour (Figure 7.1). In the figure above the slope is 1:5 for the rock, 1:1.5 for cubes and tetrapods and 1:1.33 for the acropods. This proves once more the difference in stability between rock and artificial elements. Concluding: to get a stable structure more rock volume is needed compared to an artificial structure. The related costs to rock and the elements will be further elaborated in chapter XX.

The figure is calculated for a return period of 100 years. These conditions lead to a higher water level than normal conditions, and thus a higher crest freeboard. In some cases normal waves might lead to larger stone sizes than high waves, because of the smaller crest freeboard and thus the smaller reduction factor. This is not valid for this case because here the determinative wave height is 8.6 with a required diameter of 1.76m with a crest freeboard of one meter (see annex xx). The proof of this is given in table xx, where for all return periods the crest freeboard is zero.
The core is the first part to be built, it is constructed by bulk placement of rock materials. In this case it is of no use to water tighten the core, because the breakwater is submerged. It has three other functions left, at first attenuation of the wave transmission which in this case is an important demand. A core of sand already is impermeable for waves, for just weaken the action coarser materials can be used. Taking a coarser material does not necessarily have a big influence on the wave transmission, because waves already lose most of their energy on the armour and filter layers. The next is the support of the armour. For supporting other layers interlocking is the keyword. This can be attained by a rough surface and therefore larger stones. The last function is the geotechnical stability, because the whole structure is resting on it. On the rock bottom seabed most of the failure mechanisms are out of the question. In this case the decision of what size and type of core does not primarily depend on its functions. These can all be fulfilled by sand, gravel and even light gradings.

The decision can best be made on constructional aspects. The dumping by waterborne equipment will be at a water depth of 10 meters. Further restrictions as currents, wind and waves can be minimized by working in the right season and at the right time of day. At last the stability during construction has to be guaranteed, this is enshrined by the same method as for the armour layer. The parameters, like for example the wave conditions, have been adjusted to construction circumstances. Regarding al this aspects, it results in stones of sizes of about 10 centimeter and 1 kilogram. The decision can be adjusted on available materials and their cost.

### 7.2.2.5 Filter layers

For a submerged breakwater a filter layer is a must, the armour layer is of such a diameter that the core will be washed out. The other function of a filter is drainage, this to avoid pressure build up below the armour layer. The size of the filter and stone diameter is fully determined by the armour layer and the core. In order to avoid an infinite list of options armour size has been determined. The previous sections clearly indicated that the stone size will be in the top grading and therefore the diameters are roughly known.

The structure of a regular filter mainly contains three rules, these are extensively explained in ref [xx](xx). (bed bank en shore protection) At first the total stability, the grains may not pass the stones of the top layer. The largest 85% of the grains of the lowest layer should at least be one-fifth of the size of the smallest 15% of the layer laying above it. For the internal stability of the applied layer, the range of the grain sizes should not be too large. This in order to let the larger grains block the smaller ones, the filter principle within a filter layer! The third rule is on permeability, this to avoid extra loads by pressure build up. The top layer should always be more open than its underlying layer. This is determined by the smallest grains of both layers. The smallest fifteen percent of the armour layer should be at least five times bigger than those of underlying layer.

In this case the rock size of the armour is rather big, therefore these rules can not be fully applied for the layer positioned below the filter. Extra regulations have to be applied for heavy gradings. These are stricter than the geotechnical described above and demand a diameter ratio of about 2.3 for the underlying layer. For the weight this implies a factor of 1/15 to 1/10 between the armour and the under layer. The advantages of this are the rougher surface of the under layer which increases the interlocking with the armour and the permeability.

By these regulations the under layer below the armour layer of 6 to10 ton will have a grading of 0.3 to 1 ton. The next layer can be determined by the normal regulations. This leads to the lightest grading of 10 to 60 kg. Under this last filter layer gravel can be placed as a core. This composition of filter layers can be applied for all the variations of armour and core. This is found on the fact that the armour layer has to be the heaviest grading. The diameter is of minor importance for this grading because the weight is determinative. This allows the calculated variation in the diameter of the armour layer. The same principle is for the diameter of the core. It can be the size of the last filter layer or down to a few centimeters and still be geometrically closed.

### 7.2.2.6 Dimensions

The dimensions of the breakwater depend on the thickness of the layers, the core, the slope and the crest freeboard. As said before the slope is 1:5 for a rock armour layer, 1:1.5 for cubes and tetrapods and 1:1.33 for acropods®. The
dimensions of the breakwater will be determined for different values of the crest freeboard. In Figure 7.6 a cross-section of the breakwater is given. The design conditions for a breakwater are given in Table 7.2. The average depth hundred meters offshore is 9.5m.

### Table 7.2, design conditions submerged breakwater

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Symbol</th>
<th>Design rule</th>
<th>Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crest Width</td>
<td>$B$</td>
<td>$B = 3D_{n,50}$</td>
<td></td>
</tr>
<tr>
<td>Thickness armour layer</td>
<td>$t_a$</td>
<td>$t_a = n_k k_i D_{n,50}$</td>
<td>$n_k = 2$ (Rock)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$k_i = 1$ (Cubes)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$k_i = 1.04$ (Tetrapods)</td>
</tr>
<tr>
<td>Thickness under layer</td>
<td>$t_f$</td>
<td>$t_f = 2D_{n,50}$</td>
<td></td>
</tr>
<tr>
<td>Thickness filter</td>
<td>$t_f$</td>
<td>$t_f = 2D_{n,50}$</td>
<td></td>
</tr>
</tbody>
</table>

With the given information the dimensions of the crest height, armour layers, under layers and core are determined for rock, cubes and tetrapods for different values of the crest freeboard. Also the numbers of stones in the armour layer are estimated with the next formula (7.5).

$$N_a = n_l A k_i (1-n_k) D_{n,50}^{-2}$$

(7.5)

The values of the coefficients, the dimensions of the breakwater and the number of stones are indicated in annexes xx, xx and xx.

### 7.2.2.7 The reduction of the wave height

Wave energy will mainly be dissipated by wave breaking. Waves which do not brake will lose energy by transmission over and through the breakwater. The wave height of the breaking waves can be calculated with the breaker parameter. The transmission coefficient can be calculated with an empirical formula by Van der Meer (1991).

$$C_i = a \frac{R}{D_{n,50}} + b$$

verified for $1 < \frac{H}{D_{n,50}} < 6$ and $0.01 < s_{op} < 0.05$

(7.6)

With the coefficients $a$ and $b$:

$$a = -0.24 + 0.031 \frac{H_s}{D_{n,50}}$$

(7.7)

$$b = 0.51 + 0.0323 \frac{H_s}{D_{n,50}} - 0.0017 (\frac{B}{D_{n,50}})^{1.84} - 5.42 s_{op}$$
In appendices xx, xx and xx the reduced wave height for different crest freeboards are given for rock, cubes and tetrapods.

### 7.2.2.8 Conclusion

The submerged breakwater is a good concept, it can effectively be applied to reduce the wave energy and wave height. An optimal design can be made on the basis of the needed material, economical possibilities and wave reduction demands. The required stones in the armour layer versus the reduced wave height are given for rock, cubes and tetrapods in Figure 7.7.

![Stones in armour layer versus the reduced wave height](image)

Figure 7.7, stones in armour layer versus the reduced wave height.

The figure makes clear that the number of cubes and tetrapods are much less than the rock stones. However the costs per unit of a tetrapod or a cube are much higher than stones. An economical optimisation is needed to make a well considered choice. The total volume of the breakwater, the volumes of all layer and core, are given in appendices xx, xx and xx. Also these values will be used in the economical optimisation to get the best concept.

The wave height with a return period of 100 years is very large, namely 8.6m. This wave height requires large stones in the armour layer. In combination with the restricted height of the breakwater, little room is left for adjustment of the different kind of layers. Which causes a small difference when comparing to a reef breakwater.

### 7.3 Crack formation in the foundation due to alternating loading

#### 7.3.1 Introduction

In the investigations made previously (ref XX), a number of blocks are determined. These blocks are considered to be caused by crack formation, which is the result of subsequently loading by waves, a phenomenon known as ‘fatigue’. In this paragraph the fatigue damage of the rock due to wave impacts is predicted in order to estimate the required maintenance. In order to do this the theory of Wöhler is used, which will briefly be described. When the fatigue behaviour of the rock is combined with the wave impacts on the rock a damage model is formed. As the wave impacts after constructing a hydraulic structure are added to this model a reduction of the damage for the hydraulic structure can be computed.

#### 7.3.2 Data

The total wave impact consists of many waves with different wave heights. Input data for this model is the amount of days that a certain deep sea wave exists in the project area, this data is obtained from ref XX. The absolute value of the number of waves is unknown but due to the fact that the future damage is compared with the present situation no absolute values are needed. Other input data is the state of the rock mass as described in ref XX and XX. These two investigations have both determined the number of blocks on which a movement is measured, the so called sliding blocks. When taking into account the difference in time of these investigations which is 10 years, an estimation can be made of the damaged parts which will develop in the following years.
In Table 7.3 data is given of the comparison of the report of 1989 and 1999. It can be concluded that certain sliding blocks have collapsed completely and other new ones are formed. The new blocks have to be anchored which means maintenance costs of the foundation. With this data it can be said that without any measures in the following 10 years XX blocks will be formed. However, when applying a certain wave reducing alternative as described in chapters 7.2 to XX, the height and number of waves attacking the foundation will be reduced, which will also reduce the number of repairs necessary. It has to be mentioned that it is not possible to fully take away the wave load, which induce that a certain monitoring of the foundation is always necessary.

### Table 7.3, rock damage data

<table>
<thead>
<tr>
<th>GeoCuba</th>
<th>ENIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of blocks</td>
<td>18</td>
</tr>
<tr>
<td>common</td>
<td>6</td>
</tr>
<tr>
<td>collapsed</td>
<td>-</td>
</tr>
<tr>
<td>new north</td>
<td>4</td>
</tr>
<tr>
<td>new corner</td>
<td>7</td>
</tr>
<tr>
<td>new south</td>
<td>2</td>
</tr>
</tbody>
</table>

From the table can be concluded that 12 blocks are formed in the past 10 years.

#### 7.3.3 Scaling

In order to be able to count the wave heights the waves are scaled, which takes into account the damage caused by the pressure generated by a certain wave height and the number of waves with this height. To know the relative damage caused by one wave compared to another Wöhler’s theory is used. This theory is explained in Figure 7.8, where on the horizontal axis the number of loads until failure is depicted and on the vertical axis the alternating load value. The angle of the Wöhler line represents the material fatigue behaviour, load cases above the Wöhler line will collapse the structure. For concrete like materials this angle is approximately 1:3, which can be seen as a reasonable estimation for rock. From this can be concluded that a doubled wave load causes 8 times more damage. As said before the number of waves of a certain height is unknown, hence the number of days a certain wave occurs is taken as \( N \) and can be compared.

![Wöhler curve](image)

**Figure 7.8, Wöhler curve**

The wave loads are scaled according to the Wöhler line to a wave load for a wave height of 1 m. This results in a multiplier for every wave load, and the sum of this multiplier times the number of waves for every different wave range results in the total damage.

\[
\text{Damage} = \sum N_{\text{waves},x} C_{\text{scaling},x} \quad (7.8)
\]

\[
C_{\text{scaling},x} = \left( \frac{p_{\text{wave, height}=x}}{p_{\text{wave, height}=1}} \right)^3 \quad (7.9)
\]
A scaled and a normal wave overview for the north side of the fortress is shown in Figure 7.9. The ‘real’ line gives the percentage of waves which occur at a certain height and the scaled line gives the damage that the waves of a certain height cause. The pressure is assumed to vary linearly with the wave height. As can be seen in this figure the waves between 2 and 4 m result in a relatively large damage to the rock foundation, due to their quite high frequency and their relatively high damage multiplier.

![Total wave impact](image)

**Figure 7.9, scaled wave impact and real wave occurrence**

### 7.3.4 Determination of damage after applying an alternative

When a hydraulic structure has been constructed the wave loads on the fortress change and thereby the damage to the rock foundation. Relatively low waves become smaller and high waves will not be present at all at the rock mass. Assumed is that all the waves higher than the reduced maximum height by the breakwater have the same impact on the fortress. In Figure 7.10 the impact of different deep sea wave heights on the fortress after applying a several different breakwaters is shown. The graph shows the part of the damage which is caused by a certain deep sea wave height. For instance, in the present situation is about 27 % of the damage caused by waves from 2 – 3 m height. This damage is reduced to 12 % when a breakwater with a $H_{\text{trans}} = 3.5$ m.
Figure 7.10, damage of different wave heights with several breakwaters

In figure xxx the relative cumulative damage level after constructing an offshore submerged breakwater is shown for the north side of the fortress. It points out the reduced wave height due to the submerged breakwater versus the percentage of damage compared to the present situation. When for example a breakwater with $H_{\text{trans}} = 3$ m will be applied, the damage at the corner section will be reduced to 27.5 % and 25 % in case of the north side.

Figure 7.11, cumulative damage after applying a submerged breakwater

In annex xxx these figures are shown for all alternatives and different parts of the study area.
7.4 Detailed investigation of the strengthening of the structure

7.4.1 Introduction

In this chapter the strengthening of the rock foundation will be considered according to the two concepts pointed out before. Due to the very different situations of each different damage case as described in paragraph 2.2.2, they also have to be considered separately. For both possible alternatives the damage cases where they can be applied will be described, resulting in an overview of the required materials. When the costs of different materials, parts and execution are known, the optimal solution for each damage case can be determined which will be done in chapter XX.

7.4.2 Concrete supports

In paragraph 6.7 the main idea of this alternative is already explained. Two aspects have a large influence whether this alternative can be applied on a certain damage case or not. First the strengthening of the rock has to fulfil the main demand concerning the visibility. This means that concrete supports are only applicable on damage cases with a sliding block of a limited size. Furthermore the possibility of constructing the supports should be taken into account. Damage cases with very steep slopes or sliding blocks at sea level can hardly be strengthened by this alternative when concerning the execution.

Considering the previously mentioned aspects this alternative can conveniently be applied on the following damage cases. These are together with the required amount of reinforced concrete shown in Table 7.4.

<table>
<thead>
<tr>
<th>Block</th>
<th>Section</th>
<th>Amount of Concrete Required (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>6</td>
<td>11.1</td>
</tr>
<tr>
<td>8</td>
<td>11</td>
<td>21.6</td>
</tr>
<tr>
<td>Cave</td>
<td>18</td>
<td>29.0</td>
</tr>
</tbody>
</table>

A concrete quality with a mean representative compression strength of 25 MPa and a reinforcement percentage of 0.5% is assumed to be able to withstand the pressure of a sliding block. The concrete strength is in this case about twice the intact rock strength which should be sufficient.

In annex XX cross sections are given from the damage cases which can be repaired with a concrete support.

Anchoring

The principle of this alternative is already briefly described in paragraph 6.7. A small extension is made on this principle, this because the existence of two ways of force transmitting which are possible. First the anchors can be seen as bolts, which means that the required forces are transmitted by the shear resistance of these. Secondly the force can be transmitted by friction. This induces a requirement of pretensioning the anchors in order to increase the normal pressure on the sliding plane. These two aspects are shown in Figure 7.12 is case of a maximum load caused by gravity forces.

![Figure 7.12, scheme of 2 ways of anchoring](image-url)
These two ways of force transmitting have both their own advantages and disadvantages. Due to the fact that the angle of internal friction of the cracks is rather low, a bolt is more effective. This is shown in the equations (7.10) and (7.11), where the shear resistance of the bolt is determined according to the Huber-Hencky-Von Mises yield criterion.

\[
\text{Resistance-friction} = \frac{f_{y,d} A_{\text{bolt}}}{1.1} \tan \phi_{\text{crack,max}} = 0.42 f_{y,d} A_{\text{bolt}} \tag{7.10}
\]

\[
\text{Resistance-shear} = \frac{f_{y,d} A_{\text{bolt}}}{\sqrt{3}} = 0.58 f_{y,d} A_{\text{bolt}} \tag{7.11}
\]

An advantage however of the pretensioned anchors is the absence of large stress concentrations. When considering certain blocks which are near the water level, many small loads will act on this. This is usually known as fatigue loading. When anchors are subjected to this kind of loading they are very sensitive to stress concentrations. Hence pretensioned anchors have to be used when blocks are located near mean sea level.

In paragraph 2.2.3 it is shown why no dynamic impact factor should be applied when dimensioning the anchoring. In Table 7.5 the maximum load is given from which anchor dimensions can be determined. These loads are the difference between the gravity or wave load and the resistance by friction and the apparent cohesion, including the determined safety factors. Furthermore the damage cases where a pretensioned bolt should be applied and the length of the block to a firm layer are given.

<table>
<thead>
<tr>
<th>block</th>
<th>load on anchor parallel to the slope (kN)</th>
<th>cause</th>
<th>anchor type</th>
<th>length to firm layer (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>626</td>
<td>gravity</td>
<td>bolt</td>
<td>3.2</td>
</tr>
<tr>
<td>4</td>
<td>54</td>
<td>wave</td>
<td>bolt</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>138</td>
<td>gravity</td>
<td>bolt</td>
<td>2.9</td>
</tr>
<tr>
<td>7</td>
<td>404</td>
<td>gravity</td>
<td>bolt</td>
<td>2.9</td>
</tr>
<tr>
<td>8</td>
<td>115</td>
<td>gravity</td>
<td>bolt</td>
<td>4.1</td>
</tr>
<tr>
<td>9</td>
<td>294</td>
<td>gravity</td>
<td>bolt</td>
<td>6.3</td>
</tr>
<tr>
<td>10a</td>
<td>253</td>
<td>gravity</td>
<td>bolt</td>
<td>9.8</td>
</tr>
<tr>
<td>10c</td>
<td>412</td>
<td>gravity</td>
<td>pretensioned</td>
<td>10</td>
</tr>
<tr>
<td>16</td>
<td>25</td>
<td>gravity</td>
<td>bolt</td>
<td>7.7</td>
</tr>
<tr>
<td>18</td>
<td>343</td>
<td>gravity</td>
<td>pretensioned</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The damage cases with blocks 11, 12 and 13 are not listed in the table due to the fact that their stability and strength is sufficient to withstand either gravity force or quasi-static wave impact.

Now the loads on the anchors are known, a design of their sizes can be made. The number of bolts to be applied in each separate case is depending on the diameter and the steel strength used. Various diameters from 16 mm up to 60 mm and steel strengths of 235 MPa and 355 MPa are considered. Usually high strength steels are used when pretensioning is necessary, but due to the local environmental conditions these steels are not recommended because of their sensitivity to corrosion.

Apart from the steel also an amount of grout is necessary to construct a layer between the steel and the rock. In Figure 7.13 a scheme is given of an anchor with grout surrounding.

Table 7.5, required anchoring forces
Figure 7.13, scheme of anchor and surrounding grout

The length of an anchor is dependent on the length to a firm layer which are shown in Table 7.5, and the length necessary to transmit the forces from the anchor to the rock; both are shown in Figure 7.14. The latter is dependent on the strength and diameter of the bolt and of the strength of the grout used. The transmission lengths are given in Table 7.6, assumed a compressive strength of 25 MPa of the grout.

**Figure 7.14, anchor length**

<table>
<thead>
<tr>
<th>Table 7.6, required anchoring lengths</th>
</tr>
</thead>
<tbody>
<tr>
<td>diameter</td>
</tr>
<tr>
<td>bolt strength S235</td>
</tr>
<tr>
<td>bolt strength S355</td>
</tr>
</tbody>
</table>
In order to estimate the costs, an overview is needed of the materials needed. In annex XX several tables list the amount of steel, grout and anchors for each separate bolt diameter, steel strength and damage case. As an example the steel necessary for damage case 10a is plotted versus the bolt diameter for different steel strengths.

Figure 7.15, example of amount of material needed
8 Methods of realization and costs
9 Alternatives

A survey of the chosen solutions including the costs for each alternative.
10 Decision
11 Elaboration of the chosen alternative
12 Conclusions
13 Recommendations
14 Quality assurance
15 Literature