Preliminary design of the configuration of the harbour lay-out of the proposed harbour at the Boca Vieja at La Ceiba

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PRELIMINARY DESIGN
OF THE CONFIGURATION
OF THE HARBOUR LAY-OUT OF THE
PROPOSED HARBOUR AT THE BOCA VIEJA
AT LA CEIBA

PART 1.1 (TEXT)

Pieter Bogers
Rotterdam October 25th 1991
PREFACE

This is the first part of my thesis work. The second part, titled: 'Modelling Tidal Inlets', deals with an one dimensional computer model for the sediment balance of a tidal inlet.

For both parts I would like to thank very much ir. J.P. Noppen and ir. J.A. Roelvink for their supervision, support and suggestions. I owe very much thanks to prof.ir. K. d'Angremond for his indispensable support to help me with the search for a subject for which a site visit in a foreign country was necessary.

Furthermore I would like to thank the Survey and Engineering department of the Empresa Nacional Porturia for their help and support in collecting all the required field data.

Both parts have been carried out at Aveco infrastructure consultants bv. I would like to express my thanks to all employees of this company who made my work at Aveco very pleasant.

A special word of thanks goes to S. Pearson BSc. who helped me during my stay in Honduras with organizational problems and language problems. He also gave very useful suggestions during the writing of the first part of this thesis and he checked both parts on the use of the English language. Furthermore I would like to thank him and his family for their hospitality during my stay at their home in Honduras.

Rotterdam, october 25th 1991
Pieter Bogers
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HARBOURS


SEDIMENTATION


[9] Data banks of the Royal Dutch Meteorological Institute


[12] Instituto Geografico Nacional

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1. INTRODUCTION

1.1 AUTHORIZATION

This is a report concerning the preliminary design of a proposed harbour complex at Boca Vieja near the town of La Ceiba on the Northern coast of Honduras.

In 1990 the Government of Honduras instructed the National Port Authorities “Empresa Nacional Portuaria” (ENP) to prepare a design for the construction of a new harbour complex at the port of La Ceiba.

In order to prepare a good technical design for the harbour complex, ENP thought it prudent to approach coastal engineering experts for technical assistance in the design. At that same time AVECO Infrastructure Consultants were providing technical assistance to ENP (in Honduras) for the preparation and execution of dredging works, and consequently ENP approached AVECO for assistance in the design of the harbour at La Ceiba.

Around the same time a Delft University student, P. Bogers, was working on a thesis entitled “Modelling of Tidal Inlets” with AVECO and he required some test cases in order to demonstrate how the mathematical model worked. It was thought that La Ceiba would provide an excellent test case for the mathematical model and that the student’s work could be useful to ENP on the design of the harbour.

In the late summer of 1990 P. Bogers visited Honduras to collect field data, thereafter carried out the mathematical model simulations in Holland and prepared a brief technical report of his findings. Applications of these findings are the sole responsibility of the user.

The main objective of the study is to determine the following aspects (on a preliminary design level):

1. The configuration and location of the breakwaters forming the entrance to the harbour.
2. The quantifying of longshore sediment transport before and after construction of the harbour.

in order to enable ENP to prepare a detailed design of the harbour complex.

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One would like to express sincere thanks to the Survey and Engineering Departments of ENP for their help and support in collecting all of the required field data.

1.2 PROJECT ORGANIZATION

The project team for the study comprised the following persons:

S. Pearson : Project Coordinator
P. Bogers : Project Engineer

The Project Coordinator (PC) was responsible for coordinating the work of the Project Engineer and providing technical support during the field data acquisition phase in Honduras. Thereafter the PC reviewed the results of the simulations, conclusions and recommendations.

The Project Engineer (Pi) was responsible for the collection of the field data in Honduras, the execution of the mathematical model simulations and the preparation of the final report.

1.3 DESCRIPTION OF REPORT

After this introduction one finds in Chapter 2 the conclusions and recommendations of this report. In chapter 3 a brief description is given of the project. Chapter 4 contains a detailed description of all starting points, boundary conditions and environmental data. In chapter 5 the nautical aspects of the harbour lay-out are treated in detail. In chapter 6 the morphological aspects of the harbour lay-out are treated.
2. CONCLUSIONS AND RECOMMENDATIONS

2.1 CONCLUSIONS

The results presented in this report are based on very limited data and therefore should be used with the utmost care. Furthermore, the results are only on a preliminary design level and should not be considered as a final design.

In the study of the sediment transport along the coast near La Ceiba there appeared to be a rather large difference in prediction of the sediment transport. Hindcast modelling shows a westward transport of 100,000 m³/year. The formula of Bijker predicts a westward transport of 380,000 m³/year. Similar cases show a transport between 300,000 m³/year and 500,000 m³/year in the breakerzone of a coast is often encountered. The sediment transport predicted by the use of the formula of BIJKER - westward 380,000 m³/year, eastward 30,000 m³/year - is therefore used in the conclusions written below and the fact that it has the most severe consequences on the erosion at both La Ceiba and near the new harbour. The eastward transport of 30,000 m³/year does not contain the part of sediment brought to the sea by the Rio Cangrejal. The sediment brought to the sea by the river, can increase the eastward transport considerably. The effect of sediment brought to the sea by other rivers on the total sediment pattern has not been taken into account too. The reason for not having studied these effects is the limited information on how much sediment is brought by the rivers to the sea.

The proposed location of the harbour is in an inlet. At present the inlet does not have an access channel having the necessary depth for ships to enter the harbour safely. It is advised to dredge an access channel at 5.85 m below the lowest tide level observed at La Ceiba based on a nautical point of view.

One breakwater on either side of the access channel is advised based on the sediment pattern and nautical requirements. The east side breakwater will extend to the 6.15 m depth contour. The east side breakwater will extend to the 5.3 m depth contour. The effect of sediment brought by rivers to the sea and thus increasing the longshore sediment transport is not taken into account in the determination of the length of the breakwaters. At present it is not possible to estimate the contribution of the river sediment to the total sediment transport.
The most salient conclusions are summarized below. In figure 2.1 an example is drawn which fulfils all the mentioned items.

HARBOUR LAY-OUT

1. The access channel needs an orientation between the -30 and -60 degrees (angle between centre line of access channel and perpendicular of the coastline). In this manner minimum wave action inside the harbour is expected. Wave action is considered to be the most important threat to safe navigation of ships inside the harbour.

2. A curve in the access channel is necessary due to its orientation to connect the sea entrance and the harbour. The curve requires a radius of at least 200 m.

3. The access channel is divided into three zones corresponding with the procedures of a ship entering a port:

   PARTLY PROTECTED PART, OPEN AREA
   Only one breakwater protecting the access channel:
   Depth 5.85 m
   Bottom width 30.0 m
   The 5.85 m depth should be available over the bottom width.

   PROTECTED PART, ENTRANCE
   Depth 5.85 m
   Bottom width 70.0 m
   The 70.0 m width could extend over 220 m, starting from the sea edge of the west side breakwater. Over the bottom width of 70.0 m the 5.85 m water depth could be available.

   PROTECTED PART, PAST ENTRANCE
   Depth 5.85 m
   Bottom width 30.0 m
   The 5.85 m depth could be available over the bottom width of 30.0 m.

4. For turning the ships a turning circle is necessary requiring a diameter of 200 m. It is located in front of the berths.

5. In order to stop the design ship safely a stopping length of approximately 400 m is advised.
The access channel requires two breakwaters. These breakwaters offer ships a safe passage from sea to port and vice versa. Secondly they offer protection of the channel against sedimentation due to longshore current transport.

A stilling basin should be constructed in order to reduce wave energy entering the harbour and hence minimise wave action due to resonance inside the harbour.

SEDIMENTATION PATTERN

The predominant westward longshore sediment transport in the proposed area is estimated at 380,000 m³/year.

An eastward sediment transport is present and consists of sediment brought from the Rio Cangrejal to the sea and transported to the Boca Vieja by the longshore current. Sediment transport without the contribution of river sediment is estimated at 30,000 m³/year. At present it cannot be estimated how the total eastward transport will be.

Erosion will take place on the west side of the harbour entrance as soon as the breakwaters are being built. There are two areas which therefore require extra attention.

The stroke of land separating the berth facilities from the sea is approximately 150 m wide and 420 m long (measured from the channel). If this piece of land is not protected, erosion will take place and threaten the berth facilities within 4 years. Protection can be offered by locating the west side breakwater 420 m off the channel thus offering protection of the stroke and offering space for the required stilling basin. Still erosion will take place on the west side of the west side breakwater. The coast-line is expected to retreat almost 205 m directly westward of the west side breakwater.

Erosion will take place near La Ceiba. Calculations show that at the jetty of the present port a retreat of the coast-line can be expected of almost 8.5 m.

Beach nourishment at La Ceiba is advised at places where erosion threatens houses/buildings etc.

Sedimentation both inside the access channel and in the harbour is
possible due to sediment from: rivers discharging into the Boca Vieja and sediment passing the breakwaters. The amount of sediment brought to the Boca Vieja by rivers cannot be predicted at present.

Sedimentation due to longshore transport will occur inside and/or in front the access channel. It will happen after sediment starts passing the breakwaters which is estimated to take place at 4 years after construction. Regular maintenance dredging is required to keep the access channel open.

13 The harbour entrance should be protected by two breakwaters that could be placed 420 m apart at the land-side (see also point 10). The east side breakwater is located directly east of the access channel.

14 The east side breakwater could be extended to the 6.15 m depth contour and the west side breakwater to the 5.3 m depth contour. In this manner the access channel can protected against sedimentation for a period of approximately 4 years after commence of construction.

2.2 RECOMMENDATIONS

The results presented in this report are only on a preliminary design level and therefore it is recommended that further research be carried out before the final design of both the harbour lay-out and configuration is prepared. Special attention requires, among others:

1 Sediment pattern: it is obvious that a closer estimate of the sediment transport (in the surf zone and in rivers discharging into the Boca Vieja) results in a more secure determination of both the necessary effective length of the breakwaters and the possible sedimentation inside the Boca Vieja.

2 Study of wave penetration: due to the fact that this is especially important with respect to the safe navigation and safe mooring of small ships in the harbour.

3 Erosion protection measures at La Ceiba: erosion will always occur at La Ceiba and directly westward of the west side breakwater in case of constructing breakwaters at the Boca Vieja. In case of a solution of beach nourishment, a detailed plan must be developed containing, among others, answers to the questions when, where, how much and how often nourishment has to be carried out.
3. PROJECT DESCRIPTION

3.1 GENERAL

The project is located at Boca Vieja near the town of La Ceiba on the Northern coast of Honduras.

At present La Ceiba has an open piled timber jetty which serves the export of fruit (Standard Fruit) and the loading and unloading of small coasters which run between the mainland and the Bay Islands.

The jetty is located on a very exposed coastline and consequently can not provide any form of shelter to shipping during bad weather. During a storm in the late winter of 1989 a small coaster moored alongside the jetty broke its moorings and was driven against the jetty by wave action destroying the deck and super-structure at a point halfway along the length of the jetty.

As a result the jetty was left in two parts and before trade could be commenced again the jetty had to be repaired which was both costly and time consuming and will not prevent the same accident from occurring in the future.

Due to the ever increasing need to provide safe berthing and anchorage facilities and the expected future economic expansion of La Ceiba and the surrounding area, the Honduran Government has decided to construct a new harbour complex at La Ceiba. The Boca Vieja, a small lagoon 3.5 km east of La Ceiba, was chosen as the new location of the harbour based on studies (see figure 3.1).

The basic plan of the harbour consists of a concrete piled berth 300 m long, a port of refuge for coastal ships, several warehouses, a road connecting the harbour with the high way to San Pedro Sula, the important industrial town of Honduras and two breakwaters protecting the ships during passage from sea to harbour.
3.2 STUDY PROGRAMME

The study was split-up into the following phases:

Phase 1: Study preparation.
Phase 2: Site Visit
Phase 3: Mathematical Model Simulations
Phase 4: Analysis of results
Phase 5: Preparation of final report

Phase 1 consisted of search for information in Holland on the subject of sedimentation in general in and around a lagoon and for information on the weather data, wave etc.

Phase 2 will be described briefly in paragraph 3.3. In chapter 6 phase 3 and 4 are described.

3.3 SITE VISIT

Site visits to the Boca Vieja were made in order to obtain the following information:

1. Detailed bathymetry of the coast consisting of the surf zone.
2. Grain distributions of the soil of the inlet bottom and sea bottom
3. Graphs showing distribution of suspended sediment in certain cross sections
4. Current velocity profiles during spring tide and neap tide in the inlet

Surveys were carried out during the 20th of September and the 12th up till the 14th of October 1990 for the four items mentioned above. During the first visit the bathymetry of the surf zone was determined and a visual inspection of the Boca Vieja was made. Current survey and sediment survey were carried out during the second visit.

The visual inspection of the inlet showed that only the channel connecting the two branches with the sea was changed since the topographical survey of 1988. The entrance of the inlet was moved approximately 170 m westward parallel to the sea before discharging into it. A small sand bar separated the sea from the channel. During the first visit the bar was often swashed over by waves of the sea. While visiting, the inlet opened itself. The opening had a width of approximately 10 meters and was 0.5 meters deep and the current velocities were approximately 1 m/s.
The inlet has (see figure 3.2) two branches which come together approximately 80 m before the common branch discharges into the sea. In each of the branches cross sections were chosen so as to determine possible discharge in these. In each section current velocity measurements were planned to take place as well as taking soil samples and water samples. Furthermore a cross section was established in the main branch so as to determine the tidal prism.

BATHYMETRIC SURVEY OF THE SURF ZONE
Each 25 meters starting at point B (see figure 3.2) and moving in westward direction a cross section was surveyed. In a vessel a constant profile was recorded up to a water depth of five meters.

The results (see figure 3.3) show a sandbar at approximately 60 m off shore. During the second survey one could see that waves break on this bar. The average slope of the surfzone can be estimated at 0.015.

CURRENT SURVEY
No information was obtained about the currents inside the Boca Vieja. The connection with the sea was too small and closed during the second visit.

SEDIMENT SURVEY
Soil samples were taken in the cross sections 1, 2 and 3 (see figure 3.2) and in the surf zone in front of the Boca Vieja. The samples were analyzed on particle size and specific weight.

The variation in the $D_{90}$ and $D_{50}$ are as follows:

- $D_{90} = 300 \times 10^{-6} \text{ m} - 1699 \times 10^{-6} \text{ m}$
- $D_{50} = 190 \times 10^{-6} \text{ m} - 880 \times 10^{-6} \text{ m}$

According to the Unified Soils Classification [re. 10] the sediment samples are classified as sand because the grain sizes lie between $4 \times 10^{-3} \text{ m}$ and $0.074 \times 10^{-3} \text{ m}$. The specific weight analysis showed a large deviation (varying from 1.18 to 2.50).

Water samples were taken in cross section 1 at the same locations of the soil samples of this cross section at a depth of $0.8 \times \text{ depth}$. Analysis show a suspended sediment concentration varying between $0.05 \text{ kg/m}^3$ and $0.1033 \text{ kg/m}^3$.

Originally two soil samples of the sea bottom in the surf zone were taken to analyze but one got lost during transport. The soil sample was analyzed on...
grain size distribution. The $D_{90}$ and $D_{50}$ are as follows:

$D_{90} = 840 \times 10^{-6} \text{ m}$
$D_{50} = 280 \times 10^{-6} \text{ m}$

According to the Unified Soils Classification [re. 10] the sediment sample is classified as sand because the grain sizes lie between $4 \times 10^{-3} \text{ m}$ and $0.074 \times 10^{-3} \text{ m}$. Classification [re. 10]. The specific weight is: $2650 \text{ kg/m}^3$
4. BOUNDARY CONDITIONS AND STARTING POINTS

4.1 BOUNDARY CONDITIONS

ENP have established the following boundary conditions for the preliminary design of the harbour:

- The new harbour will be located at Boca Vieja, which is 3.5 km east of the town of La Ceiba.
- The basic lay-out of the harbour will be as shown in figure 4.1.
- The harbour must provide berthing and anchorage facilities for small coastal shipping.
- Access channel to harbour will facilitate the passage of one ship at a time.
- Small coastal shipping must be able to enter and leave the harbour at any time of the day safely in significant wave heights up to a maximum of 2.5 m in deep water.
- The breakwaters must prevent sedimentation in the access channel to the harbour due to littoral drift for at least the first 3 years after final construction.

4.2 STARTING POINTS

1 DESIGN SHIP

This is the hypothetical ship, that characterizes the various ships calling at the port (it does not necessarily have to be the largest ship\(^1\)). Based on these dimensions the lay-out is made. The dimensions of the design ship are based on information handed down to us by the E.N.P., of La Ceiba (see table 4.1). All ships call at the port of La Ceiba frequently. Therefore the dimensions of the design ship are based on the largest ships calling at La Ceiba and are a combination of the dimensions of the ships "Hercules" and "Helena Z":

Length: 40 m
Beam: 6.00 m
Draught: 4.50 m

\(^1\) For instance: During high tide the available water depth in the access channel is increased. The necessary water depth can be minimised if the largest ships enter and leave the harbour only during this period.
Table 4.1: Vessels calling at the port of La Ceiba

<table>
<thead>
<tr>
<th>Name of Vessel</th>
<th>Length (m)</th>
<th>Draught (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toniac I</td>
<td>21.40</td>
<td>2.45</td>
</tr>
<tr>
<td>Toniac II</td>
<td>25.60</td>
<td>3.05</td>
</tr>
<tr>
<td>Hercules</td>
<td>25.60</td>
<td>4.30</td>
</tr>
<tr>
<td>Margarita</td>
<td>21.35</td>
<td>2.15</td>
</tr>
<tr>
<td>Helena z</td>
<td>38.45</td>
<td>4.00</td>
</tr>
<tr>
<td>Edith j</td>
<td>35.40</td>
<td>1.25</td>
</tr>
<tr>
<td>Lady Michelle</td>
<td>23.50</td>
<td>3.05</td>
</tr>
</tbody>
</table>

It is assumed that the design ship enters the harbour up to waves of 2.50 m on deep water (Thus 96.3 % time of the year ships can enter the harbour, see table 4.2).

Table 4.2: Percentage of wave occurrence in 1 year

<table>
<thead>
<tr>
<th>Significant wave height (m)</th>
<th>Wave period seconds</th>
<th>Direction 0 degrees</th>
<th>Direction 30 degrees</th>
<th>Direction 60 degrees</th>
<th>Direction -30 degrees</th>
<th>Direction -60 degrees</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>3.5</td>
<td>0.5</td>
<td>1.3</td>
<td>2.7</td>
<td>0.2</td>
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<td>1.0</td>
<td>4</td>
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<td>1.9</td>
<td>4.7</td>
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<tr>
<td></td>
<td></td>
<td>3.6 %</td>
<td>9.9 %</td>
<td>24.9 %</td>
<td>0.8 %</td>
<td>1.6 %</td>
</tr>
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</table>

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2 FREQUENCY OF CALLING

The present frequency of ships calling at the port of La Ceiba is estimated at one ship a day (see table 4.3). It is assumed that this frequency will not change after construction of the new harbour.

Table 4.3: Characteristic time distribution of vessels (mentioned in table 4.1) calling at the port of La Ceiba

<table>
<thead>
<tr>
<th>Name of Vessel</th>
<th>Calls at the port of La Ceiba during the month july 1990</th>
<th>Calls at the port of La Ceiba during the month august 1990</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toniac I</td>
<td>7</td>
<td>6</td>
</tr>
<tr>
<td>Toniac II</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Hercules</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Margarita</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Helena z</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>Edith j</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Lady Michelle</td>
<td>3</td>
<td>5</td>
</tr>
</tbody>
</table>

4.3 ENVIRONMENTAL DATA

1 WEATHER CLIMATE

There are recordings over 20 years of rainfall, wind direction, wind velocity, air-pressure, humidity, evaporation etc. due to the presence of an airfield at La Ceiba. The average rainfall pattern can be seen in figure 4.2. It can clearly be seen that the rain period lasts from October to January. February can be called a rain month too. The intensities of showers are during these months the largest. Month averages (in these rain periods vary from 135 mm to 1120 mm. The catchment area of the Boca Vieja is estimated at: 29 km$^2$. The wet surface of the Boca Vieja is estimated at 0.15 km$^2$. A shower of rain of 550 mm during 24 hours (which has been the severest, up to 1989) assuming that all will flow directly to the Boca Vieja, will cause a discharge of 185 m$^3$/s during one day.

WIND CLIMATE

Wind data are available of 1971 up till 1989 from the La Ceiba Airport. These show a dominating wind between the 0 degrees and
60 degrees. (see figure 4.3). The average wind speed in these directions varies from 4.1 m/s to 7.4 m/s.

2 WAVE CLIMATE
Wave data obtained from the KNMI [re. 9] are used. These data are obtained by visual surveys on board of merchant ships during 1971 - 1980 in an area characterised by north latitude 15.0 degrees by 17.9 degrees and east longitude 87 degrees by 88.9 degrees which contains the Gulf of Honduras. These data cannot be applied directly to the wave climate near the Boca Vieja due to shoaling\(^2\) and refraction\(^3\) of the waves. Using the linear theory as explained by Battjes [re. 6] the wave climate near the Boca Vieja is determined (see annex 4.1).

The dominating waves have near the shore a height between the 1 m and 2.72 m and a direction between the 0 degrees and 26 degrees, (angle between the perpendicular of the coastline and the wave direction).

3 CURRENT CLIMATE
Generally there are two sorts of currents near shore: Tidal driven and wave driven current. The latter only appears in the surf zone.

The current caused by tidal difference at a time along the coast can be estimated using Chezy (see annex 4.2). As the current is very low, it is not taken into account in this design.

The pilot manual [re. 8] shows a presence of a constant ocean current between 1 m/s and 2 m/s in eastward direction. It originates from the equatorial current that flows past the Windward Isles into the Caribbean Sea.

Currents due to density differences are not taken into account.

4 MATERIAL
The average \(D_{90}\) in the Boca Vieja based on the sediment survey mentioned in paragraph 3.3 is:
\[D_{90} = 1.104 \times 10^{-6} \text{ m}\]

\(^2\) Shoaling: Change of wave heights due to change in water depths.
\(^3\) Refraction: Change of the direction of waves moving in shallow water at an angle to the contours due to phase difference along the wave crests.
The average $D_{50}$ in the Boca Vieja based on the sediment survey mentioned in paragraph 3.3 is:

$$D_{50} = 535 \times 10^{-6} \text{ m}$$

As the grain analysis show clearly that the samples can be classified as sand the specific weight is taken as 2650 kg/m$^3$. The average $D_{90}$ in surf zone near the Boca Vieja based on the sediment survey mentioned in paragraph 3.3 is:

$$D_{90} = 840 \times 10^{-6} \text{ m}$$

The average $D_{50}$ in the surf zone near the Boca Vieja based on the sediment survey mentioned in paragraph 3.3 is:

$$D_{50} = 280 \times 10^{-6} \text{ m}$$

The specific weight is: 2650 kg/m$^3$

5 WATER LEVELS

The tide near La Ceiba is semi diurnal. Only during the days around neap tide the tide is more or less diurnal. The reason for this difference is the low tide difference (0.17 m average) and the inequality of two tide cycles during a day. There is no significant difference in variation of the average water level throughout the year (During the winter the water level does not alter much when compared with average water level during the summer). Elevations of tide planes, with respect to Mean Sea Level, that is based on three years of observations: 1959 - 1961 [re. 12], are as follows:

- Highest tide, observed: 0.39 m
- Mean High Tide: 0.09 m
- Mean Tide Level: 0.00 m
- Mean Low Water: -0.08 m
- Lowest tide, observed: -0.33 m

Storms cause also elevation of the water level. The elevation is estimated at 1.00 m above the highest astronomical tide. The lowest tide observed is used as reference level for the determination of the depth of the access channel.

6 COASTLINE BEHAVIOUR

There are aerial photographs available form 1954, 1977 and 1987. One topographical map is available based on aerial photographs of 1960, 1961 and 1964. A nautical map is based on aerial photo-
graphs of 1976. The analysis of the behaviour of the coast at La Ceiba during the years 1954 up till 1987 is done by using the railway, connecting La Ceiba with San Pedro Sula, as a baseline. The zero point corresponds with point A (see figure 3.1). This analysis shows (see figure 4.4) that the coastline did not move considerably during these years. Even the hurricane Fifi (September 1974) did not cause a large coastal change. A coastal change can be seen at point 1 which is close to the construction of a breakwater nearby in 1986.
5. HARBOUR LAY-OUT

In this chapter the design of the harbour lay-out is restricted to the entrance channel. Some remarks are made on the turning circle and the port of refuge. Using rules of thumb a preliminary lay-out will be given.

5.1 ACCESS CHANNEL LAY-OUT

The design of the access channel of a port can be divided into three areas corresponding with the process of a ship entering a port. This process consists of three stages:

1. Preparation of the ship for entering the port
2. Approach of the harbour entrance
3. Slowing down and berthing

The above mentioned stages together with the configuration of the local coastal area need special attention depending on the type of ship. The harbour entrance is usually protected by breakwaters. Their primary function is to offer ships a safe passage from the open unprotected sea to the protected harbour basin. Their second function is the protection of the access channel against sediment transport along the coast. For large ships (tankers of e.g. 200,000 dwt) the stopping operation starts at open sea, depending on the length of the protected access channel. Small boats can wait until they actual enter the protected access channel. The three areas will be discussed separately in this paragraph.

5.1.1 Access channel - open sea -

Here ships start preparing the approach. Large ships (such as tankers of more than 200,000 dwt) already start slowing down. Small ships (as is the case here) can navigate themselves to the entrance. Small ships have a far lower inertia mass than large ships. Therefore, among others, small ships can be navigated far more easier than large ships.

DEPTH
The required depth is not only the maximum draught of a ship. A ship sailing has an increased draught due to several phenomena of which the most important are listed here:
Squat: the uniform sinking of the ship resulting from pressure changes in the surrounding water
Trim: rotation of the ship about a horizontal crosswise axis. It results from asymmetry of the return flow patterns at bow and stern

The rule of thumb of the PIANC\textsuperscript{5} [re. 3] advises a depth consisting of the maximum draught of the design vessel increased by 20\% in unprotected areas. This rule however is based on the Euro Maas channel conditions at Rotterdam and can only be applied to large ships (200,000 dwt and more). It is therefore not used for this design.

In Port Engineering [re. 1], a method is given which calculates the required depth by summation of the following factors:

1. Loaded draught
2. Tide
3. Density change
4. Squat
5. Pitching and rolling
6. Trim
7. Empirical factor

ad 1 Here the loaded draught is 4.50 m.
ad 2 The tide has an amplitude of 0.085 m. It is stated in the boundary conditions (see paragraph 4.1) that ships enter and leave the harbour at any time of the day (except during waves higher than 2.5 m in deep water). Therefore the lowest observed water level at La Ceiba is used as reference plane for the water depth in the access channel.
ad 3 There are no density differences to be expected at the harbour entrance (see paragraph 4.3).
ad 4 Using the method of Barras [re. 3] the squat can be estimated at 0.30 m (see annex 5.1).
ad 5 Pitch: rotation of the ship over a horizontal axis parallel to its beam due to wave action.
Roll: rotation of the ship over a horizontal axis parallel to the length of the ship due to wave action.
Little is known for these phenomena. For small ships pitch can cause a severe problem especially if the length of the boat is 0.5 times the wave length.
ad 6 Eisminger advises a extra depth of 0.10 m. Others advise between the 0.30 m and 0.60 m.
For reason of safety, increase of manoeuvrability and in case of heavy littoral drift an empirical factor is applied between the 0.60 m and 1.20 m. This extra depth prevents thus that as soon as sediment starts passing the breakwaters the access channel is closed.

Following the above mentioned factors (see also figure 5.1) the depth will lie between:

<table>
<thead>
<tr>
<th>Factor</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>loaded draft</td>
<td>4.50 m</td>
</tr>
<tr>
<td>squat</td>
<td>0.10 m</td>
</tr>
<tr>
<td>trim</td>
<td>0.10 m</td>
</tr>
<tr>
<td>pitch</td>
<td></td>
</tr>
<tr>
<td>empirical factor</td>
<td>0.60 m</td>
</tr>
</tbody>
</table>

Required depth: 5.30 m + 6.40 m with respect to lowest tide level observed at La Ceiba

It is assumed that ships do not enter the port during wave heights higher than 2.5 m (see paragraph 4.1). Therefore nothing is taken into account for pitch because it will only be a problem during waves of 4 m (see annex 4.1).

The empirical factor and the trim factor will be averaged assuming that the above listed values are extremes. The depth in the channel therefore must be:

<table>
<thead>
<tr>
<th>Factor</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>loaded draft</td>
<td>4.50 m</td>
</tr>
<tr>
<td>squat</td>
<td>0.10 m</td>
</tr>
<tr>
<td>trim</td>
<td>0.35 m</td>
</tr>
<tr>
<td>empirical factor</td>
<td>0.90 m</td>
</tr>
</tbody>
</table>

Total depth: 5.85 m with respect to lowest tide level observed at La Ceiba

In this report the depth of the access channel on open sea will be taken as 5.85 m with respect to lowest tide level observed at La Ceiba.

**BOTTOM WIDTH**

The necessary bottom width of the access channel [re. 2] consists of the following factors:

1. The beam of the ship (B)
2. Ship clearance: a ship never sails a straight path due to cross winds and/or cross currents. It needs more space than its beam (0.50 B on
both sides of the ship)

3 Bank clearance: a ship leaving the centre line of the channel is sucked towards the bank. The ship tries to get away from the bank and the result will be yawing of the vessel for which extra space is needed. (1.5 B on both sides of the ship)

4 In case of two lane traffic an extra bottom width of 1 B is necessary between the two lanes

An access channel with one traffic lane is sufficient, as the average frequency of ships calling at the port is 1 per day. The total minimum bottom width is:

<table>
<thead>
<tr>
<th>Beam of ship</th>
<th>6.00 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ship clearance</td>
<td>6.00 m</td>
</tr>
<tr>
<td>Bank clearance</td>
<td>18.00 m</td>
</tr>
<tr>
<td>Total bottom width</td>
<td>30.00 m</td>
</tr>
</tbody>
</table>

Over this bottom width the full 5.85 m water depth must be present. If we assume slopes of 1:7 the overall width is 112 m at water level.

5.1.2 Access channel - transition area -

DEPTH
Within the protected channel the wave action is lower than at open sea. Other factors as squat and trim will remain the same or even increase. As there is no information available on this subject it is recommended that the depth is as calculated at the open sea.

BOTTOM WIDTH
A ship passing from open sea to the protected channel will not follow a straight course when cross currents are present. These currents cause the boat to rotate about a vertical axis (see figure 5.2). The shipmaster will not directly recognise this alteration in course. The ship will follow a path as drawn in figure 5.2 After some time the shipmaster reacts and sails the boat to the centre line. Thus an extra bottom width just behind the entrance is needed to give a ship the required manoeuvrable space. No information is available for the necessary extra bottom width for small boats. However the above described phenomenon will occur. Therefore the rules of thumb for larger ships will be used and modified.

The drift angle is often taken as 14 degrees. The reaction time of a ship-
master to the changing course lies between the 2 and 3 minutes. As small ships have a far better manoeuvrability it is assumed that the drift angle is somewhat around 10 degrees. The reaction time of a shipmaster on a small boat is far smaller than on a large tanker. Therefore a reaction time of 1 minute will be used.

Another important factor is the effective velocity of the ship entering the harbour. Due to cross winds and cross currents the vessel obtains a higher velocity than without these components. Therefore the necessary extra bottom width will be calculated with two extremes: fast approach and slow approach.

- assuming an entrance speed of \(2 \text{ m/s} \) (4 kn):
  \[ B_{\text{extra}} = \sin (10) \times (\text{length of ship} + 60 \times 2) \]
  \[ B_{\text{extra}} = 30 \text{ m} \]
  This bottom width must extend over a width of 160 m

- assuming an entrance speed of \(3 \text{ m/s} \) (6 kn):
  \[ B_{\text{extra}} = \sin (10) \times (\text{length of ship} + 60 \times 3) \]
  \[ B_{\text{extra}} = 40 \text{ m} \]
  This bottom width must extend over a length of 220 m

For reasons of safety the bottom width of \(30 \text{ m} + 40 \text{ m} = 70 \text{ m} \) is recommended. The 5.85 m water depth should be available over the bottom width.

### 5.1.3 Access channel - protected area -

After entering the protected harbour the ship sails to the berth. While sailing it slows down in order to have a minimum speed when mooring. The necessary length is estimate at 10 times the length of the design ship:

Thus 400 m stopping length is required.

### 5.2 WAVE PENETRATION

Waves entering a harbour are reflected at the end of the harbour. It is then possible that resonance occurs. Resonance is the phenomenon of amplification of a free wave or oscillation of a system by a forced wave or oscillation of exactly equal period. The forced wave may arise from an impressed force upon the system or from a boundary condition. By using a rule of thumb this can be predicted:

\[ L = (2n - 1) \times \frac{\lambda}{4} \]
Methods to prohibit or minimise the resonance can be:
- Orientation of the entrance in such a way that most waves cannot enter the harbour undisturbed.
- Inside the harbour reflection walls can be built that damp a specific incoming wave through reflection.
- Adjust the lay-out of the harbour in such a way that the mentioned rule of thumb is never fit for any incoming wave length.
- Stilling basin: In front of the harbour a basin is created where waves lose much of their energy. The wave action inside the port is thus minimised. Furthermore the chance of resonance is minimised.

Harbours for small ships often use a stilling basin. In practice this solution has proven its value and therefore it is recommended in this harbour too. The possible resonance can only be calculated after the complete design of the harbour is known.

5.3 ORIENTATION OF THE ACCESS CHANNEL

During the whole operation of entering the harbour and mooring it is wise to minimise the hinder from cross currents and cross winds. Cross currents at the protected entrance can usually not be avoided. Ships normally have the least hinder of these currents, when encountering them head on. In practice ships enter the harbour in such a direction, that cross currents tend to slow down the vessel.

Cross winds increase the difficulty of entering the protected area. They tend to increase the extra bottom width directly after the entrance as ships are given a velocity perpendicular to the centre line of the access channel (see paragraph 2.2.3.). Furthermore they necessitate a wider channel, protected as well as unprotected, as ships will have difficulty keeping a straight course. An access channel therefore must have an orientation parallel to the most frequently occurring wind direction. This is however far more important with large vessels, having a high superstructure, than with small vessels.

For small ships waves must not attack the vessel head on or from astern as
maximum pitch will occur. Thus the access channel must have an ori-
entation that does not coincide with the direction of incoming waves (which
is also not allowed with respect to wave penetration). Altogether the above
mentioned demands lead to different orientations. The matter of impor-
tance in this case is as follows:
1. Avoidance of large pitch
2. Cross currents head on when entering the harbour
3. Avoidance of cross winds

The orientation will thus lie between the -30 and -60 and degrees (angle
between the perpendicular of the coastline and the centre line of the access
channel. Due to this orientation a curve is necessary inside the stilling
basin. The radius of the curve should not be less than 5 times the length of
the design ship and preferably 10 times its length. This rule of thumb is
designed for large ships. Small ships have a far better manoeuvrability than
large ships. Therefore it is assumed that 5 times the design ship’s length is
sufficient. The curve should start behind the point at which ships are not
hindered by cross currents.

5.4 LOCATION TURNING CIRCLE AND SHELTER

The necessary space for a ship to turn its bow, is 3 times its length
assuming the ship has no bow thrusters and no tugboat assistance. Ships
entering La Ceiba do not have bow thrusters and furthermore tugboats are
not available. The turning therefore will require more room. The diameter of
the turning circle is therefore estimated at 200 m and located in front of
the berths.

It is assumed that during sheltering of vessels no ship will leave the
harbour. The space in front of the berths can in this case be used as shelter
instead of turning. It is assumed that ships anchored in open, protected
water require a space between the ships of 2 times the beam of the design
ship. The turning circle offers thus for 15 design ships shelter.

5.5 LAY-OUT OF THE ENTRANCE OF THE BOCA VIEJA HARBOUR

ORIENTATION
The access channel will have an orientation between -30 and -60 degrees
with the perpendicular of the coastline. From this direction few waves
originate (see table 4.2) and thus pitch will be minimised. The cross current
is flowing from west to east. This will result in a higher approach
velocity of the ship and thus more space is required behind the entrance. This extra space is also required due to the dominating wind direction which attacks a ship over its full length. Thus the fast approach scenario is used leading to an access channel of 60 m just behind the breakwaters extending 220 m shoreward.

LENGTH OF PROTECTED AREA
The breakerzone is a zone that is difficult to navigate through. Therefore protection must be offered at least up to the breakerzone. A simple relationship exists which relates the water depth at which waves break to the breaking wave height:

\[ H_b = \gamma \times h \]

\( \gamma \) Breaker parameter (lies between 0.6 and 0.8)
\( H_b \) Wave height at breaking
\( h \) Water depth

It is stated (see paragraph 4.1) that vessels will not enter the harbour during waves higher than 2.50 m on deep water (corresponding with a wave height of 2.72 m on shallow water). The breakwaters therefore must extend to a water depth of 3.85 m (using a gamma of 0.7). The length of the breakwaters therefore become 200 m from a nautical point of view.

At 200 m off shore the surrounding sea bottom is less deep (3.85 m) than the access channel (5.85 m). It is possible that the sediment transport along the coast threatens the access channel outside the breakwaters. To prohibit accretion inside the access channel the breakwaters need to be extended. How far will be discussed in chapter 3.

DIMENSIONS
Based on the forgoing paragraphs the access channel has the following dimensions:

Unprotected part, open area
Depth 5.85 m
Bottom width 30.0 m

Protected part, entrance
Depth 5.85 m
Bottom width 70.0 m
The 70 m bottom width must extend over 220 m, starting from the sea edge of the west side breakwater.
Protected part past entrance
Depth 5.85 m
Bottom width 30.0 m

REMARK: The 5.85 m water depth must exist over the bottom widths.

For the turning circle and the port of refuge following conclusions can be drawn:

TURNING CIRCLE
A bottom width of 200 m in front of the berths is required.

SHELTER
The turning circle can be used as a port of refuge as it is assumed that no ship will leave the harbour when ships seek shelter.
6. **SEDIMENT PATTERN AT THE BOCA VIEJA**

6.1 **INTRODUCTION**

The north coast of Honduras has a wave and wind climate with dominating winds and waves between the north and north east direction. Accordingly a sediment transport is mainly directed from east to west. Evidence of this sediment transport direction can be found at La Ceiba. In 1986 a breakwater was built. After two years the breakwater was fully accreted on the east side. Calculations\[re. 4\] show an estimated westward sediment transport of 100,000 m$^3$/year.

Studies of aerial photographs and topographical maps show a rather stable coastline near La Ceiba (see figure 4.4). The point 1 km east of point A (see figure 4.4) indicates the location of the breakwater. The effect of the constructed breakwater can clearly be seen. Effects of storms on a long term basis cannot be found.

6.2 **CURRENT SEDIMENT PATTERN AT THE BOCA VIEJA**

Looking at the tide and making a small computation (see annex 6.1) it is clear that the Boca Vieja inlet will behave as a bay storage basin. The tidal currents are weak and the tidal prism is small. Only during heavy rainfall some current inside the harbour may occur.

The currents at sea depend on the wave action (tidal induced currents are not considered (see paragraph 4.3). These currents originate from breaking waves carrying large amounts of sediment. This sediment transport can be increased by rivers discharging into the sea carrying sediment. The direction of the current depends on the direction of the waves when breaking.

At La Ceiba waves originate from between the 30 and 60 degrees thus causing a westward current. During some time of the year waves originate from between -30 and -60 degrees causing an eastward transport. Severe storms originate often from between -30 and -60 degrees. During these periods rivers rise, due to heavy rainfall and transport large quantities of sediment to the sea and even colour the sea [re.8]. The longshore current will consequently be directed to the east. It is thus possible that sediment from the Rio Cangrejal is transported eastward. How much cannot be calculated at present as no field data were obtained concerning this aspect.

Pieter Bogers
Little sediment from the sea will be brought inside the harbour as the tide currents are weak as long as no sediment passes the breakwaters. Possible sedimentation inside the harbour will be due to sediment, brought to the inlet by rain in the catchment area of the small rivers discharging in the Boca Vieja during the rain season and by sediment passing the breakwaters. The amount of sedimentation brought by the rivers is very hard to predict.

The construction of breakwaters and dredging an access channel of 5.85 m as well as the deepening of the harbour up to 5.85 m water depth will not change the behaviour of the harbour as a bay storage basin. The currents inside the harbour will remain weak and thus the sediment pattern inside harbour will not change very much. Sedimentation will take place on the east side of the breakwaters, erosion on the west side. The complete sediment pattern picture at the Boca Vieja exists of both longshore transport and cross shore transport. Lack of information prevents us at present to estimate the latter one. Longshore transport can be calculated in two different ways. The first method is the method of hindcasting. The second method is based on the use of two widely used sediment transport formulas: CERC and BIJKER [re. 11]. It is stressed that both methods cannot predict the amount of sediment brought to the sea by the Rio Cangerejal nor the amount of sediment brought to the sea by other rivers.

HINDCAST modelling
By looking at the past the mathematical model COSIM (see footnote 12, page 31) is fit to the local situation. Parameters in the model are adjusted in such a manner that the behaviour of the coast calculated by the model equals the behaviour of the coast in nature.

CERC formula
Observations in both nature and model indicated a correlation between the longshore component and the energy flux at the outer edge of the surfzone. Expressed in a formula the sand transport rate is:

\[ S = A \times (H_b)^2 \times n_b \times c_b \times \cos(\varphi_b) \times \sin(\varphi_b) \]

\( \varphi_b \)  \textit{angle of wave attack at the breakerzone}
\( A \)  Dimensionless coefficient varying from 0.028 to 0.08
\( H_b \)  Wave height at the breakerzone
\( n_b \)  Factor relating the wave velocity at deep water to the wave velocity at shallow water (0.5 at deep water, 1 at shallow water)
\( c_b \)  Wave velocity at shallow water
Short comings in this formula are:
- The constant A
- The influence of the grain size is neglected
- Influence of the beach slope, cannot be taken into account
- The influence of the width of the breaker zone cannot be taken into account
- Influence of other driving forces than waves cannot be taken into account

BIJKER formula
The sand transport is a combination of bottom transport and suspended transport. Bijker adapted a bed load river transport formula and connected it with a suspended sediment transport. This connection resulted in the following formula:

\[ S_s = 1.83 \cdot Q \cdot S_b \]
\[ S_{tot} = S_b + S_s \]

\( S_b \) bed load transport

\( S_s \) suspended transport

\[ S_b = \exp \left( \frac{5D_{50}V\sqrt{g}}{C} \cdot \exp\left[-0.027 \Delta D_{50} \rho_g \right] \right) \]

\[ Q = I_1 \ln(33h/r) + I_2 \]

\( D_{50} \) Particle diameter
\( V \) Mean velocity
\( C \) Chezy coefficient
\( r \) Bottom roughness
\( g \) Gravitational acceleration
\( I_1 \) Einstein integral
\( I_2 \) Einstein integral

\( \Delta \) relative density of bedmaterial \( \left( \frac{\rho_s - \rho}{\rho} \right) \)

\( \rho \) mass density of water
\( \rho_s \) mass density of bed material

\( \mu \) ripple factor \( \left( \frac{C}{C_{90}} \right) \)

\( C_{90} \) Chezy coefficient based on \( D_{90} \)

\( \tau_{cw} \) bed shear stress due to waves and current

\[ \tau_{cw} = \frac{\rho g V^2}{C^2} \times [1 + 0.5(\frac{\mu}{V})^2] \]
\[ \xi = C \sqrt{\frac{f_w}{2g}} \quad \text{(Bijker's parameter)} \]

\[ f_w = \exp(-5.977 + 5.213 \times \left( \frac{a_0}{r} \right)^{0.194}) \]

\[ a_0 \quad \text{maximum horizontal displacement of water particles just outside the boundary layer} \]

AVECO has developed a program called SANSIM\(^6\), that calculates for different wave heights and for different wave angles the yearly sediment transport according to CERC and BIJKER. The situation at the breakwater near La Ceiba was used to determine which formula could best be used at the Boca Vieja. The input data are listed in annex 6.2. The results (see annex 6.3) show a rather large difference in sediment transport. Looking at the hindcast modelling its is clear that CERC gives a very high transport (1,140,000 m\(^3\)/year westward, 69,000 m\(^3\)/year eastward). BIJKER shows a three times higher transport than calculated by hindcasting (436,000 m\(^3\)/year westward, 26,000 m\(^3\)/year eastward). Due to the rather large difference in prediction between the hindcast model and CERC the latter is abandoned due to the fact that it is based on very rigid assumptions making it suitable for only few locations. BIJKER on the contrary is far more universal to use and therefore not immediately rejected.

In order to come to a conclusion Sansim is also used at the Boca Vieja. The input data are listed in annex 6.4. Here although the bottom profile and the orientation of the coastline is quite different BIJKER gives the same sediment transport as calculated at the breakwater (see annex 6.5). This result is confirmed by the coastline study\(^7\).

There is no real proof which of the two models predicts the sediment transport at the Boca Vieja correctly. Therefore both BIJKER (west ward sediment transport = 380,000 m\(^3\)/year, 30,000 m\(^3\)/year) and the hindcast prediction (west ward sediment transport = 100,000 m\(^3\)/year, 8,000

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\(^6\) Some of the features of SANSIM are:
1. The bottom profile is schemed to parallel depth contours
2. Wave heights of various directions at deep water are used as boundary conditions at deep water
3. The root mean square height is used (significant wave heights are transformed in the program to root mean square heights)
4. Frequency of occurrence of different wave heights can be taken into account
5. Shoal cannot be treated

\(^7\) CERC too gives the same result but it is indifferent to a change in bottom profile so there is no extra support for CERC.
m³/year\(^8\)) will be used in the further analysis.

**DISCUSSION OF RESULTS SANSIM APPLIED AT THE BOCA VIEJA**

The deep water wave data already show that the most important waves are those between 1 m and 2.5 m high with an angle at deep water between 30 degrees and 60 degrees with the perpendicular of the coastline. It is clear that most transport is caused in this range (60 %). Still the sediment transport caused by the waves higher than 2.5 m, although not frequently occurring, cannot be neglected.

The breakerzone at the Boca Vieja can be estimated as described in paragraph 2.5. Here not the significant wave height\(^9\) is used but the root mean square wave height\(^10\) as SANSIM calculates with the latter. The relationship between these two is:

\[
H_{\text{sign}}^2 = 2 \times H_{\text{rms}}^2
\]

Thus the breakerzone at the Boca Vieja extends to somewhere between the 3.9 m and 5.2 m depth contour\(^11\). Sediment will start passing the breakwater within days after construction if the length of a breakwater is not larger than the width of the breakerzone. The breakwater can only block the sand transport effectively if it is extended past the breakerzone. This extension is called the effective length of a breakwater. The effective length can be estimated by the Pelnard Considère theory [re. 11] that is described in the following paragraph.

Both breakwaters need not to have the same effective length. Due to the dominating westward transport the east side breakwater has to be far longer than the west side breakwater.

**6.3 SEDIMENT PATTERN AFTER CONSTRUCTION OF BREAKWATERS**

Breakwaters always cause accretion and erosion. They block partly or completely the breaker zone causing sedimentation 'upstream' of the

---

\(^8\) It is assumed that in case of a 100,000 m³/year westward sediment transport a eastward sediment transport is present. The ratio westward-eastward sediment transport in this case is approximately the same as in case of a westward sediment transport of 380,000 m³/year.

\(^9\) Significant wave height: The average height of one third highest waves of a given wave group

\(^10\) Root mean square wave height: The root of the average wave height of a given wave group

\(^11\) Not the 4.63 m wave height in shallow water is used as this wave height occurs when waves come form 0 degrees thus not causing a sediment transport
breakwater and erosion downstream. The severeness of accretion/erosion can be estimated by use of the Pelnard Considère theory.

The coastline is schematised to a profile of parallel depth contour lines. It is assumed that there is a point at deep water, past which no changes take place due to distortions in the sediment transport. Within the shoreline and this point the bottom at every point has the same distortion. A coast has reached its equilibrium if along the coastline the sediment transport does not change. Changes in sediment transport cause accretion or erosion.

The basic equation is the combination of continuity and mass transport. Continuity can be given by:

\[
\frac{\partial S_x}{\partial x} + d \times \frac{\partial y}{\partial t} = 0
\]

\( S_x \) Longshore sediment transport
\( d \) Depth, indicating the location past which no changes take place due to variation of sediment transport
\( x, y \) location of coast

The most important variables for changes in sediment transport along a coast, are the change of wave height and change of wave angle relative to the coastline. It is assumed that the deep water wave conditions do not change. The sediment transport can now be related directly to the changes in the angle of wave attack. Assuming further small changes in the sediment transport when changing the angle of wave attack the following relationship between the variation of wave angle and coastline can be derived:

\[ s_x = \frac{\partial S_x}{\partial \phi} \]

For small variations of phi a relation exists between phi and the \( x, y \) coordinates:

\[ \partial \phi = \frac{\partial y}{\partial x} \]

The angle between the wave crest at a depth \( h \) and the coastline and a time \( t \) can be described as:

\[ \phi = \phi' - \frac{\partial y}{\partial x} \]

Substituting the above formulas into each other one arrives at the following
equation: which needs two boundary conditions and one initial condition:
\[
\frac{s_x \cdot \frac{\partial^2 y}{\partial t^2} - \frac{\partial y}{\partial t}}{d} = 0
\]

This equation is the basis of the COSIM\textsuperscript{12} model which is developed at AVECO. With COSIM the behaviour of the coastline is studied after construction of the breakwaters. Again it is stressed that the effect of sediment brought by the Rio Cangrejal via the longshore current to the Boca Vieja is not taken into account.

**ALTERNATIVE I**
The computations are started with two breakwaters 112 m apart (with slopes 1:7 this is the minimum width of the access channel at the water level). The effective length of the east side breakwater is 200 m and the west side breakwater is 50 m. The results are shown in annex 6.7 and annex 6.8.

**ALTERNATIVE II**
The two breakwaters are 420 m apart. This alternative has a stilling basin. The results are shown in annex 6.9 and annex 6.10.

### 6.4 DISCUSSION OF ALTERNATIVES

**ALTERNATIVE I**
Two breakwaters 105 m apart
effective length: east side breakwater 200 m
west side breakwater 50 m

Advantages:
1. Shortest possible breakwaters in case of a 4 year protection of the access channel against sedimentation due to longshore sediment transport

Disadvantages:
1. Erosion westward of the west side breakwater will threaten the

\textsuperscript{12} Some features of the COSIM model are:
1 The influence of breakwaters, offshore breakwaters, shore protection, beach nourishment, river discharges, special boundary conditions on the coastline can be studied with COSIM
2 Complex combinations of the items mentioned in 1 can be taken into account by COSIM
3 All influences are time depended

Pieter Bogers
berth facilities after 4 years in case of a gross sediment transport of 410,000 m$^3$/year and in 14.4 years in case of a gross sediment transport of 108,000 m$^3$/year.

2 Erosion occurs near the city of La Ceiba. In case of a gross sediment transport of 410,000 m$^3$/year an erosion at the present port of approximately 5 m in 5 years and approximately 2 m in 15 years in case of a gross sediment transport of 108,000 m$^3$/year.

3 No room for stilling basin, thus possibility of agitating wave action inside the harbour basin.

4 After 4 years, in case of a gross sediment transport 410,000 m$^3$/year sand will pass the east side breakwater and the access channel will sedimentate. In case of a gross sediment transport 108,000 m$^3$/year sand will pass the east side breakwater after 14.5 years and the access channel will sedimentate.

ALTERNATIVE II
Two breakwaters 420 apart
effective length: east side breakwater 200 m
west side breakwater 50 m

Advantages
1 No extra protection is required for the stroke of land separating the berth facilities from the sea.

2 Enough room for the required stilling basin, thus minimising agitating wave action in side the harbour basin.

Disadvantages

1 Erosion occurs near the city of La Ceiba. In case of a gross sediment transport of 410,000 m$^3$/year an erosion at the present port of approximately 8.5 m in 5 years and approximately 3.7 m in 15 years in case of a gross sediment transport of 108,000 m$^3$/year.

2 After 4 years, in case a gross sediment transport of 410,000 m$^3$/year sand will pass the east side breakwater and the access channel will sedimentate. In case of a gross sediment transport of 108,000 m$^3$/year sand will pass the east side breakwater after 14.9 years and the access channel will sedimentate.

---

Gross sediment transport: Sum of sediment transport in eastward direction and westward direction.
Alternative II does not require an extra shore protection at the Boca Vieja (the erosion will take place ‘behind’ the harbour) and it offers a stilling basin. It offers the most safest passage of ships from sea to harbour and does not require difficult shore protection measures. The safety of the passage ships is considered to be most important function of the breakwaters. Therefore and the ‘automatic’ protection of the stroke of land that separates the berth facilities are the reasons why alternative II is chosen.

It is clear from the calculations that sooner or later the sediment will start passing the east side breakwater. The sediment will be deposited in front of the access channel making a shoal\(^1\). This shoal can become a large hinderance for the ships and can even shut the port. It is thus obvious that as soon as sediment starts passing the east side breakwater regular dredging is necessary in order to prevent difficult navigation of ships when entering or leaving the port and in the worst case to prevent the port from closing.

Measures to protect the city against the erosion can be: shore protection, off shore breakwaters, groins and sand suppletion. Shore protection will need a very expensive erosion protection for its foundation. Furthermore erosion will start on the west side of the protection thus only moving the problem. Off shore breakwaters will protect the coast and cause accretion between the coast and the off shore breakwater. However the same happens as with the shore protection: erosion on the west side of the off shore breakwater. Beach nourishment does not have the disadvantage of causing erosion on the west side of the suppletion but it is necessary to carry out the nourishment regularly.

Erosion can not be tolerated at La Ceiba due to the location of the present harbour, which will be used even after the construction of the new harbour for the export of, among others, bananas. Furthermore the people in La Ceiba live very close to the coast thus erosion will threaten their houses [re. 4]. Therefore, based on the present knowledge of the situation, beach nourishment is advised as protection.

Concerning the effective length of the east side breakwater the following can be said: Due to the dominating sediment transport in westward direction a 50 m effective length of the west side breakwater is not required as

---

\(^1\) Sediment passing the east side breakwater encounters deeper water and a more quiet sea (no breaker-zone). Therefore sediment will sink to the sea bottom and thus cause a shoal in front of the access channel.
follows from the calculations done by COSIM. Again it is stressed that the
effect of sediment of the Rio Cangrejal on the eastward sediment transport
nor the effect of sediment brought to the sea by other rivers on the total
sediment pattern has not been studied due to too limited information. In
order to prohibit sediment transport in east ward direction sedimentating
the access channel it is advised to construct the west side breakwater
further that the 5.30 m depth contour.
EMPRESA ANCIONAL PORTUARIA
PUERTO CORTES
HONDURAS C.A.

PRELIMINARY DESIGN
OF THE CONFIGURATION
OF THE HARBOUR LAY-OUT OF THE
PROPOSED HARBOUR AT HTE BOCA VIEJA
AT LA CEIBA

PART 1.2 (ANNEXES AND FIGURES)

Pieter Bogers
Rotterdam october 25th 1991
Figure 2.1: Preliminary harbour design (example)
Figure 3.2: The Boca Vieja
Figure 3.3: Coastal profiles at the Boca Vieja
Coastal profiles at the Boca Vieja

Distance between profiles is 25 m

Water depth with respect to MSL (m)

Figure 3.3: Coastal profiles at the Boca Vieja
Coastal profiles at the Boca Vieja

Distance between profiles is 25 m

Water depth with respect to MSL (m)

Figure 3.3: Coastal profiles at the Boca Vieja
Coastal profiles at the Boca Vieja

Distance between profiles is 25 m

Figure 3.3: Coastal profiles at the Boca Vieja
Coastal profiles at the Boca Vieja

Distance between profiles is 25 m

Water depth with respect to MSL (m)

Figure 3.3: Coastal profiles at the Boca Vieja
Figure 4.1: Basic harbour lay-out
Rainfall data at La Ceiba Airport

month average (1000 mm)

Figure 4.2: Average rainfall pattern near the Boca Vieja
Rainfall data at La Ceiba Airport

Figure 4.2: Average rainfall pattern near the Boca Vieja
Wind direction distribution, data
obtained at La Ceiba airport 1971-1989

Figure 4.3: Wind direction data

Not in brackets are the wind directions.

0°  North
90°  East
180°  South
270°  West

In brackets are the percentages of occurrence in one year of a certain wind direction.
Coastline development near La Ceiba
from 1954 up to 1987

Figure 4.4: Coastline development near La Ceiba

distance survey point—base line (1000 m)

Point A

distances between survey points is 1 km

× coastline in 1981 ▽ coastline in 1987

Point a is the 'zero point' of the baseline drawn in figure 3.1. All coastal changes are measured with respect to this line.
Figure 5.1: Overview of the various parameters determining the required depth of the access channel
Figure 5.2: Ship movement when entering a harbour due to cross currents
ANNEX 4.1: SHORT WAVE CALCULATIONS

The waves on shallow water can be calculated based on wave data at deep water by the following formula:

\[
\frac{H_1}{H_0} = \sqrt{\frac{c_0}{2n_1 c_1}} \times \frac{\cos \varphi_0}{\cos \varphi_1}
\]

\(\varphi_1\) wave angle at shallow water

\(\varphi_0\) wave angle at deep water

\(c_0\) Wave velocity at deep water
\(c_1\) Wave velocity at shallow water
\(n_1\) Ratio of wave group velocity at shallow water and \(c_1\)
\(H_1\) Wave height at shallow water
\(H_0\) Wave height at deep water

Table a: Wave height and wave angle at shallow water for waves at deep water from an angle of 0 degrees (angle between wave direction and north pole)

<table>
<thead>
<tr>
<th>Wave height at deep water (m)</th>
<th>Wave length at deep water (m)</th>
<th>Wave angle at deep water (degree)</th>
<th>Wave height when breaking (m)</th>
<th>Wave length when breaking (m)</th>
<th>Wave angle when breaking (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>19.1</td>
<td>0</td>
<td>0.54</td>
<td>8.9</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>24.9</td>
<td>0</td>
<td>1.01</td>
<td>13.2</td>
<td>0</td>
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<tr>
<td>1.5</td>
<td>39</td>
<td>0</td>
<td>1.53</td>
<td>20.8</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>56.2</td>
<td>0</td>
<td>2.07</td>
<td>28.8</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>99.9</td>
<td>0</td>
<td>2.72</td>
<td>45.3</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>126.5</td>
<td>0</td>
<td>3.32</td>
<td>55.4</td>
<td>0</td>
</tr>
<tr>
<td>3.5</td>
<td>156.1</td>
<td>0</td>
<td>3.89</td>
<td>67.5</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>224.8</td>
<td>0</td>
<td>4.63</td>
<td>88.8</td>
<td>0</td>
</tr>
</tbody>
</table>

Annex 4.1: Short wave calculations
Table b: Wave height and wave angle at shallow water for waves at deep water from an angle of 30 degrees (angle between wave direction and north pole)

<table>
<thead>
<tr>
<th>Wave height at deep water (m)</th>
<th>Wave length at deep water (m)</th>
<th>Wave angle at deep water (degree)</th>
<th>Wave height when breaking (m)</th>
<th>Wave length when breaking (m)</th>
<th>Wave angle when breaking (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>19.1</td>
<td>30</td>
<td>0.51</td>
<td>8.9</td>
<td>13.5</td>
</tr>
<tr>
<td>1</td>
<td>24.9</td>
<td>30</td>
<td>0.96</td>
<td>13.3</td>
<td>15.4</td>
</tr>
<tr>
<td>1.5</td>
<td>39.1</td>
<td>30</td>
<td>1.46</td>
<td>20.4</td>
<td>15.1</td>
</tr>
<tr>
<td>2</td>
<td>56.2</td>
<td>30</td>
<td>1.98</td>
<td>28.3</td>
<td>14.6</td>
</tr>
<tr>
<td>2.5</td>
<td>99.9</td>
<td>30</td>
<td>2.59</td>
<td>44.1</td>
<td>12.7</td>
</tr>
<tr>
<td>3</td>
<td>126.5</td>
<td>30</td>
<td>3.15</td>
<td>54.2</td>
<td>12.4</td>
</tr>
<tr>
<td>3.5</td>
<td>156.1</td>
<td>30</td>
<td>3.71</td>
<td>65.6</td>
<td>12.1</td>
</tr>
<tr>
<td>4</td>
<td>224.8</td>
<td>30</td>
<td>4.39</td>
<td>86.6</td>
<td>11.1</td>
</tr>
</tbody>
</table>

Table c: Wave height and wave angle at shallow water for waves at deep water from an angle of 60 degrees (angle between wave direction and north pole)

<table>
<thead>
<tr>
<th>Wave height at deep water (m)</th>
<th>Wave length at deep water (m)</th>
<th>Wave angle at deep water (degree)</th>
<th>Wave height when breaking (m)</th>
<th>Wave length when breaking (m)</th>
<th>Wave angle when breaking (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>19.1</td>
<td>60</td>
<td>0.42</td>
<td>7.7</td>
<td>20.2</td>
</tr>
<tr>
<td>1</td>
<td>24.9</td>
<td>60</td>
<td>0.78</td>
<td>12.3</td>
<td>25.2</td>
</tr>
<tr>
<td>1.5</td>
<td>39.1</td>
<td>60</td>
<td>1.18</td>
<td>18.8</td>
<td>24.7</td>
</tr>
<tr>
<td>2</td>
<td>56.2</td>
<td>60</td>
<td>1.59</td>
<td>26.2</td>
<td>23.8</td>
</tr>
<tr>
<td>2.5</td>
<td>99.9</td>
<td>60</td>
<td>2.10</td>
<td>40.2</td>
<td>20.4</td>
</tr>
<tr>
<td>3</td>
<td>126.5</td>
<td>60</td>
<td>2.54</td>
<td>49.4</td>
<td>19.8</td>
</tr>
<tr>
<td>3.5</td>
<td>156.1</td>
<td>60</td>
<td>2.99</td>
<td>59.3</td>
<td>19.2</td>
</tr>
<tr>
<td>4</td>
<td>224.8</td>
<td>60</td>
<td>3.55</td>
<td>78.1</td>
<td>17.5</td>
</tr>
</tbody>
</table>

For waves from -30 degrees and -60 degrees the same results are obtained as with the waves from 30 degrees and 60 degrees. (The wave angles are in this case to be multiplied with -1).

Annex 4.1: Short wave calculations
The formula of Chezy is as follows:

\[ u = C \times \sqrt{a \times i} \]

- \( u \) current velocity
- \( C \) bottom friction (constant)
- \( a \) water depth
- \( i \) slope

The bottom roughness can be calculated by use of the formula of white cool brook [re. 11]

\[ C = 18 \times \log \frac{12 \times h}{k_n} \]

- \( h \) water depth
- \( k_n \) \( D_{50} \) (parameter of Nikuradse)

The water depth is taken as 5 m. If \( D_{50} \) is taken as \( 280 \times 10^{-6} \) m (based on soil sample taken in the breaker zone near the Boca Vieja) the bottom roughness of the sea can be calculated as:

\[ C = 95.95 \, \text{m}^{0.5}/\text{s} \]

The distance between Puerto Cortes and Puerto Castilla is approximately 600 km. The time difference in occurrence of the maximum tide is estimated at 30 minutes.

The slope can be calculated by:

\[ i = \frac{\Delta h}{\Delta x} \]

\( \Delta h = \text{Water level difference between Puerto Castilla and Puerto Cortes} \)

\( \Delta x = \text{Distance between Puerto Castilla and Puerto Cortes} \)

The water level difference is estimated at 0.17 m. The distance is approximately 600,000 m. The slope is thus: \( 3 \times 10^{-7} \).

This results in a current velocity of 0.12 m/s.
The formula of Barras [re. 3] reads:

$$\delta = \frac{v^2}{2g} \times 3.75 \times C_b \times S_2^{0.75} \times \left( \frac{v}{V_s} \right)^{\frac{1}{12}}$$

\(v\) Vessel's speed  
\(V_s\) Vessel's service speed  
\(C_b\) Block factor (estimated at 1)  
\(g\) Gravitational acceleration (9.81 m\(^2\)/s)

\[S = \frac{b \times d}{B \times D}\]

\[S_2 = \frac{S}{1 - S}\]

\(b\) Beam of vessel  
\(d\) Maximum draught of vessel  
\(B\) Width of channel  
\(D\) Waterdepth in channel

Using a water depth of 6 m, a width of the channel of 120 m, a velocity of 3 m/s and a service velocity of 4 m/s gives a squat of

$$\delta = 0.10 \text{ m}$$
The calculation is based on the long wave theory in one dimension. The inlet can be schematized as follows:

![Diagram showing the bowl and access channel](image)

In the whole bay the water level will respond to the tide at the same time, thus causing no difference in water level at the same time in the bay. This is only true when the dimensions of the bay are small compared with the wave length (approximately 210 km) of the tide. In the case of the Boca Vieja this is true.

Therefore the current in point 2 can be calculated by a simple continuity equation (1) - The parameters are explained on the next page -:

\[ B \frac{\partial h(x_2, t)}{\partial t} + \frac{\partial Q(x_2, t)}{\partial x} = 0 \]

This formula can be integrated over x resulting in the following formula (2):

\[ Q(t) = A_p \times \frac{dh(x_2, t)}{dt} \]

The access channel however will have a current and thus a difference in water level. Now the momentum equation must be used (3):

\[ \frac{\partial Q}{\partial t} + \frac{\partial Q^2}{A} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2AxR} = 0 \]

Assuming that there is no storage of water in the access channel and integrating over x along the total length of the access channel one arrives at the following equation (4):

\[ h(x_2, t) - h(x_1, t) = -\frac{\partial Q}{\partial t} \times L - \frac{gQ|Q|}{C^2xA^2xR} \times L \]
In point 1 the boundary condition for the waterlevel at sea is given by the following equation (5):

\[ h(x_1, t) = h_{01} \times \cos(\omega \times t) \]

The solution of equation 4, with the boundary conditions given by equation 2 and 5 reads as follows:

\[ h(x_2, t) = h_{02} \times \cos(\omega \times t - \gamma) \]
\[ h(x_1, t) = h_{01} \times \cos(\omega \times t) \]

\[ k = \frac{8 \times Q_0 \times L}{3 \times \pi \times C^2 \times A^2 \times R} \]

\[ \gamma = \arctan\left(\frac{\omega \times k \times A_p}{1 - \omega^2 \times M \times A_p}\right) \]

\[ h_{02} = \frac{h_{01}}{\sqrt{(1 - \omega^2 \times M \times A_p)^2 + \omega^2 \times k^2 \times A_p^2}} \]

\[ Q(t) = A_p \times \omega \times h_{02} \times \cos(\omega \times t - \gamma + \pi/2) \]

in these formulas are:
- \( Q(t) \): discharge \( (m^3/s) \) on time \( t \)
- \( Q_0 \): Amplitude of the discharge \( Q(t) \)
- \( h(x_1, t) \): waterlevel at position \( x_1 \) and time \( t \)
- \( h(x_2, t) \): waterlevel at position \( x_2 \) and time \( t \)
- \( h_{01} \): amplitude of the tide in position 1
- \( h_{02} \): amplitude of the tide in position 2
- \( A_p \): The water surface of the bay
- \( A \): surface of the cross section of the access channel
- \( L \): Length of the access channel
- \( B \): Width of the access channel
- \( d \): Depth of the access channel
- \( C \): Chezy coefficient (friction coefficient)
- \( R \): Hydraulical radius
- \( \omega \): angular velocity of the tidal wave
- \( g \): gravity acceleration
- \( M \): \( L/g \times A_p \)

Annex 6.1: Tidal current computations in the Boca Vieja
For the case of the Soca Vieja the parameters have the following value:

\[ A_p = 150.000 \text{ m}^2 \]
\[ d = 5.85 \text{ m} \]
\[ L = 600 \text{ m} \]
\[ B = 60 \text{ m} \]
\[ A = 351 \text{ m}^2 \]
\[ h_{s1} = 0.17 \text{ m} \]
\[ R = 5.85 \text{ m} \]
\[ C = 92.21 \text{ m}^{0.5}/\text{s} \]
\[ \omega = 1.406 \times 10^{-4} \]
\[ g = 9.81 \text{ m}^2/\text{s} \]

The results are

\[ Q(t) = 3.595293 \cos(1.406 \times 10^{-4} \cdot t + 1.57079) \]

The maximum current will thus be:

\[ u = \frac{Q}{A} = 0.0102 \text{ m}/\text{s} \]

Thus one sees that the current in the Soca Vieja will be very low. Only with the rainfall the currents will increase. The maximum discharge due to rainfall, which occurred only once in a recording time of 20 years, is estimated at 185 m$^3$/s. The current will then be:

\[ u = \frac{Q}{A} = 0.4987 \text{ m}/\text{s} \]

This current will bring some sediment to the harbour (due to erosion of the land through which the rivers flow). At present it is not possible to estimate how much this sediment transport will be.

Annex 6.1: Tidal current computations in the Boca Vieja
Annex 6.2: Input data SANSIM, sediment transport at La Ceiba.
### Results SANSIM, sediment transport at La Ceiba

<table>
<thead>
<tr>
<th>Result</th>
<th>Bjiker</th>
<th>Cerc</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height (deep water)</td>
<td>Wave directions</td>
<td>Wave directions</td>
</tr>
<tr>
<td>H 0.5 m</td>
<td>65</td>
<td>35</td>
</tr>
<tr>
<td>H 1.0 m</td>
<td>1</td>
<td>40</td>
</tr>
<tr>
<td>H 2.0 m</td>
<td>69721</td>
<td>25950</td>
</tr>
<tr>
<td>H 2.5 m</td>
<td>42513</td>
<td>30210</td>
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<tr>
<td>H 3.0 m</td>
<td>42041</td>
<td>29763</td>
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<td>19727</td>
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</tr>
<tr>
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Annex 6.3:
# Scenario

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<td>——</td>
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<td>3 Date</td>
<td>06.05.1991</td>
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<tr>
<td>4 Scenario</td>
<td>Transport undisturbed</td>
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</tbody>
</table>

## Range

| 1 Wave height | 0.5 | 0.5 | 8 |
| 2 Wave direction | 68 | -38 | 9 |

## Environment

| 1 Mass density of water | 1025 |
| 2 Mass density of sand | 2650 |
| 3 D50 material size | 280 |
| 4 D90 material size | 640 |
| 5 Breaker index | 0.8 |
| 6 Bed roughness | 0.06 |
| 7 Byker coefficient | 5 |
| 8 Coastal orientation | 8 |

## Wave Period 60° 30° 0° -30° -60°

| 1 Hs = 0.5 m | [s] | 3.5 | 3.5 | 3.5 | 3.5 |
| 2 Hs = 1 m | [s] | 4 | 4 | 4 | 4 |
| 3 Hs = 1.5 m | [s] | 5 | 5 | 5 | 5 |
| 4 Hs = 2 m | [s] | 6 | 6 | 6 | 6 |
| 5 Hs = 2.5 m | [s] | 8 | 8 | 8 | 8 |
| 6 Hs = 3 m | [s] | 9 | 9 | 9 | 9 |
| 7 Hs = 3.5 m | [s] | 10 | 10 | 10 | 10 |
| 8 Hs = 4 m | [s] | 12 | 12 | 12 | 12 |

## Occurrence 60° 30° 0° -30° -60°

| 1 Hs = 0.5 m | [%] | 2.7 | 1.3 | 0.5 | 0.2 | 0.2 |
| 2 Hs = 1 m | [%] | 5.8 | 2.3 | 0.7 | 0.3 | 0.3 |
| 3 Hs = 1.5 m | [%] | 7.2 | 2.5 | 0.7 | 0.3 | 0.3 |
| 4 Hs = 2 m | [%] | 4.7 | 1.9 | 0.3 | 0.1 | 0 |
| 5 Hs = 2.5 m | [%] | 2.3 | 1.1 | 0.6 | 0.2 | 0 |
| 6 Hs = 3 m | [%] | 1.4 | 0.6 | 0.3 | 0.1 | 0 |
| 7 Hs = 3.5 m | [%] | 0.8 | 0.1 | 0.1 | 0 | 0 |
| 8 Hs = 4 m | [%] | 0.1 | 0.1 | 0.1 | 0 | 0 |

## Coastal Profile

| 1 Depth Point No. 1 | [m] | 0 | 8 |
| 2 Depth Point No. 2 | [m] | 18 | 0.97 |
| 3 Depth Point No. 3 | [m] | 61.5 | 0.98 |
| 4 Depth Point No. 4 | [m] | 97 | 1.48 |
| 5 Depth Point No. 5 | [m] | 127.5 | 2.23 |
| 6 Depth Point No. 6 | [m] | 162.5 | 2.8 |
| 7 Depth Point No. 7 | [m] | 195 | 1.42 |
| 8 Depth Point No. 8 | [m] | 225 | 1.91 |
| 9 Depth Point No. 9 | [m] | 259 | 4.46 |
| 10 Depth Point No. 10 | [m] | 290 | 4.8 |
| 11 Depth Point No. 11 | [m] | 327 | 5.15 |
| 12 Depth Point No. 12 | [m] | 352 | 5.36 |
| 13 Depth Point No. 13 | [m] | 382 | 5.64 |
| 14 Depth Point No. 14 | [m] | 450 | 6 |
| 15 Depth Point No. 15 | [m] | 550 | 6.5 |
| 16 Depth Point No. 16 | [m] | 625 | 7 |
| 17 Depth Point No. 17 | [m] | 748 | 7.5 |
| 18 Depth Point No. 18 | [m] | 850 | 8 |

Annex 6.4: Input data SANSIM, sediment transport at the Boca Vieja
Annex 6.5: Results SANSIM, sediment transport at the Boca Vieja

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<th>Wave height</th>
<th>Result</th>
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Bijker

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Annex 6.6: Clarification of annexes 6.7 up to 6.10

\( S_0 \) Total sediment transport. Sum of the westward transport and the eastward transport. Through an internal parameter COSIM calculates, which part of this total transport flows in eastward direction and which part in westward direction. \( \text{(unit m}^3/\text{year)} \)

\( S \) Actual sediment transport at a time. \( \text{(unit m}^3/\text{year)} \)

\( S/S_0 \) Ratio between the above mentioned parameters at a specific place and time. If at a given time the ratio increases between two points, erosion occurs at that interval, when reading the graphs from right to left. Sedimentation occurs in an interval at a given time, if the ratio decreases over this interval, when reading the graph from right to left. \( \text{(unit -)} \)

\( A_0 \) Average angle between wave direction and perpendicular of the coast. \( \text{(unit degree)} \)

\( Z_0 \) The active depth. Behind this depth contour no changes of the bottom profile take place due to changes in sediment transport: Pelnard Considère approach. \( \text{(unit m)} \)

All shore line changes are related to the ‘\( T = 0 \) yr coast-line’.

\( T \) Time parameter \( \text{(unit: year)} \)
\( y \) co-ordinate indicating the shore-line change at a point. \( \text{(unit m)} \)
\( x \) co-ordinate indicating the location of a point. \( \text{(unit m)} \)

Special points: (in all calculations)
\( x = 0 \) Location of present port of La Ceiba
\( x = 3600 \) Location of east side breakwater
Annex 6.7: Results COSIM, coast-line changes alternative I
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.7: Results COSIM, coast-line changes alternative I
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.7: Results COSIM, coast-line changes alternative I
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.7: Results COSIM, coast-line changes alternative I
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.7: Results COSIM, coast-line changes alternative 1
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.7: Results COSIM, coast-line changes alternative 1
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative 1
sediment transport 108,000 m³/year
For clarification: see annex 6.6

Scenario: Two breakwaters

PLAN VIEW

Time = 0 yr
So = -108000 m³/yr
Ao = -12 degr
Zo = 8 m

La Ceiba
Boca Viela
Annex 6.8: Results COSIM, coastline changes alternative I sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative I sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative I 
sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative 1
sediment transport 108,000 m³/year
For clarification: see annex 6.6
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COASTLINE SIMULATION MODEL

Annex 6.8: Results COSIM, coastline changes alternative I

For clarification: see annex 6.8
Annex 6.8: Results COSIM, coastline changes alternative I
sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative 1
sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.8: Results COSIM, coastline changes alternative I
sediment transport 108,000 m³/year
For clarification: see annex 6.6
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Annex 6.8: Results COSIM, coastline changes alternative I
sediment transport 108,000 m³/year
For clarification: see annex 6.6
Annex 6.9: Results COSIM, coast-line changes alternative II
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.9: Results COSIM, coast-line changes alternative II
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.9: Results COSIM, coast-line changes alternative II
Sediment transport 410,000 m$^3$/year
For clarification: see annex 6.6
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Annex 6.9: Results COSIM, coast-line changes alternative II
Sediment transport 410,000 m³/year
For clarification: see annex 6.6
Annex 6.10: Results COSIM, coastline changes alternative II
sediment transport 108,000 m³/year
For clarification: see annex 6.6
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For clarification: see annex 6.6
Annex 6.10: Results COSIM, coastline changes alternative II
sediment transport 108,000 m$^3$/year
For clarification: see annex 6.6
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