Review of a harbour layout
and tsunami research
in Golfo de Arauco
Preface

This report contains the results of a research project for graduation at the Faculty of Civil Engineering and Geosciences of the Technical University Delft.

The research is done at Delta Marine Consultants in Gouda. The research comprises the review of a harbour layout designed by DMC and the research of the behaviour of a tsunami wave in the bay the harbour is situated in.

A number of people were of great support. At first all the members of my graduation committee, Prof. Ir. H. Ligtering, Ir. R. Groenveld, Dr. Ir. N. Booy and Ir. J.S. Reedijk. They were a big help during the research process. Further I wish to thank all my family and friends for their support.

Annemieke Schimmel

Summary

The shipping company CSAV wants a new harbour to be built in Golfo de Arauco in Chile. The bay is situated directly south of the city Concepcion. DMC designed a layout for this harbour. Along the coast of Chile tsunamis can occur. The design made by DMC does not reckon with this phenomenon. In this report at first a review of the harbour layout made by DMC is done. After that research is done on the tsunami subject with the help of a simulation program. DMC proposed to use MIKE21 because it was available as an entry level version at the time of the research.

The review of the harbour leads to an increase of the number of container berthing places based on the used starting points. The container storage area is increased as well.

The tsunami is described with the help of empirical relations. An aftershock generates a tsunami area after which it travels towards the coast of Chile and towards the ocean where it even can reach Japan. The aftershock area is schematised to a one-dimensional model. Whit the help of this model is made clear that the height of the tsunami will be larger when the vertical amplitude of the aftershock area is larger. Golfo de Arauco is implemented into the simulation model as well. It is made clear that the tsunami will bend along the island Santa Maria that is situated in the bay area. The site of the harbour is the first area on the coast hit by the tsunami. The height of the tsunami is not fully developed however, because of the trench that is situated perpendicular to the coast north of the site.

To minimise the damage of the tsunami the orientation of the containers on the terminal must be changed. The traffic lanes must have an orientation in the propagation direction of the tsunami. The tank farm must be placed under the ground.
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1. Introduction

In this first chapter an overall description is given of this report. In chapter 2 a description of the Bio Bio project is given as treated by DMC. In chapter 3 the problem handled in this report is described followed by the approach of the research process. In chapter 4 a review of the harbour layout for the Bio Bio project as designed by DMC is done. In chapter 5 the characteristics of tsunami are determined. A description of the simulation program MIKE21 is included in this chapter also. In chapter 6 the results of the simulation are described. The conclusions and recommendations are described in the last chapter.
2. The Bio Bio project

In this chapter a description is given of the harbour project Bio Bio as developed by DMC.

2.1. General description of Chile

Republica de Chile extends over 4275km from north to south and has a width of 120km. It is a long stretched country with clearly perceptible boundaries as is shown in figure 2.1: the Pacific Ocean on the west wide and the Andes Mountain range on the east side. There only is one major highway running from the north to the south, called the Pan-American Highway. In the past ten years the economy of Chile has grown with an average of 6% per year. The number of people living in poverty is decreasing and goods as wine, grapes and wood-products are exported all over the world. The country is divided into thirteen regions of which Bio Bio is the most important for this report.

2.2. Bio Bio project description

DMC received an assignment from Puerto Industrial del Bio Bio S.A. (PBB) to develop a conceptual harbour layout for a new harbour in Golfo de Arauco, which is situated in the region Bio Bio. A picture of the bay area is presented in figure 2.2. The bay situated just below the river Bio Bio and the city of Concepcion. Sudamericana Agencias Aereas y Maritimas S.A. (SAAM), which is owned by a shipping company called CSAV, established PBB especially for this occasion. PBB acquired a piece of land called Lomas Colorades for the new harbour. The coordinates of the site are 36°51’ S, 73°09’ W. The site has a rectangular shape and measures approximately 250 hectares. The north boundary of the site lies 3km south of the river mouth. The Pan-American Highway is the east boundary and the ocean is the west boundary of the site.

The majority of the harbours in Chile are owned by the state. In this project the shipping company CSAV finances the harbour. This shipping company owns the vessels that will use the port. The investment in the harbour must be earned back by the harbour itself. The

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solution for this problem is found in real estate. The biggest part of the surface area of the site is reserved for real estate development. The client PBB considers this a very important factor in the design process.

In Golfo de Arauco an island called Santa Maria is situated. This island is situated in front of the site and is protecting the harbour from waves from the western direction. Interesting is the role the island will have in the tsunami research.

Figure 2.2 Golfo de Arauco with the site Lomas Colorades
3. **Project definition**

In this chapter the approach of the project treated in this report is described. At first a problem definition is given. After that the objectives are described followed by the starting points to begin with the research process.

### 3.1. Problem definition

DMC presented a layout for the harbour in Golfo de Arauco. The layout is designed without taking into account the effects of a tsunami attack. The influence of the shape of the bay and the shape of the foreshore on the propagation of the tsunami is not known.

### 3.2. Definition of objectives

The major objective of this project is to provide a better layout for the harbour at Lomas Colorades in Golfo de Arauco.

The major objective is reached with the help of two sub-objectives:
- The layout designed by DMC is to be reviewed;
- The propagation of the tsunami in front of the coast of Golfo de Arauco is to be researched by making use of the simulation program MIKE21.

### 3.3. The approach of the project

The research can be expressed in four different questions:
- What are the improvements after reviewing the harbour?
- What is the development of the tsunami in the bay area and what is the height of the tsunami in front of the site?
- What arrangements should be made to minimise the damage caused by a tsunami attack?

At first the layout designed by DMC is reviewed. The following restrictions are made before reviewing.
- Measurements show that no waves with a period larger than 30 seconds are detected in the area;
- Morphological impact is not taken into account;
- Environmental impact is not taken into account;
- The costs are not taken into account;
- The soil is suitable for building the harbour;
- Effects caused by El Niño are not taken into account;
- The logistics on the terminal is not taken into account;
- The location of the site is a fact;
- The site is situated in a bay. This means that the so called Peru Current has no influence on the site as is shown in figure 3.1;
- The effect of an earthquake on the superstructures of the harbour is not included in this report.
After reviewing the layout the research on the tsunami subject will be started. At first literature is studied to find out the generation mechanism of a tsunami. An analytical description of the tsunami in general will be given followed by a tsunami that is used in the simulation program MIKE21. The simulation is used to find out what the effect of the shape of the bay and the foreshore will be on the propagation of the tsunami. Finally the height of the tsunami in front of the site will be determined which is important for the final design of the harbour layout. The following restrictions are made before the research is started:

- The propagation direction of the tsunami is perpendicular to the coast;
- The tsunami is caused by an earthquake;
- The tsunami is generated directly in front of Golfo de Arauco and has not travelled over a long distance before it reaches the bay area.
4. The review of the harbour layout

The harbour layout as designed by DMC will be reviewed in this chapter. At first a description is given followed by the boundary conditions. After that the actual review is made.

4.1. Description of the present harbour layout

A drawing of the layout designed by DMC is presented in figure 4.1.

![Harbour layout designed by DMC](image)

Figure 4.1 Harbour layout designed by DMC
The entrance of the harbour is headed 230°N. The approach channel is based on single lane traffic and the bottom level is CD-15m. It starts at the outer breakwaters and ends at the inner breakwater. The cross sections of the breakwaters are presented in appendix 4A. The stopping distance is counted from the inner breakwater until the slope at the end of the turning basin. In case of an emergency the vessel will run up this slope. Another slope is found at the end of the harbour basin. The function of this slope is to absorb the waves that will occasionally come into the harbour.

When the vessel approaches the entrance of the harbour, tugboats are used to lead the ship correctly into the harbour. The turning circle is used for stopping and turning the vessel in the right direction. The orientation of the harbour basin is a wish of the client PBB and is in such a way that the vessel can navigate calmly towards the berth. The navigation towards the berth is tug assisted at any time. The bottom level of the harbour basin is CD-14m.

The quay wall is a concrete deck on piles. A drawing of the quay wall is presented in appendix 4B. The slender construction is designed to respond more flexible to seismic loads. The deck on piles offers a wave absorbing quay wall construction. The rubble mound slope underneath the deck will absorb waves, which penetrate into the harbour basin.

The harbour consists of four terminals. Three terminals are situated on the West Side of the harbour and one terminal is situated on the East Side of the harbour.

The terminals on the West Side are the liquid bulk terminal, the bulk terminal and the general cargo terminal.

The liquid bulk terminal consists of a tank farm with 6 tanks. It is assumed that all petroleum products allow for transport through the same pipeline. Space is reserved for a pump house, metering facilities, fire fighting provisions and vapour recovery. The dry bulk, coal and wood pulp is exported only. The cargo is for 100% delivered by road trucks.

The general cargo terminal consists of two berths. The general cargo consists of wood and wood products and steel products for export and other products for both import and export. The hinterland transport is divided in 80% road and 20% rail transport. Road trucks are loaded and unloaded immediately adjacent to the storage position in the yard. Trains are loaded and unloaded by forklifts.

At the East Side of the harbour basin the container terminal is situated. This the most important terminal of the harbour. The terminal has an empty container storage yard and a full container storage yard. Less than 5% of the containers will be reefers, which are cooled containers.

Each terminal on the site is developed as a separate entity, all with their own road accesses etc. This provides the possibility to issue concessions to outside parties for operating one or more of the terminals, instead of keeping all operations in one hand.

The railroad runs along the South side of the harbour, passing the container terminal, the backside of the harbour basin, turning across the general cargo terminal to the bulk terminals. All terminals are reached in this way. The road runs besides the railway and also has access to all terminals.
4.2. **Boundary conditions**

To start with the review of the harbour it is necessary to determine the boundary conditions. They are different for each harbour and therefore very important. The boundary conditions for this harbour are presented in two categories, the physical site conditions and the terminal layout restrictions.

4.2.1. **Physical site conditions**

4.2.1.1. Wave climate

DMC translated the offshore data into a nearshore wave climate. The results are presented below. The offshore data were obtained from the British Meteorological Office (BMO).

- The scatter diagram for the nearshore wind wave heights is presented below:

<table>
<thead>
<tr>
<th>$H_s$</th>
<th>[m]</th>
<th>180 N</th>
<th>210 N</th>
<th>240 N</th>
<th>270 N</th>
<th>300 N</th>
<th>330 N</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 TO 0.25</td>
<td>0.145</td>
<td>0.011</td>
<td>0.011</td>
<td>0.072</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>0.25 TO 0.75</td>
<td>0.142</td>
<td>0.128</td>
<td>0.040</td>
<td>0.035</td>
<td>0.050</td>
<td></td>
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<tr>
<td>0.75 TO 1.25</td>
<td>0.003</td>
<td>0.024</td>
<td>0.080</td>
<td>0.016</td>
<td>0.019</td>
<td>0.005</td>
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<tr>
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<td>0.015</td>
<td>0.009</td>
<td>0.011</td>
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<tr>
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<td>0.004</td>
<td>0.007</td>
<td>0.004</td>
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<tr>
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<td>0.002</td>
<td>0.004</td>
<td>0.001</td>
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<tr>
<td>2.75 TO 3.25</td>
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<td>0.003</td>
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<td>0.290</td>
<td>0.160</td>
<td>0.100</td>
<td>0.085</td>
<td>0.094</td>
<td>0.145</td>
<td>0.874</td>
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<td>Calm</td>
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</tr>
</tbody>
</table>

**Table 4-1 Nearshore wind wave height scatter diagram (% of time)**

The directions in the first row are averages. For example 270°N means that waves from directions 255°N till 285°N are included.
The scatter diagram for the nearshore swell wave height is presented below.

<table>
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<tr>
<th>Hs</th>
<th>[m]</th>
<th>240 N - 260 N</th>
<th>260 N - 280 N</th>
<th>280 N - 300 N</th>
<th>Total</th>
</tr>
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<td>0.0 TO 0.25</td>
<td>0.378</td>
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</tr>
<tr>
<td>0.25 TO 0.75</td>
<td>0.156</td>
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<tr>
<td>0.75 TO 1.25</td>
<td>0.002</td>
<td>0.157</td>
<td>0.115</td>
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<tr>
<td>1.25 TO 1.75</td>
<td>0.037</td>
<td>0.020</td>
<td>0.022</td>
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</tr>
<tr>
<td>1.75 TO 2.25</td>
<td>0.019</td>
<td>0.010</td>
<td>0.006</td>
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<td></td>
</tr>
<tr>
<td>2.25 TO 2.75</td>
<td>0.015</td>
<td>0.008</td>
<td>0.001</td>
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</tr>
<tr>
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<td>0.006</td>
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<td>3.25 TO 4.25</td>
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<tr>
<td>20.25 OR MORE</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>0.607</td>
<td>0.200</td>
<td>0.144</td>
<td>0.951</td>
<td></td>
</tr>
<tr>
<td>Calm</td>
<td>0.049</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4-2 Nearshore swell wave scatter diagram (% of time)

4.2.1.2. Extreme wave heights

The extreme wave heights were obtained with the help of the Gumbel and Weibull distributions. They were used to determine the wave height with a certain return period. In this case the return period was 100 years. The reason for the use of these functions was that they lead to a good fit of the wave height distribution. The significant periods of the extreme wave heights were determined with the help of the wave steepness.

\[
\text{wave steepness} = \frac{2\pi H_s}{gT_p^2}
\]  
(4.1)

In this expression the peak period instead of the significant period is presented. The significant period is approximately 0.9 times the peak period. Assumed was that the wave steepness of the swell waves is 0.02 and the wave steepness of the wind waves is 0.04. The results were:
• The extreme significant swell wave height with a return period of 100 years on the 20m contour is 6.0m with a period of 14.4 seconds from the WNW direction;
• The extreme significant wind wave height with a return period of 100 years on the 20m contour is 7.0m with a period of 11.3 seconds from the WNW direction.

4.2.1.3. The tide
• The tidal velocity is less than 0.1m/s;
• PBB provided the following tide levels:
  | HAT     | CD + 2.01m | High Astronomic Tide |
  | MHWS    | CD + 1.88m | Mean High Water Spring |
  | MSL     | CD + 0.92m | Mean Sea Level |
  | LAT     | CD + 0.23m | Low Astronomic Tide |
• In Chile Nivel Reduccicon de Sondas (NRS) is used which is equal to CD. The drawings made by DMC use the Chilean NRS.

4.2.1.4. Bathymetry
• The coastline is a straight beach aligned almost N-S, slightly curved to the West at Punta Coronel in the South and the mouth of the river Bio Bio to the North. Generally, the beach looks highly stable in its current situation;
• A steep beach profile approximately 1:6 at the still water, 1:10 at the breaker zone, falling slightly to 1:15 at a water depth of 10m;
• The river Bio Bio flows along a geological fault. This fault can be traced tens of kilometres offshore. At a distance of 1.5km from the mouth of the river, the depth already exceeds 150m;
• The bottom contour of the 15, 20 and 25m strongly bends westward in the area of the river mouth. The 25m contour heads approximately 320°N.

4.2.2. Terminal and marine infrastructure layout restrictions

4.2.2.1. Terminal layout conditions
• The total volume of cargo projected for the year 2005 for the harbour is presented in the table below. The amount of cargo was provided by PBB;

<table>
<thead>
<tr>
<th>Type of cargo</th>
<th>Tonnes per year</th>
<th>Percentage of total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Containers</td>
<td>8,000,000</td>
<td>49%</td>
</tr>
<tr>
<td>General Cargo</td>
<td>3,400,000</td>
<td>21%</td>
</tr>
<tr>
<td>Dry Bulk</td>
<td>2,500,000</td>
<td>15%</td>
</tr>
<tr>
<td>Liquid Bulk</td>
<td>2,500,000</td>
<td>15%</td>
</tr>
<tr>
<td>Total</td>
<td>16,400,000</td>
<td>100%</td>
</tr>
</tbody>
</table>

Table 4-3 Total volume of cargo for the year 2005

• The vessels are of different scale and sizes varying from 2000 DWT till 65000 DWT. The dimensions of the maximum and design vessels are presented below;
**Buiter:**
design vessel 46500 DWT: \( B = 32.5 \text{ m}, D = 11.8 \text{ m}, L_{oa} = 199 \text{ m}. \)
maximum vessel 65000 DWT: \( B = 32.5 \text{ m}, D = 12.0 \text{ m}, L_{oa} = 230 \text{ m}. \)

**Container vessel:**
design vessel 40000 DWT: \( B = 32.3 \text{ m}, D = 11.8 \text{ m}, L_{oa} = 225 \text{ m}. \)
maximum vessel 41000 DWT: \( B = 32.3 \text{ m}, D = 11.8 \text{ m}, L_{oa} = 242 \text{ m}. \)

The design vessel is used to determine the length of the quay wall except when the quay wall consists in only one berth. In that case the maximum vessel is used. The maximum vessel is also used to determine the dimensions of the turning circle for example.

- The maximum vessel can contain 3500 TEU;
- The client PBB wanted the harbour to be equipped with 8 berthing places divided in 4 container terminal berthing places, 2 general cargo berthing places, 1 liquid bulk berthing place and 1 dry bulk berthing place;
- The TEU-factor is 1.4, this means 60% 20-foot containers and 40% 40-foot containers;
- At the container terminal no stripping, stuffing, repairs, etc. occurs. Only container handling and storage is done;
- Assumed is that the containers can contain all sorts of cargo. The average weight of a container was assumed to be 11 tonnes;
- The general cargo terminal does not have to be rectangular;
- Each terminal is developed as a separate entity;
- In Chile 2 public holidays exists, one year comprises 350 working days;
- The operation is round the clock.

4.2.2.2. **Marine layout conditions**

- The harbour will be built inshore;
- The width of the approach channel and entrance channel is based on single lane traffic;
- This width of the harbour basin is based on tug assisted berthing;
- The vessels are assisted when manoeuvring towards the berths by push and pull tugs. A push tug can not be used for making fast when \( H_z > 1.0 \text{ m} \);
- A pull tug can not be used for making fast when \( H_z > 2.0 \text{ m} \);

4.2.2.3. **Site layout restrictions**

- The harbour is financed by a private party and not as most harbours in Chile by the government, therefore PBB wants as much land of the site as possible to be used for real-estate development;
- On the north side of the harbour basin a corridor of at least 75m width has to be provided, which is a wish of the client PBB;
- The design life for the harbour is 100 years, except for the quay walls that have a design life of 30 years.
4.3. **Review of the design layout**

4.3.1. **The orientation of the harbour entrance**

From the scatter diagrams of the swell and the wind waves the average wave heights for different directions were obtained. This was called the operational wave climate. The tables below show the average significant wave heights.

<table>
<thead>
<tr>
<th>Direction</th>
<th>$H_{avg}$</th>
<th>Occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td>240°N - 260°N</td>
<td>0.43 m</td>
<td>60.0%</td>
</tr>
<tr>
<td>260°N - 280°N</td>
<td>1.23 m</td>
<td>20.0%</td>
</tr>
<tr>
<td>280°N - 300°N</td>
<td>1.13 m</td>
<td>14.4%</td>
</tr>
</tbody>
</table>

**Table 4-4 Average swell wave heights and occurrences**

<table>
<thead>
<tr>
<th>Direction</th>
<th>$H_{avg}$</th>
<th>Occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td>180°N</td>
<td>0.12 m</td>
<td>29%</td>
</tr>
<tr>
<td>210°N</td>
<td>0.63 m</td>
<td>16%</td>
</tr>
<tr>
<td>240°N</td>
<td>1.13 m</td>
<td>10%</td>
</tr>
<tr>
<td>270°N</td>
<td>0.89 m</td>
<td>8.5%</td>
</tr>
<tr>
<td>300°N</td>
<td>1.10 m</td>
<td>9.4%</td>
</tr>
<tr>
<td>330°N</td>
<td>0.49 m</td>
<td>14.5%</td>
</tr>
</tbody>
</table>

**Table 4-5 Average wind wave heights and occurrences**

In appendix 4C these figures are expressed in four graphics. Most swell waves are from direction 240°N-260°N. These waves have a height of 0.43m, which is not very high. The biggest swell waves are from direction 280°N-300°N. They occur 14.4% of the time. The orientation of the entrance of the harbour, 230°N, is well chosen when considering the swell waves. The wind waves are travelling almost one third of the time from direction 180°N, or South, towards the harbour. These wind waves are locally generated and do not have a large wave height because of the limited fetch. The orientation of the harbour entrance is chosen well too, when considering the wind wave distribution.

The extreme waves travel from direction WNW towards the harbour. They can not penetrate into the harbour but are stopped by the northern breakwater.

4.3.2. **The breakwaters**

The coast in front of the site is very steep. This means that breakwaters extending far out of the coastline will be very expensive. The breakwaters reach until the 15m contour line. A little overtopping is allowed. The breakwaters have a height of NRS+5.30m, which is sufficient. When the breakwaters are constructed with the help of land based equipment the height of the breakwater above NRS must at least be 1m above NRS+1.88m.

4.3.3. **The approach channel**

The largest draught of the design vessels is 12m.
Lowest Astronomical Tide – draft vessel – squat and trim – dredge tolerance – 0.5*Hs = CD+0.23m-12m-1m-1m-0.5*2m=CD-14.77m≈CD-15m.
The width of the approach channel is based on single lane traffic. This means that the width of the approach channel is equal to 5 times the width of the largest vessel. This means 5*32.5m=162.5m. This is provided for in the design.

4.3.4. The stopping distance

The stopping distance in the original layout of the harbour has a value of 670m. The distance is taken from the inner breakwater till the end of the turning circle. A vessel will enter the approach channel after passing the outer breakwaters with a speed of at least 2m/s. Tugboats are lining up in between the inner and outer breakwaters. This procedure will take 10 to 15 minutes. Meanwhile the vessel travels with a speed of 2m/s into the harbour. The ship has travelled 1200m to 1800m when the lining up is finished. This much space is not provided for in the harbour layout.

The stopping procedure has to be changed. Lining up outside the harbour is a possibility. After that the vessels will enter the harbour tug-assisted. Lining up outside the harbour requires a maximum significant wave height of 1m for push tugs and 2m for pull tugs. The swell waves are not considered because of their long wavelengths. The movements of the tugs are more influenced by the wind waves. The wavelength of the wind waves can cause rapid movements that will trouble the lining up procedure.

In figure 4.2 the chance of non-exceedence of the significant wave height is presented. The chance that the significant wind wave height is smaller than 2m is 98.2%. The chance that the significant wave height is less than 1m is even 90.2%. This means that the tug boats are able to make fast outside the harbour in 98.2% of the time.

![Figure 4.2 Chance of non-exceedence of Hs](image)
4.3.5. The turning circle

The turning circle dimension depends on the size of the biggest ship that enters the harbour. The diameter of the turning circle is twice the length of the vessel, which is an average estimate. This means $2 \times 242m = 484m$. The actual size is 450m, which is sufficient.

4.3.6. The harbour basin

The harbour basin depth was determined the same way as the depth of the approach channel without the squat, because of the low velocity of the vessel. The depth of the harbour basin will be CD-14m.

4.3.7. Terminal dimensions

4.3.7.1. Container terminal dimensions

The container terminal will handle import and export containers. The export containers are either full or empty. More goods are imported than exported. The harbour is supposed to be a closed system for container handling. This implies that the inflow and outflow of containers must be equal to each other. The container handling is divided into three different flows and is stored on the terminal in three different cargo storage areas. The percentages of the different flows are:

- Import full containers: 50%
- Export full containers: 5%
- Export empty containers: 45%

In the figure below the container flow is illustrated.

![Container Flow Diagram](image)

**Figure 4.3 Container flow**

- Number of containers handled every year

The estimate for the year 2005 for container handling is 8,000,000 tonnes in a year. The average weight of a full container is assumed to be 11 tonnes per TEU and the TEU factor
1.4. With these boundary conditions the total number of containers handled every year is determined. A full container has an average weight of 11 tonnes. The container handling consists of 55% full containers. This means that the average weight of a container when the empty containers are included is 0.55 * 11 = 6.05 tonnes.

\[
\frac{8,000,000 \text{ tonnes per year}}{6.05 \text{ tonnes} \times 1.4} = 944,510 = 945,000 \text{ containers or } 1,322,300 \text{ TEU}
\]

- Surface area of the container terminal
The containers are stored in three different areas, which correspond with the different flows. The terminal surface area is divided in an export, import and an empty container storage area. The number of TEU per year is divided in the three categories:

Import full containers: 661157 TEU;
Export full containers: 66116 TEU;
Export empty containers: 595041 TEU.

For each category the storage area needed is determined with the help of the following equation\(^2\).

\[
O = \frac{C_i \bar{t}_d F}{r \cdot 365 \cdot m_i}
\]

(4.2)

with:
\(O\) = area required in m\(^2\).
\(C_i\) = number of container movements per year per type of stack in TEU.
\(\bar{t}_d\) = average dwell time in days.
\(F\) = required area per TEU inclusive of equipment travelling lanes in m\(^2\).
\(r\) = average stacking height/nominal stacking height (0.6 - 0.9).
\(m_i\) = acceptable average occupancy rate (0.65 - 0.80).

The dwell time of a full container is 7 days and of the empty container it is 14 days. The empty containers can be stacked higher than full containers. Straddle carriers are used to handle the full containers which leads to a value of \(F = 10.2 \text{ m}^2/\text{TEU}\). Empty containers are stacked 5-high and 10 deep, this is done with the help of reach stackers which results in a value of \(F = 4.6 \text{ m}^2/\text{TEU}\).

Full containers:

\[
O_{\text{full, import}} = \frac{661157 \times 7 \times 10.2}{0.8 \times 365 \times 0.70} = 230952 \text{ m}^2 = 231,000 \text{ m}^2
\]

\[
O_{\text{full, export}} = \frac{66116 \times 7 \times 10.2}{0.6 \times 365 \times 0.70} = 30793 \text{ m}^2 = 31,000 \text{ m}^2
\]

\(^2\) Velsink, Prof. Ir. H.; Ports and Terminals, Planning and Functional design; TU Delft, 1996.
Empty containers:

\[ O_{\text{empy}} = \frac{595041 \times 14 \times 4.6}{0.9 \times 365 \times 0.80} = 145817 m^2 = 146,000 m^2 \]

The overall storage area covers 409,000 m². This number does not include space for the apron. The total area covering the container terminal is in order of 450,000 m². The surface area on the drawing of the harbour designed by DMC is 320,000 m². This is not enough and should be expanded based on the used starting points.

- **Number of vessels**
A total of 1,322,300 TEU containers is handled every year. This amount is imported and exported by vessel varying in size from 2,000 DWT till 60,000 DWT. The biggest ship has a capacity of 3500 TEU. Assumed is that the average vessel contains 1000 TEU and has a parcel size of 70%. This means that the ship will unload 700 TEU and load 700 TEU. A total of 1400 TEU per average vessel is handled. The number of average vessels needed to handle the total number of containers is 945.

- **Determination of the berth occupancy rate**
The container terminal is equipped with 4 berthing places and the berths receive 945 vessels every year. The handling time of an average vessel is dependent on the number of cranes at the quay. Assumed is that two cranes will handle one vessel. The handling time of an average vessel is:

\[ \text{handling time} = \frac{1000}{2 \times 480} = 1.04 \text{ days} \]

The berth occupancy can be determined.

\[ \text{berth occupancy} = \frac{\text{number of vessels} \times \text{handling time}}{\text{number of berths} \times \text{workdays}} = \frac{945 \times 1.04}{4 \times 350} \times 100\% = 70\% \]

The berth occupancy is too high based on the used starting points. The waiting time of a vessel will be too long, which is explained on the next page. The number of berthing places must be increased to 5 berths, the occupancy rate than decreases to:

\[ \text{berth occupancy} = \frac{\text{number of vessels} \times \text{handling time}}{\text{number of berths} \times \text{workdays}} = \frac{945 \times 1.04}{5 \times 350} \times 100\% = 56\% \]

This is an acceptable berth occupancy rate.

- **Number of cranes on the berth, handling time of the average vessel.**
At first the number of containers handled by one crane is determined with the following parameters:
The net production of a crane is 20 moves/hour;
This includes loss of time due to opening and closing of holds, moving of the cranes and
time in between shifts. In India and America the productivity of the cranes is in between 25
and 30 moves per hour. In Australia this is much lower, here the productivity is 18 moves
per hour. The estimate of 20 moves per hour is acceptable³.
Production per day is 24 hours;
Production per year is 350 days.

The net production of one crane per day is 20*24 = 480 containers. The total of 945,000
containers is handled by 945,000/480*350 = 5.6 ≈ 6 cranes on 5 berthing places.
This would cause probably an unacceptable waiting time when all berths are busy. Each
berth must have 2 cranes leading to 10 cranes for the container terminal.

- Waiting time.
The waiting time is dependent on the number of vessels calling the port, the service time of
the average vessel, the number of berths and the berth occupancy rate. The waiting time is
determined in the so-called E₂/E₂/5-system. The first letter implies an Erlang inter arrival
time distribution of the vessels. The second letter implies an Erlang service time distribution
of the vessels. For the container terminal it is normal to use the Erlang distribution. The
vessels are of different sizes and the inter arrival time is variable too.

With 4 berths the berth occupancy is 70%. A total of 945 vessels are calling the port each
year. The service time of an average vessel is 25 hours. With help of an average waiting
timetable⁴ the waiting time is expressed in units of average service time. The average waiting
time is 0.1441*25 = 3.6 hours = 3 hours and 36 minutes.

With 5 berths the berth occupancy is 56%. The average waiting time is 0.0297*25 = 0.74
hours = 45 minutes.

The waiting time is too high when 4 berths are used. It is recommended to increase the
number of berths to 5.

- Amount of cargo in 2005
The estimate of 8,000,000 tonnes per year for the throughput of containers for the year 2005
is provided by PBB. An extrapolation is made of the cargo that will be handled based on the
cargo handled on this moment in the harbours in this region. This extrapolation shows that
the throughput in 2005 is in order of 1,200,000 tonnes per year. The starting points of this
project must be reconsidered. The precise numbers for the cargo volume is presented in
appendix 4D.

4.3.7.2. General cargo terminal dimensions
This terminal will be used for wood, wood products, steel products and others. The total
amount of transshipment has a weight of 3,400,000 tonnes in a year handled on 2 berthing
places, which is a wish of the client PBB.

³ Mundy M.; Quay Measures; Port Development International, July/August 1997, pages 17-23.
Determination of the dimensions of the surface area of the terminal

The required storage area is determined with the following formula:

\[ O_n = \frac{f_1 \cdot f_2 \cdot T_n \cdot t_{sw}}{m_n \cdot b \cdot \rho \cdot 365} \]  

(4.3)

with:
- \( O_n \) = required floor area in \( m^2 \);
- \( f_1 \) = proportion gross/net surface in connection with traffic lanes and forklifts, etc;
- \( f_2 \) = bulking factor due to stripping and separately stacking of special consignments;
- \( T_n \) = fraction of total annual tonnage \( T \) which passes the transit shed;
- \( t_{sw} \) = average dwell time of the cargo in days;
- \( m_n \) = average rate of occupation of the transit shed storage (e.g. 0.6);
- \( h \) = average stacking height (e.g. 5 m);
- \( \rho \) = average relative density of the cargo as stowed in the ship;

The dwell time of the cargo is 14 days and the average stacking height can be as much as 5 m.

\[ O_s = \frac{1.5 \times 1.2 \times 3,400,000 \times 14}{0.75 \times 5 \times 0.6 \times 350} = \frac{108,800}{109,000} m^2 = 109,000 m^2 \]

The surface area consists in transit sheds and open storage as well. It depends on the sort of cargo. Steel products can be stored in open storage areas, but not to long due to the corrosion danger. The apron width is 30 m alongside the quay wall. This covers an area of 15000 m². Approximately 15% of the space is reserved for trucks, trains, parking, unloading and loading. The total area needed for the general cargo terminal is finally 140,000 m².

The surface area on the layout presented by DMC is 142,500 m² and is matching with the above.

- Number of vessels

Assumed is that an average vessel contains 12,600 tonnes. This means that 270 vessels every year are needed. The size of an average vessel is 20,000 DWT.

- The berth occupancy

Each berthing place is equipped with 2 cranes. Each crane has a lift capacity of 9 tonnes and works with a speed of 25 cycles per hour.

Each crane handles \( 9 \times 25 \times 24 = 5400 \) tonnes/day.

The berth capacity is \( \frac{3,400,000}{4 \times 5400 \times 350} \times 100\% = 45\% \).

- Determination of the length of the quay wall

The same procedure as in the section handling the container terminal is used. The length of the quay wall is \( 1.10 \times (\text{number of vessels} \times \text{vessel length}) + 15 \times (\text{number of vessels} - 1) \).
This means $1.10 \times 2 \times 199 + 15 = 453$ m. In the design of DMC a quay wall length of nearly 500 m is used. This is sufficient.

- **Waiting time**
  Again the waiting time is determined in the so-called $E_2/E_2/5$ system. Two berthing places handle 270 vessels each year. The berth occupancy rate is 45% and the average service time is 1.5 days. The average waiting time is $0.08785 \times 1.5$ days $= 0.13$ days $= 3$ hours and 10 minutes. This is a bit high in my opinion. The capacity of the cranes must be increased.

4.3.7.3. **Bulk terminal dimensions**

This terminal will be used for export only. The products to be exported are coal and wood pulp. The total amount of cargo handled in a year at this terminal will be 2,500,000 tonnes.

- determination of the dimensions of the surface area of the terminal
  Expression (4.3) can be used for the dry bulk terminal to. The average stacking height is 10 m and the dwell time of the cargo is 21 days.

$$O_n = \frac{f_1 \cdot f_2 \cdot T_n \cdot \bar{t}_u}{m_u \cdot b \cdot \rho \cdot 365} = \frac{1.5 \times 1.2 \times 2,500,000 \times 21}{0.75 \times 10 \times 0.75 \times 350} = 48,000 \text{ m}^2$$

Another 15% is necessary for trucks, loading, unloading and parking which brings the total surface area on approximately 55,000 m$^2$. The floor area according to the layout of DMC is 73,000 m$^2$. Enough room is available.

- **Number of vessels**
  The average vessel is loaded with 18,000 tonnes. This means that 139 vessels are needed. The shiploader has a capacity of 1500 tonnes/hour. This means that the vessel is loaded in 12 hours.

- **The berth occupancy.**
  The coal and wood pulp is exported only. The cargo is transported by road trucks to the terminal and loaded into the vessel with the help of a shiploader. This can be a radial shiploader, which will be less expensive than other alternatives. The shiploader will turn around one point and does not need a crane lane. A telescopic boom that can reach the ships hull is needed because of the dusty character of the cargo. The fall velocity should be reduced to a minimum to prevent the cargo from swirling up. The capacity of the shiploader is in the range of 1500 - 2000 tonnes/hour.

  The berth occupancy is $\frac{2,500,000 \text{ tonnes}}{1500 \times 24 \times 350} \times 100\% = 20\%$.

- **The length of the quay wall**
  The quay wall consists in one berth only, thus the length of the quay wall will be 1.10 * vessel length. This means $1.10 \times 199 = 219$ m. DMC uses a length of 285 m which will be sufficient.
• Waiting time
The berth occupancy is very low. No problems with the waiting time are expected.

4.3.7.4. Liquid bulk terminal dimensions
The liquid bulk terminal will handle import and export of petroleum products. The total amount of cargo is 2,500,000 tonnes/year. The client of DMC wants a minimum storage capacity of 80,000 tonnes or 100,000 m³, which is a months supply.

• determination of the terminal dimensions
A total of six tanks are designed. A bund wall to prevent major soil pollution in case of spills or leakage surrounds each tank. The height and the diameter of each tank are 15 m and 40 m. The theoretical volume of the tanks is 18,850 m³. The total capacity of the tank farm is 6 * 18,850 m³ = 113,100 m³ and agrees with the wish of the client.
The length of the quay wall does not have to cover the entire ships length, but can be smaller. In the layout DMC presented the length of the quay wall is 180 m.

• number of vessels calling the port
A liquid bulk tanker is handled as quickly as possible, because of the damage the vessel can cause in case anything goes wrong. The average vessel size is 20,000 DWT and the handle time is at the most 10 hours. The number of vessels needed in a year is 157. The berth occupancy is about 25%.

• Waiting time
The berth occupancy is very low. The waiting time is expected to be acceptable.

4.3.8. Conclusions of the harbour review
• The wish of the client to design a container terminal with 4 berths would lead to high waiting time. Four berths are not sufficient to handle 8,000,000 tonnes of cargo. The number of berthing places must be increased to 5 places. The occupancy rate in that case is in order of 56%, which is acceptable;
• The container storage area must be increased based on the used starting points;
• The estimate for the throughput of the harbour for the year 2005 in my opinion is too high. The present cargo volume in the 8th region of Chile is extrapolated. The estimate for the year 2005 will be approximately 6.5 times lower based on the throughput at this time. To be on the safe side the estimate can be as much as 4 million tonnes. In this case it is not necessary to increase the number of containers berths;
• The stopping distance is too short when making fast of the tugs is done inside the harbour. The stopping distance is sufficient when the tugs make fast outside the harbour. The operational wave climate allows making fast outside the harbour.
5. Determination of the characteristics of a tsunami

In this chapter a description is given of a tsunami in general and of the tsunami occurring in 1960 in front of the coast of Chile. This tsunami is an important reference in this research. Tsunamis are described with the help of empirical relations. After that the strength of the tsunami used in the simulation is determined. Finally the simulation program MIKE21 is described.

5.1. Introduction

Tsunamis have been reported since ancient times. The first recorded tsunami occurred off the coast of Syria in 2000 B.C. Since 1900, the beginning of instrumentally located earthquakes, most tsunamis have been generated in Japan, Peru, Chile, New Guinea and the Solomon Islands. Hawaii has experienced tsunamis generated in all parts of the Pacific Oceans, because of its central position in the Pacific Basin. Only a few tsunamis have been reported in other parts of the world.

On 22 May 1960 an earthquake occurred in front of the coast of South Central Chile. A Pacific-wide tsunami was triggered by the earthquake. Along the Peru-Chilean coast the estimated loss of life ranged from 330 to 200 people. On the island Chiloe 200 deaths were reported. After 14.8 hours the tsunami arrived at Hilo on Hawaii and caused 61 deaths. After approximately 22 hours after the generation the tsunami arrived in Japan, where again it did its devastating work. The route of the tsunami is presented in figure 5.1.

Figure 5.1 propagation of the 1960 tsunami towards Japan
A tsunami can be characterised as a shallow water wave. It has a very long wavelength in comparison with the water depth. The velocity of the tsunami can be described by

\[ c = \sqrt{gd} \]  \hspace{1cm} (5.1)

with:
- \( c \) = the velocity of the wave
- \( g \) = acceleration of gravity
- \( d \) = water depth

Shallow water waves are different from wind generated waves, which usually have a period in the range of 5 to 20 seconds and have a wavelength of 100 to 200 meters. A tsunami can have a period ranging from 10 minutes to 2 hours and have a wavelength of more than 500 km. The wavelength is determined by

\[ \lambda = cT \]  \hspace{1cm} (5.2)

with:
- \( \lambda \) = the wavelength
- \( c \) = the velocity of the wave
- \( T \) = the wave period

Tsunamis are able to cross the entire Pacific Ocean in less than one day, without losing much of their strength. This is because the loss of energy is inversely related to the wavelength. When the tsunami enters the coastal region it undergoes a transformation. Because of the decreasing speed in more shallow water and the shape of the coast the tsunami will grow in height. The amplitude of the wave will grow according to the next expression

\[ \frac{A_1}{A_2} = \sqrt{\frac{c_2}{c_1}} \]  \hspace{1cm} (5.3)

with:
- \( A_1 \) = amplitude of wave in deeper water
- \( A_2 \) = amplitude of wave in more shallow water
- \( c_1 \) = velocity of the wave in deeper water
- \( c_2 \) = velocity of the wave in more shallow water

When arriving in the coastal area the tsunami may appear as a rapidly falling tide, a series of breaking waves, or even a bore. A bore is a violent rush of water with an abrupt front. Reefs, bays, entrances of rivers, undersea features and the slope of the beach all help to shape the tsunami.
5.2. The generation process of a tsunami in front of the Chilean coast

5.2.1. The generation of a tsunami

The basic principle of the generation of a tsunami is a sudden shock or movement of the earth or water, creating a wave. The movement of the bottom or the water surface can be caused by the following events:

- An earthquake on land;
- An earthquake in the ocean or seaseake;
- A landslide;
- A volcanic eruption;
- A nuclear explosion;
- The impact of meteorites.

After that the wave propagates along the water surface towards the coast where it develops into a tsunami.

Most tsunamis are created by a rapid vertical movement along a break in the earth’s crust on the bottom of the ocean. The tsunami is generated when a large mass of earth drops or rises and displaces the column of water directly above it. This type of displacement commonly occurs in large subsidence zones. The collision of two tectonic plates causes the oceanic plate to dip beneath the continental plate to form deep ocean trenches, like the Peru-Chilean trench.

In Golfo de Arauco generally two types of tsunamis occur. An earthquake triggers both types. The first type is generated far away from the Chilean coast after which it travels towards the coast of Chile and develops into a huge wave. This type can cross the entire ocean unseen and suddenly arise out of the water when approaching the shore. The second type is generated by an earthquake occurring near the coast of Chile with its epicentre on land. After this main shock several aftershocks occur in front of the coast, which cause the actual subsidence of the bottom of the sea. It is observed that this type generates long-period tsunamis.

The tsunami of 1960 included both types of tsunami generation. The tsunami was generated in front of the coast of Chile and travelled across the ocean towards Japan. In Japan the tsunami suddenly came ‘out of the sea’, the first type, and in Chile the tsunami occurred within an hour after the earthquake, the second type.

The tsunami used in this study is of the second type. The epicentre of the earthquake is situated on land.

5.2.2. Description of the generation of a tsunami in front of the coast

An earthquake consists of a main shock and a number of aftershocks. The main shock has its epicentre on land. The epicentres of the aftershocks all lie within an ellipse-shaped area along the coast, which is called the aftershock zone. In figure 5.1 the aftershock of the 1960 tsunami is shown. The numbers in parenthesis indicate the period of the tsunami in minutes. The other numbers indicate the wave height of the tsunami in meters. The inundation height of the tsunami is defined as the tsunami wave height.

---

1 Hatoh, T.; Directivity of Tsunamis; Bulletin of the Earthquake Research Institute, Vol. 41 (1963), pp. 61-81

Annemieke Schimmel 28 November 1999
Figure 5.2 The aftershock zone of the 1960 tsunami

Investigations show that the area of aftershocks corresponds approximately to that of crustal deformation. The original area of a tsunami corresponds to the area of crustal deformation. Thus, tsunami waves must be generated from an aftershock area. During an aftershock the oceanic plate pushes the land plate away while dipping underneath. In figure 5.3 the land plate is situated on the right side and the oceanic plate on the left side. The triangle marks the trench axis in front of the coast.

6 Comer, R.P.; Tsunami Earthquakes and Undersea Deformation; Tsunamis: Their Science and Engineering, edited by K. Iida and T. Iwasaki, 77-89.
Figure 5.3 The oceanic plate dips underneath the land plate, the triangle is the trench axis

Figure 5.4 shows that the land plate will transform and locally even dips down. The movement of the bottom is supposed to occur instantaneously and is translated immediately to a water level change.

Figure 5.4 Vertical surface displacement of the bottom of the sea perpendicular to the coast due to an earthquake

The figure also illustrates two kinds of faulting, which might occur on the landward side of the trench, which is the right side in the pictures. The upper picture in figure 5.4 shows a vertical displacement due to a shallow dipping fault. This is very likely the cause for the most ordinary tsunamiogenic earthquakes. The lower picture is an example of a steep dipping fault, which may occur during tsunami generating earthquakes.
The first type is the one that most likely caused the tsunami of 1960 and is used in this study as an assumption. It produces a landward subsidence and a trenchward uplift. The second type is not further taken into account.
Barrientos\textsuperscript{7} investigated details of the 1960 faulting revealed by surface movements. In figure 5.5\textsuperscript{7} the dotted line is the trench line in front of the coast. In front of Golfo de Arauco the trench is situated at a distance of approximately 130km. Barrientos tries to find a model that can determine the deformation of the bottom of the sea. To test the model used in the article they used observations of sea level changes, which are shown in figure 5.6\textsuperscript{7}. The sea level changes are supposed to correspond with the bottom level change. The circles in the figure are the observed sea level changes and are interesting for this study.

\textbf{Figure 5.5 Trenchline 130 km in front of the coast}

\textsuperscript{7} Barrientos, S.E. and S.N. Ward; The 1960 Chile earthquake: inversion for slip distribution from surface deformation; Geophys. J. Int. (1990) 103, 589-598.
Figure 5.6 Bottom level change perpendicular to the coast translated to water surface elevation

The sea level changes found by Barrientos are implemented into the simulation program instead of the complete aftershock area. The entire aftershock area is too large for the simulation program, which is explained in paragraph 5.5.1.

5.3. Empirical relations

5.3.1. Introduction

A lot of research is done on the tsunami subject. Researchers especially are interested in the generation process and what aspects influence the tsunami wave height and hope to find a solution to tackle the tsunami problem. My interest is focused on the development of the tsunami in the bay area and the tsunami wave height reached at the site.

5.3.2. Factors that influence the generation of the tsunami caused by an earthquake

Not all quakes do generate a tsunami. That depends on a number of factors shown in the enumeration below:

- The strength of the earthquake;
- The water depth;
- The velocity of the bottom movement;
- The depth of the centre of the quake beneath the bottom of the sea, the focal depth;
- The coincidental displacement of sediment in the area;
- The energy transfer from the bottom of the ocean to the surface of the water.

The strength of the earthquake has to have a minimum value to be able to generate a tsunami, which is explained in paragraph 5.3.4. The water depth in front of the coast of
Chile is a given value of 4000m. The bottom is supposed to move instantaneously, which means that the shape of the bottom can be translated to the water surface. The depth of the centre of the quake, the displacement of sediment and the energy transfer from the bottom of the sea to the water surface are not taken into account in this report.

5.3.3. Factors that influence the tsunami wave height

- The inclination of the foreshore slope;
- The shape of the aftershock area;
- The resistance of the shore in case of flooding;
- The generation process of the tsunami;
- The shape of the coastline, for example the existence of a bay.

If the foreshore is less steep the amplitude of the tsunami will be higher. This is because less of the wave is reflected towards the open sea and more of the wave is propagating towards the shore. The influence of the shape of the aftershock is not known so far. In this study this aspect is not researched. It is a subject for further investigation. The resistance onshore has been taken equal to resistance in the water. The interest focuses on the tsunami wave height at the site. The generation process is determined already and not variable in this study. The influence of the shape of the bay is researched in the second part of the experiments, which will be explained in chapter 6.

5.3.4. The earthquake magnitude $M$

The Richter scale is based on the amount of energy that is produced during the earthquake. When the earthquake is stronger the amount of energy will be higher, thus the earthquake magnitude will have a larger value. This is expressed in the next relation.

$$M = \left(\frac{\log E - 11.8}{1.5}\right)$$

(5.4)

with:

- $M$ = earthquake magnitude on the Richter scale;
- $E$ = energy in ergs $= g \text{ cm}^2/\text{s}^2 = 10^3 J$.

For tsunamis with a maximum wave height on shore of 50 - 75 cm the following empirical expression is found between the magnitude of the quake and the focal depth $h$. The focal depth is the depth of the centre of the quake beneath the surface of the earth and can vary between 10 - 700 km.

$$M = 6.3 + 0.005h$$

(5.5)

with:

--

Camfield F.E.; Tsunami Engineering; U.S. Army Corps of Engineers, special report no. 6, 1980.

M = earthquake magnitude on the Richter scale;
h = focal depth in km.

This formula shows that the minimum value of an earthquake is 6.3 on the Richter scale. Beneath this value the earthquake will not be able to generate a tsunami.

For tsunamis with a minimum wave height on shore of 4 - 6 m the following empirical expression is derived.

\[ M = 7.7 + 0.009b \]  \hspace{1cm} (5.6)

This relation shows that the minimum value of the earthquake is 7.7 on the Richter scale. The expressions 5.5 and 5.6 are based on tsunamis that occurred in Japan. It is assumed that they are useful for tsunamis occurring in Chile because of the similar geometry of the foreshore. The bathymetry of the Japanese coastal area is very similar to that of Chile. In more detail there are differences of course, but the assumption is made that the empirical relations can be used in the Chilean area.

5.3.5. Relation between the earthquake magnitude M and the shape of the aftershock zone

The aftershock area seems to be roughly elliptic in shape as is observed in figure 5.2, although the precision is limited by the accuracy of observation. A relation is obtained between the axis of the aftershock area and the earthquake magnitude M.

\[ \frac{b}{a} = AM + B \]  \hspace{1cm} (5.7)

with:
\[ a \] = the major axis of the ellipse;
\[ b \] = the minor axis of the ellipse.

Numerical values of the constants A and B obtained graphically are

\[ A = -0.13 - 0.28 \]
\[ B = 1.69 - 2.56 \]

The aftershock area appears to become strongly elliptical when the earthquake magnitude increases. For \( b/a = 1 \), the earthquake magnitude becomes 6 from the above equation. It is noticed that the earthquake magnitude 6.3 is the limiting magnitude below which tsunamis do not occur as far as the Japanese data is concerned. In this case the aftershock area will be roughly circular.

In my opinion the shoreline and the location of the trench influence the shape of the ellipse as well. For an earthquake with strength 8.4 on the Richter scale, as was the case in 1960, the ratio between the major and minor axis of the ellipse is in order of 1.5. In figure 5.2 the aftershock zone is printed and the ratio is much bigger than relation 5.7 points out. This indicates that the formula derived graphically is very inaccurate.
5.3.6. **Relation between the axis of the aftershock zone and the tsunami wave height.**

In the illustration of the aftershock zone of the 1960 tsunami in figure 5.2 the distribution of the tsunami wave heights and periods are shown also. The periods are expressed in between parenthesis. In the figure the epicentre is the place of the main shock and is shown as ☼. The epicentre is located at a corner of the after-shock area. This pattern is seen in most cases. Although the wave height of a tsunami is expected to be affected by various factors, such as refraction, diffraction and reflection by the bottom irregularities, and also by the shape of the bay, the wave heights in the direction of the minor axis, as a rule, appear to be higher than those in the direction of the major axis of an after shock area.

The tsunami wave height ratio seems to increase with the increase of ellipticity of the origin area. The formula is as follows:

\[ \frac{R_b}{R_a} = \frac{a}{b} \]  

(5.8)

with:

- \( R_a \) = the wave height in the direction of the major axis of the ellipse
- \( R_b \) = the wave height in the direction of the minor axis of the ellipse
- \( a \) = length of the major axis
- \( b \) = length of the minor axis

From experiments a similar result is obtained. For a circular area of the aftershock, it seems that no directivity of wave heights appears. On the other hand, tsunamis accompanying great earthquakes have shown a conspicuous directivity and the ratio may reach one-fourth. The tsunami of 1960 does indicate that the tsunami wave heights in direction of the minor axis are bigger than the wave heights in the direction of the major axis.

5.3.7. **Relation between the earthquake magnitude \( M \) and the tsunami period \( T \)**

The relationship between the period of the primary tsunami, which carries the maximum energy, and the earthquake magnitude is as follows\(^{10}\).

\[ 10 \log T = 0.625M - 3.31 \]  

(5.9)

The relation between \( M \) and \( T \) is an indirect connection. The period \( T \) is dependent on the size of the subduction zone, which is caused by the earthquake with a certain magnitude \( M \). This makes the relation less reliable. The equation also does not reckon with the directivity of the period of a tsunami. The elliptical shape of the aftershock zone has an influence on the period too. The period \( T_a \) in the direction of the major axis of an aftershock area is longer than \( T_b \) in the direction of the minor axis. The difference in periods in the major and minor axes becomes larger when an aftershock area has a more elliptical shape. In figure 5.1 this is illustrated as well. Figure 5.7 illustrates that the period of the tsunami is increasing when the earthquake magnitude \( M \) increases.

5.3.8. The tsunami magnitude \( m \)

The tsunami magnitude \( m \) is defined by Iida\textsuperscript{11} and others as

\[
\log R_{\text{runup}} = m
\]  

(5.10)

with:
\( m \) = tsunami magnitude;
\( R_{\text{runup}} \) = maximum runup height.

The magnitude scale has a minimum value of \( m = -2 \) and a maximum value of \( m = 5 \). These values and the values in between are printed in table 5.1 and figure 5.8.

<table>
<thead>
<tr>
<th>Magnitude ( m )</th>
<th>Max runup height ( R ) in m</th>
</tr>
</thead>
<tbody>
<tr>
<td>-2</td>
<td>&lt; 0.25</td>
</tr>
<tr>
<td>-1</td>
<td>0.5</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>&gt; 16</td>
</tr>
</tbody>
</table>

Table 5-1 Tsunami magnitude scale

\textsuperscript{11} Iida K. Et al.; Preliminary catalog of tsunamis occurring in the Pacific Ocean.
When the tsunami magnitude $m$ is increasing the runup height increases also. The runup of the tsunami is dependent on the location. When the shore is steep the runup height will be larger than when the shore is shallower.

![Figure 5.8 Magnitude $m$ versus wave height $R_{\text{runup}}$](image)

5.3.9. Relation between earthquake magnitude $M$ and the tsunami magnitude $m$

The relationship\textsuperscript{12} between the tsunami magnitude $m$ and the earthquake magnitude $M$ is a very difficult one. The empirical relation that is derived has very wide ranges. This is because a vertical component is supposed to be present during the faulting process. This vertical component and the focal depth are the reason for occasional mismatches between $m$ and $M$.

$$m = (2.6 \pm 0.2)M - (18.4 \pm 0.5)$$

(5.11)

![Figure 5.9 Earthquake magnitude $M$ versus tsunami magnitude $m$](image)

5.4. *The occurrence of a tsunami in Golfo de Arauco*

The last known major tsunami that hit the shore of Chile was the tsunami of 1960. It was a devastating tsunami. Nearly 2000 people died because of the tsunami. A record exists of tsunamis occurring in Golfo de Arauco from 1570. It is printed in table 5.2. These tsunamis are found in a database found on Internet too. The epicentres vary from 20 till 45 S and 69 till 76 W.

<table>
<thead>
<tr>
<th>Year</th>
<th>Epicentre</th>
<th>Origin</th>
<th>Magnitude</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 Feb. 1570</td>
<td>Concepcion</td>
<td>Sea</td>
<td>8 - 8 1/2</td>
<td>Destructive Tsunami</td>
</tr>
<tr>
<td>15 Mar. 1657</td>
<td>Concepcion</td>
<td>Sea</td>
<td>8</td>
<td>grand Tsunami</td>
</tr>
<tr>
<td>25 May 1751</td>
<td>Concepcion</td>
<td>Sea</td>
<td>8 1/2</td>
<td>grand Tsunami</td>
</tr>
<tr>
<td>20 Feb. 1835</td>
<td>Concepcion</td>
<td>Sea</td>
<td>8 - 8 1/4</td>
<td>grand Tsunami</td>
</tr>
<tr>
<td>24 Jan. 1939</td>
<td>Chillan</td>
<td>Coast</td>
<td>8.3</td>
<td>Tsunami</td>
</tr>
<tr>
<td>6 Mar. 1953</td>
<td>Chillan</td>
<td>Coast</td>
<td>7.5</td>
<td>Tsunami</td>
</tr>
<tr>
<td>27 May. 1960</td>
<td>Concepcion</td>
<td>Sea</td>
<td>7.5</td>
<td>grand Tsunami</td>
</tr>
</tbody>
</table>

Table 5-2 Occurrence of tsunamis in Golfo the Arauco

With the help of this record an empirical formula is found for the chance of occurring of a tsunami with a certain magnitude.

\[
10^{\log N} = 0.3 \text{ m} + 0.8 \tag{5.12}
\]

with:

- \( N \) = return period in years;
- \( m \) = tsunami magnitude.

The chance a tsunami will occur can be expressed per year. In the next table these chances are printed.

<table>
<thead>
<tr>
<th>M</th>
<th>N</th>
<th>Chance of occurrence per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>-2</td>
<td>1.6</td>
<td>0.63</td>
</tr>
<tr>
<td>-1</td>
<td>3.2</td>
<td>0.31</td>
</tr>
<tr>
<td>0</td>
<td>6.3</td>
<td>0.16</td>
</tr>
<tr>
<td>1</td>
<td>12.6</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>25</td>
<td>0.04</td>
</tr>
<tr>
<td>3</td>
<td>50</td>
<td>0.02</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>0.01</td>
</tr>
<tr>
<td>5</td>
<td>200</td>
<td>5 \times 10^{-3}</td>
</tr>
</tbody>
</table>

Table 5-3 Occurrences of tsunamis per year

Table 5.2 shows that the tsunamis where all generated by earthquakes with a magnitude of approximately \( M = 8 \). This corresponds with a tsunami magnitude of \( m = 3 \) or 4. A tsunami with magnitude \( m = 3 \) will occur every 50 year and a tsunami with a magnitude of \( m = 4 \) every
100 year. The design life of the harbour is 100 year. This means that the harbour will be hit two or three times by a tsunami.

5.5. **The simulation program Mike21**

5.5.1. **Introduction**

DMC proposed to use the simulation program MIKE21\(^\text{13}\). It is a comprehensive modelling system for two-dimensional free surface flows where stratification is neglected. The program can handle all sorts of coastal problems as distribution of pollution, sediment transportation along the coast, tidal movements. The version bought by DMC is a entry level version of the HD-module. This module can be used for modelling of tidal hydraulics, wind and wave generated currents, storm surges and flood waves. The equations used by this program are the long wave equations and therefore the program can be used in this study for the tsunami problem.

The entry level version is limited to 5000 water points, which is the reason for not implementing the entire aftershock area. Instead a cross section perpendicular to the coast is used as input, which is explained in more detail in the next chapter.

5.5.2. **Equations used by MIKE21**

The program uses the long wave equations that are transformed into two-dimensional equations as\(^\text{14}\):

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \quad M = \int_{\zeta}^{h} y \, dz \quad N = \int_{\zeta}^{h} \nu \, dz
\]  \hspace{1cm} (5.13)

\[
\frac{\partial M}{\partial t} = fN - \frac{1}{(d + \zeta)} \left( \frac{\partial}{\partial x} (M^2) + \frac{\partial}{\partial y} (MN) \right) - (d + \zeta) \frac{\nu}{(d + \zeta)^2} \frac{\partial \zeta}{\partial x} + \frac{\nu^2}{(d + \zeta)^4} M \sqrt{M^2 + N^2}
\]  \hspace{1cm} (5.14)

\[
\frac{\partial N}{\partial t} = -fM - \frac{1}{(d + \zeta)} \left( \frac{\partial}{\partial x} (MN) + \frac{\partial}{\partial y} (N^2) \right) - (d + \zeta) \frac{\nu}{(d + \zeta)^2} \frac{\partial \zeta}{\partial y} + \frac{\nu^2}{(d + \zeta)^4} N \sqrt{M^2 + N^2}
\]  \hspace{1cm} (5.15)

with:

\[
\begin{align*}
\text{\( u \)} &= \text{velocity component in x-direction;} \\
\text{\( v \)} &= \text{velocity component in y-direction;} \\
\text{\( d \)} &= \text{the water depth;} \\
\text{\( \zeta \)} &= \text{elevation of the water surface;} \\
\text{\( \nu^2 \)} &= \text{friction coefficient at the sea bottom;} \\
\text{\( f \)} &= \text{the Coriolis parameter;} \\
\text{\( g \)} &= \text{the gravitational acceleration.}
\end{align*}
\]


The coefficient of friction $\gamma'$ at the sea bottom is related to the Chézy number according to the next relation: $\gamma' = \frac{g}{C^2}$, with $C$ is the Chézy coefficient.

Equation 5.13 is the continuity function and equations 5.14 and 5.15 respectively are the $x$- and $y$-momentum functions. The velocity component is an average of over the entire depth. The ratio of the water depth and the wavelength is of order 0.

5.5.3. Restrictions for the two-dimensional equation

The restrictions are:
- the gravitational acceleration is constant;
- the sea water is not compressible;
- no variation in salinity;
- no variation in temperature;
- surface tension is neglected;
- no interaction between water and air;
- the bottom of the sea cannot move as from $t = 0^+$, the movement of the bottom occurs at $t=0$ after which the bottom will not move again;
- the bottom is impenetrable;
- no variation in air pressure;
- The slope of the trench parallel to the coast. When the slope becomes steeper both reflection and shoaling play an important role. When the slope is too steep the reflection will become the most important factor causing the biggest part of the wave to be reflected towards the sea. On the other hand when the slope is not steep enough, shoaling cannot develop completely and the wave will be less high then is the case with a steeper slope. This effect is not taken into account in the simulation but can be used when interpreting the results of the simulations;
- For the bed resistance MIKE21 uses the following formula:

$$\frac{gu^3}{C^2}$$  \hspace{1cm} (5.16)

with:
- $g$ = gravity acceleration
- $u$ = velocity of the wave
- $C$ = number of Chézy

The Chézy number can be expressed in the number of Manning according to the following formula:

$$C = Mh^K$$  \hspace{1cm} (5.17)

---

with:
\[ M = \text{the Manning number} \]
\[ h = \text{the water depth} \]

The standard value for the Manning number in MIKE21 is 32 \( m^{1/3}/s \), this means for the Chézy number a value of 50 \( m^{1/3}/s \) in 15 m deep water in front of the shore. This value is considered to be normal in this area;
- The tide is not a parameter in the computer simulation. When the tide is high the tsunami wave height will be higher too.

5.5.4. The numerical scheme
MIKE21 uses a numerical method of approach for the long wave equations. The approach is done with the help of a staggered numerical scheme as shown in figure 5.10. This scheme is unconditionally stable. A fractionated-step technique combined with an Alternating Direction Implicit (ADI) algorithm is used in the solution to avoid necessity for iteration. Second order accuracy is ensured through the centring in time and space of all derivatives and coefficients.

![Numerical scheme of MIKE21](image)

Figure 5.10 numerical scheme of MIKE21

The ADI method exists in basically two steps. The equations are solved in one-dimensional sweeps, alternating between \( x \) and \( y \) directions. In the \( y \)-sweep the continuity and \( y \)-momentum equations are solved and in the \( x \)-sweep the continuity and \( x \)-momentum equations are solved. Adding of the two sweeps together gives a perfect time centring. This is made clear with the help of the next example:

\[
\frac{\partial u}{\partial t} + g \frac{\partial \zeta}{\partial x} = 0
\] (5.18)
\[ \frac{\partial v}{\partial t} + g \frac{\partial \zeta}{\partial y} = 0 \] (5.19)

These equations are transformed with the help of figure 5.10 into

\[ \frac{u_{j+\frac{1}{2},k}^{s+1} - u_{j+\frac{1}{2},k}^s}{\Delta t} + g \frac{\zeta_{j+1,k}^{s+1} - \zeta_{j,k}^{s+1}}{2\Delta x} + g \frac{\zeta_{j+1,k}^s - \zeta_{j,k}^s}{2\Delta x} = 0 \] (5.20)

\[ \frac{v_{j,k+\frac{1}{2}}^{s+1} - v_{j,k+\frac{1}{2}}^s}{\Delta t} + g \frac{\zeta_{j,k+1}^{s+1} - \zeta_{j,k}^{s+1}}{2\Delta y} + g \frac{\zeta_{j,k+1}^s - \zeta_{j,k}^s}{2\Delta y} = 0 \] (5.21)

with:
- \( s \) = time parameter;
- \( j \) = parameter for the points in x-direction;
- \( k \) = parameter for the points in y-direction;
- \( \Delta t \) = time-step;
- \( \Delta x \) = distance in x direction;
- \( \Delta y \) = distance in y direction.

By using the ADI method the solution is derived in two steps in a more easy way as is shown in the next example\(^\text{16}\).

First step:

\[ \frac{u_{j+\frac{1}{2},k}^{s+\frac{1}{2}} - u_{j+\frac{1}{2},k}^s}{\frac{1}{2}\Delta t} + g \frac{\zeta_{j+1,k}^s - \zeta_{j,k}^s}{2\Delta x} = 0 \] (5.22)

\[ \frac{v_{j,k+\frac{1}{2}}^{s+\frac{1}{2}} - v_{j,k+\frac{1}{2}}^s}{\frac{1}{2}\Delta t} + g \frac{\zeta_{j,k+1}^s - \zeta_{j,k}^s}{2\Delta y} = 0 \] (5.23)

Second step:

\[ \frac{u_{j+\frac{1}{2},k}^{s+1} - u_{j+\frac{1}{2},k}^{s+\frac{1}{2}}}{\frac{1}{2}\Delta t} + g \frac{\zeta_{j+1,k}^{s+\frac{1}{2}} - \zeta_{j,k}^{s+\frac{1}{2}}}{2\Delta x} = 0 \] (5.24)

\[ \frac{v_{j,k+\frac{1}{2}}^{s+1} - v_{j,k+\frac{1}{2}}^{s+\frac{1}{2}}}{\frac{1}{2}\Delta t} + g \frac{\zeta_{j,k+1}^{s+\frac{1}{2}} - \zeta_{j,k}^{s+\frac{1}{2}}}{2\Delta y} = 0 \] (5.25)

\(^\text{16}\) Bijl W.; Tsunami golven in het noordzeegebied, afstudeerrapport, TU Delft, 1993
6. Experiments with MIKE21

In this chapter a description of the research process with its results is given. The process is divided in two parts. The first part handles a one-dimensional model that is used to carry out a sensitivity study. The second part handles the finer grid model, which is two-dimensional and covers the entire bay area. It is used to determine the actual tsunami height at Lomas Coloradas.

6.1. The procedure of the research process

The simulation of the tsunami is divided into a one-dimensional model and a two-dimensional model. The one-dimensional model is used to carry out a sensitivity study and for determination of the influence of the foreshore of Golfo the Arauco. The two-dimensional model is used to determine the influence of the bay and of the island Santa Maria. Finally the actual tsunami height at the site is determined.

The one-dimensional grid is situated perpendicular to the coast in front of the bay of interest. The aftershock area is stretched along the coast. The length of the aftershock area is much longer than the width, which makes it possible to use a one-dimensional model. The total length of the grid is roughly 2500 km and covers a part of the ocean and a part of Chile. The grid is adopted from a grid found on Internet\textsuperscript{17}, which has a grid size of 5*5 minutes. A cross section of the grid is given in figure 6.1 and figure 6.2. One minute corresponds to one nautical mile, 1853m. At first the proper time step and grid size are determined.

![Figure 6.1 cross section perpendicular to the coast](image)

\textsuperscript{17} http://ingrid.ldgo.columbia.edu/SOURCES/.WORLDDBATH/.bath/

The second step is to determine the effect of the following factors on the development on the height of the tsunami:

- the magnitude of the vertical water level change;

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\* Annemieke Schimmel 43 November 1999
• the size of the horizontal water level change;
• the position of the water level change in front of the coast;
• the inclination of the water level change.

Finally the Barrientos profile is introduced in the one-dimensional model to simulate as close as possible the 1960 tsunami.

![Figure 6.2 cross section perpendicular to the coast in 3-D view](image)

In this study the two-dimensional model will cover the bay grid area including the trench perpendicular to the coast near the river Bio Bio. This grid is used to compare the known height of the tsunami in 1960 with the simulation results of this event. The purpose is to determine the development of the tsunami in the bay and the determination of the route of future tsunamis. The shape of the bay and the trench are the most important influences for the propagation of tsunamis.

6.2. **The one-dimensional model**

6.2.1. **The geometry of the model**

The one-dimensional model is situated near Golfo de Arauco perpendicular to the coast. It has a total length of roughly 2500km. It is built up with a grid size of 9265m, which is adopted from the grid found on Internet. A coloured copy of the one-dimensional grid is found in The One-dimensional test report.

The one-dimensional grid is divided into four parts. This is illustrated in figure 6.3.
• Part 1 is the ocean part with a depth of 4000 m. It is a very long part, the reason for this is in paragraph 6.3.1;
• Part 2 is the trench which is very steep and the expectation is that the tsunami will grow on this part;
Part 3 represents Golfo de Arauco with a piece of land included. It is a lot less steep than the trench. In this part the tsunami will get its final height;

Part 4 portrays a section of the mountain range the Andes. This part is not necessary for the modelling of the tsunami because it will never reach this part of the land, but it completes the entire image.

The one-dimensional model has the appearance of a two-dimensional model. It will behave as a one-dimensional model however, because of the great length according to the width. The model has the same depth over the entire width. With the help of a vector plot of the velocity while running a test the one-dimensional behaviour of the model is presented, which is explained in paragraph 6.3.1.

![Figure 6.3 the one-dimensional model](image)

6.2.2. **The boundaries of the model**

The one-dimensional model has five boundaries. One boundary at the open sea, two boundaries alongside the model, one boundary at the land side and the water surface is the fifth boundary.

- The boundary on the ocean side is an open boundary, which means that mass can freely cross it and that the water level is fixed;
- The two side boundaries are closed. This indicates that no mass of water will enter or leave these boundaries;
- The land side is the fourth boundary of the model. MIKE21 has the possibility of flooding and drying on the land side. This means that the boundary must move with the water onto the land;
- The water surface gets an initial water level change before the calculations start. This water level change is stretched out in the length direction of the model over 10 grid points or 93km.

A vertical water movement is the generator of the tsunami. The initial water level change must be given before the calculations start. In chapter 5 is described that the water level is changing in the so-called subduction zone. This zone has a width of approximately 200km
from the trench to the shore. The vertical water level changes in the sensitivity study will have a varying width of 93km till 278km.

6.2.3. The grid space and the time step
The first estimate of the grid spacing is adopted from the data from Internet as mentioned before. The grid spacing over the width of the model is not important because of the one-dimensional character of the model, but the grid spacing in the other direction certainly is important. A rule of thumb is that the wavelength of a tsunami contains ten grid points. As a first estimate the wavelength has a value of 100km. Approximately the size of the water level change area. The grid points have to be 10km apart from each other. The data from Internet have a grid size of five nautical miles, 9265m, and this is used as a first estimate. The time step is related to the grid space according to the following relation

$$C_s = \frac{\Delta t}{\Delta x}$$

with:

- $C_s$ = the courant number
- $\Delta t$ = the time step in seconds
- $\Delta x$ = the grid spacing in m
- $c$ = the velocity of the wave

MIKE21 does not have a limit for the Courant number. To be on the safe side the courant number is chosen to be close to one. The celerity of a tsunami in water with a depth of 4000m is 200 m/s. This results in:

$$1.4 = 200 \times \frac{\Delta t}{9265} \rightarrow \Delta t \approx 60 \text{ s.}$$

6.2.4. Data points in the model
The data points are printed in the picture below. In each data point an output of the water elevation is generated. All points together create a complete picture of the propagation of the tsunami.

Figure 6.4 data points of the one-dimensional model
The distance of each point to the shoreline and the depth are presented in table 6.1. The numbers between parenthesis indicate the distance from that point to the shoreline. The depths of the data points are printed in the second and the fourth column.

<table>
<thead>
<tr>
<th>Point number</th>
<th>Depth</th>
<th>Point number</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point 1 (2450 km)</td>
<td>4000 m</td>
<td>Point 9 (101 km)</td>
<td>409 m</td>
</tr>
<tr>
<td>Point 2 (1600 km)</td>
<td>4000 m</td>
<td>Point 10 (93 km)</td>
<td>375 m</td>
</tr>
<tr>
<td>Point 3 (760 km)</td>
<td>4000 m</td>
<td>Point 11 (84 km)</td>
<td>341 m</td>
</tr>
<tr>
<td>Point 4 (177 km)</td>
<td>4000 m</td>
<td>Point 12 (59 km)</td>
<td>240 m</td>
</tr>
<tr>
<td>Point 5 (169 km)</td>
<td>3601 m</td>
<td>Point 13 (34 km)</td>
<td>138 m</td>
</tr>
<tr>
<td>Point 6 (152 km)</td>
<td>2803 m</td>
<td>Point 14 (17 km)</td>
<td>71 m</td>
</tr>
<tr>
<td>Point 7 (135 km)</td>
<td>2005 m</td>
<td>Point 15 (8 km)</td>
<td>37 m</td>
</tr>
<tr>
<td>Point 8 (118 km)</td>
<td>1207 m</td>
<td>Point 16 (0 km)</td>
<td>3 m</td>
</tr>
</tbody>
</table>

Table 6.1 data points with water depth and distance to the shoreline

6.3. **Experiments with the one-dimensional model**

At first the structure of the standard experiment is explained. The standard experiment gives a general view of the propagation of the tsunami in the one-dimensional model. After that the sensitivity study is carried out and a start point for the two-dimensional model is determined.

6.3.1. **General description of the results of MIKE21**

The standard experiment exists in a water level change over 93 km approximately 300km in front of the coast, in between the measure points 3 and 4. In figure 6.5(a) the starting point of the numerical calculation with the water level change is shown. The depth scale is exaggerated in accordance to the length scale. The water level change has a magnitude of 1m downward and is situated in deep water. This is done to minimise the influence of the trench on the origination of the tsunami.

In figure 6.5(a-f) the birthing process of a tsunami is pictured. The area with a water level of 1m below sea level is filled with water from both sides. The width of the water level change area is 93km. It is illustrated that two water flows from both sides meet each other in the middle of the water level change area. After the collision two waves preceded by two valleys each run away from the collision point. One wave is propagating towards the open sea and the other wave is propagating towards the coast. The last one is the tsunami, which is of interesting for this study.
Figure 6.5(a) The tsunami birthing process on t=0s

Figure 6.5(b) the tsunami birthing process on t=120s

Figure 6.5(c) the tsunami birthing process on t=240s
Figure 6.5(d) the tsunami birthing process on t=360s

Figure 6.5(e) the tsunami birthing process on t=480s

Figure 6.5(f) the tsunami birthing process on t=600s
The water level change is determined in every data point. The complete run of all data points is found in The One-dimensional test report. In figure 6.6 the first two data points are printed. Point 1 has the value 0 during the complete simulation time, because this point is situated at an open boundary. The water can flow in and out and the water level remains constant. In point 2 the first changes can be seen after approximately 90 minutes after the origination of the tsunami. At first a trough is passing by. The trough is caused by the downward water level change area and is followed by a little peak. The peak is the result of the collision in the middle of the water level change area and the actual tsunami. After four hours a second peak occurs. This peak is caused by the reflection on the open boundary and is a numerical phenomenon. In reality this will not happen. To minimise the disturbance through this reflection part 1 of the model is made as long as possible. Everything happening after the second peak is not of any interest for this study.

Figure 6.6 data points 1 and 2 of figure 6.4 of the standard test

In figure 6.7 the data points 3 and 4 are printed. Point 3 is situated more closely towards the trench on the west side, or sea side, of the water level change area. The trough has reached point 3 after 30 minutes already. Again the trough is followed by a little peak propagating towards the open ocean. In this plot three peaks are detected instead of two, as is the case in the previous data point. The second peak of point 3 is the reflection on the shore of the tsunami propagating towards the shoreline. The third peak is the reflection on the open boundary of the model. Point 4 is situated on the east side, or land side, of the water level change area. After 6 minutes a trough passes by. This is not the same trough as in the
previous points. This trough is propagating towards the shore where it will appear as a withdrawal. The peak that follows the trough still is very small but should be growing when entering the bay area. A general description of the propagation of a tsunami is given here. The water level changes of the remaining points can be found in the appendix of this report. A plot of the flux in The One-dimensional test report shows that the model does behave as a one-dimensional model.

![Graph showing data points 3 and 4 of figure 6.4 of the standard test](image)

**Figure 6.7** data points 3 and 4 of figure 6.4 of the standard test

6.3.2. **Grid space variation**

Two tests are run to determine the ideal space step. The standard test is a reference test to compare the other tests with. The standard test has \( \Delta x = 9265\text{m} \), which is adopted from the Internet database. The time step is \( \Delta t = 60\text{s} \). The two other tests have the following parameters:

a) standard test with \( \Delta t = 60\text{s} \) and \( \Delta x = 4633\text{m} \)

b) standard test with \( \Delta t = 60\text{s} \) and \( \Delta x = 18530\text{m} \)

The test results are found in The One-dimensional test report. These tests are compared to the standard test. A first quick look shows that the difference between the tests is not significant. When looking in more detail a few things are noticed. Test b is less precise than the other two and test a is the most precise. The differences between the three tests are very small. The grid space of the standard test is sufficient for the
goals of this study. The propagation of the tsunami is to be determined and the wave height of the tsunami is finally determined in the two-dimensional model simulation.

6.3.3. Time step variation

To determine the proper time step, the same procedure as with the grid space variation is followed. The standard test is used as a reference. The grid space has the same value as the standard test and the time step is varied. The two comparing tests have the following parameters:

c) standard test with $\Delta x = 9265\text{m}$ and $\Delta t = 30\text{s}$
d) standard test with $\Delta x = 9265\text{m}$ and $\Delta t = 300\text{s}$

The standard test and test c do not differ very much. Test c has a slightly bigger peak of the wave in the bay area than the standard test. Test d is very inaccurate. This time step is too big and can not be used. The standard test is well to be used for the variation tests as is described in the following paragraph. It is a good instrument for this purpose.

6.3.4. Variation of the magnitude of the instant water movement

A comparison is made between three tests. The tests all have a different vertical water level change. Test 1 has a downward water level change of 1m, test 2 has a downward water level change of 2m and test 3 has an upward water level change of 1m. The changes are expressed in the following figures in grey. The distance of the water level change area from the shoreline is the same for the three tests and is approximately 110km. The horizontal water level change also is the same for all three tests and measures 93km.

Test 1:
1m downward instant water level change

![Figure 6.8 Water level change test 1](image)

Test 2:
2m downward instant water level change

![Figure 6.9 Water level change test 2](image)
Test 3:
1m upward instant water level change

The plots of the tests are found in The One-dimensional test report. The results of the test are described below:

- The three different plots of the tests look very similar. The results of test 3 are the same as test 1 except that the results of test 3 have a mirror image with respect to the horizontal axis;
- With a downward instant water level change a withdrawal can occur before the tsunami will hit the shore. When the water level change is an upward change the tsunami is most likely not preceded by a withdrawal;
- Test 1 and test 2 show a wave trough in the first period of the simulation. The trough of test 2 is twice as deep as the trough of test 1. Test 3 shows a similar image as test 1, except for the trough being a wave peak. The value of the peak from test 3 is the same as the value of the trough of test 1;
- The peak that follows the trough shows the same behaviour as is described for the trough in the previous point;
- The periods for each measure point are the same for each different test. The period of the tsunami does not depend on the magnitude of the instant water level change;
- A variation in the period is noticed for each individual test, which is shown in appendix 6A in figure 6A.1 and figure 6A.3. The data points 4 till 8 are not used because they are in the middle of the water level change area. When propagating away from the origination area the period tends to become bigger. This means that the wavelength of the tsunami becomes bigger when propagating towards the open boundary of the system, which in reality is the ocean. When propagating towards the shore the wavelength does not become bigger. This is because the wavelength depends on the water depth too. The water depth becomes smaller when approaching the shore and this effect is stronger than the decrease of the period. Figure 6A.4 in appendix 6A shows that the water depth becomes the stronger component from point 12.

6.3.5. Variation of the size of the horizontal water level change

Three different runs with an increasing change area are compared:

Test 1:
1m downward water level change over 93km.

Figure 6.10 Water level change test 3

Figure 6.11 Water level change of test 1
Test 4: 
1m downward water level change over 186km.

![Figure 6.12 Water level change of test 4](image)

Test 5: 
1m downward water level change over 278km.

![Figure 6.13 Water level change of test 5](image)

The plots of the tests are presented in The One-dimensional test report. The figures of the periods and the wavelength are presented in appendix 6B. The results of the tests are described below:

- When the origination area is bigger the period of the tsunami is bigger too;
- When the origination area is bigger the wavelength of the tsunami is bigger;
- The increase of the wavelength is mainly caused by the increasing withdrawal. The peak following the withdrawal hardly differs in wavelength;
- The peak following the withdrawal is slightly higher when the origination area is bigger.

6.3.6. Variation of the place of the water level change area

Four different runs with a moving change area are compared:

Standard test:
1m downward water level change 303km in front of the coast.

![Figure 6.14 Water level change of standard test](image)
Test 7:
1m downward water level change 211km in front of the coast.

Figure 6.15 Water level change of test 7

Test 1:
1m downward water level change 118km in front of the coast.

Figure 6.16 Water level change of test 1

Test 8:
1m downward water level change 25km in front of the coast.

Figure 6.17 Water level change of test 8

The plots of the tests are presented in The One-dimensional test report and the graphics are presented in appendix 6C. The results are described below:

- The development of the period of all tests is similar to that of the standard test. Test 8 can not develop the complete pattern because the origination area is too close to shore;
- The development of the wavelength of all tests is similar to the standard test too. It seems that test 8 has a different behaviour, but when looking more closely it is clear that it similar too. The wavelength of test 8 suddenly drops down near data point 4. This is the beginning of the trench in deep water. It is the same process as is the case with the other tests. The periods decrease from the open sea towards the origination area, which causes a decrease in the wavelength. This happens with test 8 too. The difference is that the decrease of the wavelength is much faster than for the other tests. This is because the wavelength is determined by the period of the wave as well as the water depth. The depth of the water is decreasing fast and is the important factor for the sudden drop in wavelength of test 8;
- When the origination area is closer to the shore the periods and the wavelengths are larger. This means that when a tsunami is generated close to the shore it generates a tsunami with a larger wavelength travelling overseas;
- Very little is to be said about the height of the tsunami. The value of the peak after the withdrawal hardly differs for these tests. Test 7 tends to have the biggest value.
6.3.7. The influence of the inclination of the origination area

Two different runs with an inclination are carried out. They are compared to run 1 with a downward movement of 1m. The origination area of all these tests lies 110km in front of the coast.

Test 1:
1m downward water level change 118km in front of the coast.

Test 9:
a 'negative' inclination from 1m till -1m over 93km.

Test 10:
a 'positive' inclination from -1m till 1m over 93km.

The plots are presented in The One-dimensional test report and the graphics are presented in appendix 6D. The results of the tests are described below:

- Test 9 and test 10 have the same development but they are reflected from each other on the ocean side;
- The withdrawal of test 10 in comparison with test 1 is approximately four times smaller, but the peak that follows the withdrawal of test 10 is nearly three times bigger at data point 2;
- Test 1 has the highest peak of all tests at data point 9;
- The highest value of the peak appears at data point 13, test 9. The peak almost has a value of 0.8 m above still water level;
- The biggest withdrawal is seen at the same point 13, test 10 and measures almost 0.8 m below still water level;
- Test 9 has a downward water level change on the shore side and an upward level change on the open sea side. The downward water level change causes a withdrawal arriving at the shore at first and the upward movement causes a wave top to propagate towards the ocean;
• The negative inclination causes a higher peak than an upward movement with the same maximum value;
• The withdrawal and the peak are equally represented in the total wavelength. The previous tests all have withdrawal with a bigger period than is the case in these tests;
• In appendix 6D in figure 6D.1 is shown that the development of the period in the bay is the same for the test 1 and tests 9, 10. The test 9 and 10 have the same period at every data point at the same time;
• The wavelengths of the test also hardly differ from each other as is expressed in appendix 6D in figure 6D.2.

6.3.8. **Comparison between inclination and parabolic water level change**

A comparison is made between test 9, the negative inclination, and a parabolic water level change according to the findings of Barrientos. In reality the parabolic water level change occurred just before the coast, but for a good comparison the origination area is moved a little to the open sea side on the same place the inclination is put at.

**Test 9:**
A ‘negative’ inclination from 1m till -1m over 93km.

![Figure 6.21 Water level change of test 9](image)

**Test 11:**
A ‘parabolic’ water level change with a maximum of 3m above sea level and a minimum of 2m below sea level over an area of 93km.

![Figure 6.22 Parabolic water level change test 11](image)

The plots of the tests are presented in The One-dimensional test report and the graphics are found in appendix 6E. The results are described below:

• The periods and the wavelengths of the two tests hardly differ from each other. The development near shore has the same pattern;
• The development of the peak has two important parameters. The shoaling factor and the friction of the bottom. The peak of test 11 reaches a maximum approximately 100km in front of the coast. It is possible that this development will change when the two-dimensional model of the bay area is used. After that the peak is decreasing because of the stronger friction of the bottom;
• The development of the withdrawal has a similar course. The maximum value is also reached 100km before the shore. This is the changeover of the trench to the bay area. It is possible that the decrease in value is caused by a to abrupt change of the bathymetry;
Towards the open boundary of the model the height of the peak slowly decreases with an increasing wavelength;

- The peak and withdrawal of test 9 are approximately twice as small as the values of test 11. This is probably caused by the difference in downward water level change. Test 9 has a maximum of 1m and test 11 has a maximum of 2m below still water level.

6.3.9. The input water level of the tsunami of 1960

A comparison is made between test 11 and the water level change area suggested by Barrientos. The last profile is used to create an input for the two-dimensional grid.

Test 11:
a 'parabolic' water level change with a maximum of 3m above sea level and a minimum of 2m below sea level over an area of 93km.

![Figure 6.23 Water level change area of test 11](image)

Test 12:
Test with profile suggested by Barrientos. The maximum value above still water level and the minimum water level is 2m below still water level. The origination area is stretched from the trench till the shoreline.

![Figure 6.24 Water level change tsunami 1960](image)

The plots are presented in The One-dimensional test report and the graphics are presented in appendix 6. The results of the tests are described below:

- The period and the wavelength increase quickly towards the open boundary of the model, because the origination area is situated so close to the shore. This is clear when compared with test 8. The development of the period and the wavelength is quite similar to each other;

- The trend of the development of the height of the peak is similar too, but the maximum peak of the Barrientos test is moved towards the coast. This is because the origination area is moved towards the coast too;

- The pattern of the Barrientos test is disturbed by the origination area. The development of the peak in the bay of test 11 is not disturbed by this. This can be the reason for the local differences between the two tests.
6.4. Conclusions of the one-dimensional model

The conclusions of the sensitivity study are presented here:

- The magnitude of the water level change has no influence on the period and wavelength;
- The horizontal size of the water level change area has an influence on the period and the wavelength. If the horizontal size is larger the period and the wavelength will be larger too;
- When the water level change is downward, the tsunami will be preceded by a withdrawal. When the water level change is upward, the tsunami will hit the shore as first;
- The period of the tsunami increases when propagating away from the origination area;
- The wavelength increases when propagating towards the open sea. Development towards the shore shows a different behaviour. The wavelength at first increases, but when moving more closely towards the shore it decreases. This is caused by the competition between the period and the water depth. Close to the shore the water depth is the dominant influence;
- If the magnitude of the water level change is larger, the height of the peak and the withdrawal will be larger too;
- The magnitude of the tsunami will be a slightly larger when the horizontal size of the origination area is larger;
- When the origination area is situated just in front of the coast the wavelength and the period are approximately twice as big as is the case when the water level change area is situated more towards the open boundary;
- An inclination causes a little bit lower period and wavelength;
- The parabolic water level change has the same development of the wavelength and the period near the shore as the inclination change;
- The peak of the parabolic water level change is approximately twice as high, which is counting for the withdrawal too;
- When the parabolic water level change area is moved closely towards the shore, the maximum peak will move to the shore too;
- The period of the tsunami caused by the Barrientos profile has a period of 30 minutes 40km in front of the coast. This results in an earthquake magnitude \( M = 7.7 \) according to expression 5.9. In this case is the tsunami magnitude \( m \) has a maximum value of 4. In Chile it is possible that this tsunami occurs.

A table with the results is presented in appendix 6G.

An output file of the one-dimensional model will be used as an input file in the two-dimensional model. The choice for a suitable place in the one-dimensional model for generation of an output file depends on a few factors. At first the size of the two-dimensional model is to be determined. After that the water depth at the open boundary just outside the bay will be known. Then the right place of the generation of the output file is determined.

6.5. The finer grid model

The finer grid model is a two-dimensional model. It covers the entire bay area of Golfo de Arauco. The island Santa Maria and the trench in front of the river Bio Bio perpendicular to
the coast are included in the grid. The two-dimensional model is used to determine the
height and the development of the tsunami in the bay area. The results of the simulation are
compared to the observations of the tsunami of 1960. The goal of this chapter is to
determine the influence of a tsunami on the future harbour to be built on Lomas Colorades.

6.5.1. Description of the area covered by the finer grid
The finer grid covers the entire bay area. It reaches in east-west direction from the ocean just
outside the bay area to the land of Chile with the site of the future harbour. In north-south
direction the grid covers the trench perpendicular to the coast in the north and the bay land
border in the south. The area measures approximately 61 km in the north-south direction
and 57 km in the east-west direction. The grid spacing is 850m in both directions. In figure
6.25 the bathymetry is shown.

An input file for MIKE21 is made with the help of a sea chart. The chart was scanned into
the computer and traced in AutoCAD. After that the AutoCAD file was translated into a
database which was introduced in Rockworks to digitise the points to the right grid size.
After that the file was translated into a file with the help of notebook to introduce the grid
into MIKE21. In MIKE21 two 3D views were composed which are shown in The Finer grid
model test report. The first picture shows the same view direction as figure 2.2 in the first
chapter of this report. The second picture is rotated over 120 degrees and gives a nice sight
into the bay area. The colour orange represents the first land area bordered on the water.
The deepest part of the water is indicated by a dark blue colour. The site is approximately
situated at grid point (55,56).
Figure 6.25 Bathymetry of the bay area with the site

6.5.2. The boundary conditions

The two-dimensional model has three boundaries. One boundary is situated at the open sea side or west side of the grid. It will be an open boundary and is used to introduce the results of the one-dimensional model into the bay area. A second boundary is found on the north side of the bay and will have a closed character, which means that the water level can vary freely and no water will cross this boundary. It is assumed that the bay has a mirror image with respect to this boundary. The last boundary is the water surface. An initial water level change is introduced into the simulation to approach the real situation as close as possible. The simulation starts with an instant water level change area, as is the case in the one-dimensional model.
The open sea boundary is used to introduce the results of the one-dimensional model into the two-dimensional model. The one-dimensional model has a uniform depth over the entire width. This is not the case with the open sea border of the finer grid. In figure 6.26 the course of the water depth along the open sea boundary is printed. The water depth of the open sea boundary varies in value between 0m and 220m, decreasing from the north to the south. The output file of the one-dimensional model will be made at a water depth of 172m. This is done because the north side of the boundary in the bay grid will have the biggest influence on the site of the future harbour. The tsunami will bend along the island Santa Maria towards the site.

![Figure 6.26 water depth along the open sea boundary](image)

The output of the one-dimensional will be translated into a useful input for the open sea boundary in the two-dimensional model.

6.5.3. **Determination of the grid size**

The number of points that is available is decisive for the grid size. The entry level version of MIKE21 has a maximum capacity of 5000 grid points. With this value an area of 3660km² must be covered. This means that the minimum size is 850m. The time step of the simulation is adopted from the output file of the one-dimensional model and has a value of 60sec. Variation of the time step in the finer grid shows that this value is small enough.

6.6. **Experiments with the finer grid model**

A total of five experiments are carried out with the two-dimensional grid. The effect of the water level change in the bay area and the influence of the input at the open sea boundary on the course of the tsunami in the bay area are determined.
6.6.1. The first experiment

The first experiment consists in a downward water level change of 1 m in the entire bay area.

![Figure 6.27 Cross section of the bay and Chile with downward water level change of 1 m in the bay area](image)

The water level in the beginning of the simulation has a value of 1m below still water level in the entire bay. As soon as the simulation is started the water level wants to return to the still water level. The only way is through the open sea boundary at the west side of the grid. The north boundary of the grid is a closed boundary, which means that no water can enter the bay area. In the 2D plots in The Finer grid model test report is shown that the bay is filled with water coming from the open sea boundary. The shape of the wave indicates that the wave turns around the island Santa Maria towards the southeast. The wave is propagating towards the site now. The effect of the water level change in the bay is of a minimum effect on the tsunami height. The maximum height that is reached is a little more than 1 m. A video is made of the water level change perpendicular to the coast in front of the site. The video is called levelA and is found on the disk included in the report. To activate the video the instruction written on the disk should be followed.

6.6.2. The second experiment

This experiment consists in a water level input on the open sea boundary. The water level in the bay remains constant in the beginning of the simulation. The influence of the input of the one-dimensional grid is determined.

![Figure 6.28 Input of the 1D model](image)

From the open sea boundary a wave is propagating into the bay area towards the coast. The wave is preceded by a withdrawal. The propagating pattern is the same as in the previous experiment. It will bend along the island Santa Maria to the southeast in the direction of the site. The bending in the two experiments is equally strong. After 20 minutes the first disturbance reaches the shore just above the site. The trench situated directly above the site causes this effect. The water depth has the biggest value at the trench that causes the biggest wave velocity. The disturbance is propagating along the shore towards both sides. On the north side the disturbance gets bigger because the boundaries of the grid are nearby. The withdrawal reflects on the shore and is getting bigger. The tsunami follows the same pattern. The image of the tsunami is very heavily disturbed by the boundaries. It looks like the tsunami fades away before it can reach the shore at the site.
6.6.3. The third experiment

In this experiment the first two experiments are combined. The downward water level movement of 1m and the input generated with the help of the one-dimensional model.

Figure 6.29 Input 1D model and water level change

The withdrawal is bigger than in the other two experiments. The pattern in the bay of the waves is sooner becoming irregular than in the other two experiments. The results of this experiment are not realistic.

6.6.4. The fourth experiment

In this experiment the parabolic water level change according to the article of Barrientos is used. The water level change is downward with respect to the still water level change and varies from 0m till 2m as a maximum at the open sea boundary.

Figure 6.30 Input Barrientos water level change

The bay area is filled with water from the open sea boundary, as is the case in the first experiment. The wave that is coming into the bay area again bends along the island Santa Maria. Because of the spread over a bigger area the wave is getting smaller when entering the second half of the bay.

6.6.5. The last experiment

The last experiment combines the parabolic water level change with the input generated with the help of the one-dimensional grid. This is the final experiment and is supposed to approach reality as closely as possible.

Figure 6.31 Barrientos water level change and input 1D model

A tsunami is propagating towards the shore with a lot less disturbances than in the other four experiments. The maximum height a tsunami will reach in the bay is in order of 1m according to this simulation. The observations of the tsunami of 1960 indicate that the wave height of the tsunami is much bigger than is simulated. How is this possible? An explanation can be that the simulation area is much too small. The boundaries of the grid have a big influence on the propagation of the tsunami.
6.7. **Comparison of the experiments with the tsunami of 1960**

The one-dimensional gives a maximum peak of approximately 2m. This should be compared to the observed values of the 1960 tsunami. In figure 5.2 is the aftershock presented. The numbers written in the figure are the inundation heights of the tsunami as noticed by Chilean civilians. The numbers in between parentheses represent the periods of the tsunami. In figure 5.2 is Golfo de Arauco situated north of the aftershock area. The tsunami inundation height is 2 - 3m. This is not a proper comparison because the one-dimensional is situated near the centre of the aftershock area. A comparison between the results of the one-dimensional model and the inundation heights near the city of Maullín in figure 5.2 should be made. The inundation height varies from 5 till 10m. The difference between the model and the observed heights is great. This is because the height in the simulation model is the wave height of the tsunami and the observed values are inundation heights. Inundation heights are depended on the steepness of the shore and the shape of the shore. For a proper comparison it is necessary to implement the entire aftershock area into the simulation model.

6.8. **Conclusions considering the finer grid model**

- The course of the tsunami is made clear. The tsunami enters the bay area at the open sea boundary after which it bends along the island Santa Maria towards the site of the future harbour. After that it travels to the north and the south part of the bay;
- The largest tsunami wave heights are found in the south part of the bay. Two waves meet in this part, one wave travelling via the north side of Santa Maria and one wave travelling via the south side of the island;
- The trench causes the decrease in tsunami wave height. The water depth at the trench has a greater value than the rest of the bay;
- Although the site of the future harbour is the first piece of land hit by the tsunami, the site is not the area in the bay whit the greatest risk. The south of the bay has the largest wave heights to expect.

6.9. **Conclusions considering the simulation program MIKE21**

- The simulation program MIKE21 gives a good image of the course of the tsunami;
- The generation process of the tsunami is made clear with the help of the program;
- The capacity in number of grid points of MIKE21 is not sufficient for a proper determination of the tsunami height. For the tsunami problem in front of the coast of Chile it is necessary to implement the entire aftershock area for the determination of the course of the tsunami;
- The translation of files from one grid size to a smaller grid size costs a great deal of time. The nest function is worthwhile considering.
7. Conclusions and recommendations

In this chapter the research done can be expressed in a few questions.

- What are the improvements after reviewing the harbour?
- What is the development of the tsunami in the bay area and what is the height of the tsunami in front of the site?
- What arrangements should be made to minimise the damage caused by a tsunami attack?

7.1. Review of the harbour layout

- The wish of the client to design a container terminal with 4 berths would lead to high waiting time. Four berths are not sufficient to handle 8,000,000 tonnes of cargo. The number of berthing places must be increased to 5 places. The occupancy rate in that case is in order of 56%, which is acceptable;
- The container storage area must be increased based on the used starting points;
- The estimate for the throughput of the harbour for the year 2005 in my opinion is too high. The present cargo volume in the 8th region of Chile is extrapolated. The estimate for the year 2005 will be approximately 6.5 times lower based on the throughput at this time. To be on the safe side the estimate can be as much as 4 million tonnes. In this case it is not necessary to increase the number of containers berths;
- The stopping distance is too short when making fast of the tugs is done inside the harbour. The stopping distance is sufficient when the tugs make fast outside the harbour. The operational wave climate allows making fast outside the harbour.

7.2. Determination of the tsunami course and wave height

- The course of the tsunami is made clear. The tsunami enters the bay area at the open sea boundary after which it bends along the island Santa Maria towards the site of the future harbour. After that it travels to the north and the south part of the bay;
- The largest tsunami wave heights are found in the south part of the bay. Two waves meet in this part, one wave travelling via the north side of Santa Maria and one wave travelling via the south side of the island;
- The trench causes the decrease in tsunami wave height. The water depth at the trench has a greater value than the rest of the bay;
- Although the site of the future harbour is the first piece of land hit by the tsunami, the site is not the area in the bay with the greatest risk. The south of the bay has the largest wave heights to expect;
- The best results will be derived when the entire aftershock area is implemented into the simulation program.

7.3. Possibilities for minimising damage due to tsunami wave attack

An earthquake as is explained in chapter 5 generates the tsunami. After approximately 30 minutes the tsunami will hit the site. This means that not much time is left to hide anything.
The most important thing is after an earthquake or tsunami warning that everybody will leave the site as quickly as possible.

It is possible to arrange the surface area of the terminals. The container terminal must be situated in the back site of the harbour otherwise the containers will be dragged with the wave across the entire harbour. While being dragged across the entire harbour the containers will hit the rest of the harbour equipment. The container terminal is to be designed in a way that the wave impact is as little as possible. The traffic lanes on the terminal must be in the propagation direction of the tsunami wave. The liquid bulk tanks can be placed under the ground. The tanks are safe for the tsunami attack. Of course it is still possible that the earthquake will damage the tanks, but that is not a subject of this study.

The space on the east side of the container terminal is reserved for real estate. This means that a lot of buildings are situated on this area. In case of a tsunami attack it is possible that the containers will hit the buildings. A better place for the container terminal in this case is at the south side of the harbour.
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Appendix 4A

Drawings of the breakwaters.
Appendix 4B

Drawing of the quay walls.
Appendix 4C

Graphics of the swell waves and the wind waves.

**figure 4C. 1 swell wave heights**

**figure 4C. 2 swell wave percentages**
figure 4C. 3 wind wave heights

figure 4C. 4 wind wave percentages
Appendix 4D

Cargo volume for the year 2005.
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Appendix 6A

Variation of the magnitude of the instant vertical water movement. Tests 1, 2 and 3.

Figure 6A. 1 Periods of test 1, 2 and 3

Figure 6A. 2 Wavelengths of test 1, 2 and 3
Figure 6A. 3 Periods of test 1,2 and 3 till 120 km

Figure 6A. 4 Wavelengths of test 1,2 and 3 till 120 km
Appendix 6B

Variation of the size of the horizontal water level change. Tests 1, 4 and 5.

Figure 6B. 1 Periods of tests 1, 4 and 5

Figure 6B. 2 Periods in more detail of tests 1, 4 and 5
Figure 6B. 3 Wavelengths of tests 1, 4 and 5

Figure 6B. 4 Wavelengths in more detail of tests 1, 4 and 5
Appendix 6C

Variation of the place of the water level change area. Standard test, test 7, 1 and 8.

Figure 6C. 1 Periods of the standard test and tests 7, 1 and 8

Figure 6C. 2 Periods in more detail
Figure 6C.3 Wavelengths of the standard test and tests 7, 1 and 8

Figure 6C.4 Wavelengths in more detail
Appendix 6D

The influence of the inclination of the origination area. Tests 1, 9 and 10.

Figure 6D. 1 Periods of tests 1, 9 and 10

Figure 6D. 2 Wavelengths of tests 1, 9 and 10
Appendix 6E

Comparison between inclination and parabolic water level change. Tests 9 and 11.

Figure 6E. 1 Periods of test 9 and 11

Figure 6E. 2 Periods in more detail
Figure 6E. 3 Wavelengths of test 9 and 11

Figure 6E. 4 Wavelengths in more detail
Figure 6E. 5 Withdrawal height of test 9 and 11

Figure 6E. 6 Withdrawal height in more detail
Figure 6E. 7 Peak height of test 9 and 11

Figure 6E. 8 Peak heights in more detail
Appendix 6F

The input water level of the tsunami of 1960 according to Barrientos. Test 11 and Barrientos profile.

Figure 6F. 1 Periods of test 11 and Barrientos profile

Figure 6F. 2 Periods in more detail
Figure 6F. 3 Wavelengths of test 11 and Barrientos profile

Figure 6F. 4 Wavelengths in more detail
Figure 6F. 5 Withdrawal height of test 11 and Barrientos

Figure 6F. 6 Withdrawal in more detail
Figure 6F. 7 Peak heights of test 11 and Barrientos profile

Figure 6F. 8 Peak heights in more detail
Appendix 6G

Tables of all tests
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