Vung Tau – Go Cong Dam
Vietnam

Preliminary Design Study
Final report

Project group Flood Defence HCMC
TU Delft

Water Resources University, Second Base
Institute for Water and Environment Research

October 2011
**General notice to the reader:**

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

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

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

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

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

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Delft University of Technology
Project group Flood Defence HCMC
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This report is the result of the multidisciplinary project, part of our MSc-study Civil Engineering at Delft University of Technology, and is therefore written for educational purposes.

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PREFACE

This report is the result of the multidisciplinary master project we performed as part of our study at Delft University of Technology. We are a group of five students with different specialisations (Coastal Engineering, Geo-engineering, Hydraulic Structures and Water Management) and decided to do this project abroad, to experience how it is to work in a different culture with different working methods.

Via contacts between the Water Resources University (WRU) in Vietnam and Delft University of Technology we were invited to work on the design of the Vung Tau – Go Cong dam for a time span of 9 weeks, starting in the middle of August. We were stationed at the second base of the WRU in Ho Chi Minh City; at the Institute for Water and Environment Research (IWER).

The project leader of the design team, Mr. Le Xuan Bao MSc., supervised our work on the project during our stay in Vietnam. The supervisors from the TU Delft were ir. Schiereck and prof.dr.ir. Stive.

We would like to thank everybody who helped us during the project. First of all we would like to thank Mr. Schiereck, who helped us starting up the project, supervised our work during the project and even sacrificed a weekend in Ho Chi Minh City to give us a crash course on closure dams. Also we like to thank Mr. Le Xuan Bao, who always made time to help us finding information and discuss with us about the project. Furthermore we want to thank Mr. Verhagen, who informed us about this project, provided information and gave useful advice. We also thank Prof. Dr. Ir. Stive as second supervisor.

We like to thank people from external organisations for their advice and input: Mr. Duy, Mr. Nguyen and Mr. Van.

Additionally we are grateful to the staff of teachers of the WRU, who were working on the project themselves and were eager to help us. Mr. An, Mr. Binh, Mr. Tam, Mr. Trung, and all the others, thanks for your help and for the soccer games, the karaoke, the parties and the funny moments in the roller-disco.

Finally we would like to thank the companies that supported us, i.e. Ballast Nedam, Damen and Iv-Groep. In addition, we also thank the Executive Board Fund of the TU Delft for their contribution. This support made it possible for us to gain a valuable study experience in a foreign country.

Ho Chi Minh City, 13 October 2011

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SUMMARY

The area around Ho Chi Minh City (HCMC) faces flooding and salt intrusion problems. Flooding problems are caused by intensive rainfall in the city, high river discharges and high tides on sea. Dr. Hoc, the vice-minister of MARD (Vietnamese Ministry of Agriculture and Rural Development) proposed to construct a hydraulic structure downstream of HCMC. This will solve both the flooding and siltation problem. A possibility is to construct a dam between Vung Tau and Go Cong. This solution has been worked out into a preliminary design, which includes a design for the cross section and an investigation into the closure method.

The hydraulic system where the project will be constructed consists of the Saigon - Dong Nai river system and the East Sea, and can be classified as an estuary with a (mainly semi-diurnal) mixed tide. Waves are moderate in the area. A dam will be constructed between Go Cong and the Dong Nai navigation channel. Over the Dong-Nai navigation channel to Vung Tau a bridge is planned, to minimise navigation delays and changes in tidal and salinity range in the Can Gio mangrove forest. To make a closed basin, Can Thanh will be connected with the dam near the Dong Nai navigation channel. Ship locks and discharge sluices will be constructed at the sides of the existing navigation channel in the Soi Rap. The final closure of the dam will take place in the Soi Rap navigation channel. A road on the dam will connect Vung Tau with Go Cong.

Because there is no subsoil information available at the location of the dam, the soil layering and parameters are estimated based on information from locations in the vicinity of the dam. Three different soil profiles were drawn up. Measures have to be taken to increase the bearing capacity of the subsoil and decrease settlements during the lifetime of the dam. If thick layers of weak subsoil is present, drainage in combination with pre-loading will be the best solution. In case of a stronger subsoil partial replacement with sand is favourable.

To build the dam, the use of local material is a cheap and easy solution. Near Vung Tau a number of areas are present where sand can be obtained. Rock can be found in mountains in the neighbourhood of Vung Tau. Since there is already a stone quarry in this area, it is assumed that the rock is of good quality. By means of a Multi Criteria Analysis it was found that a building a dam with a sand core is the best solution in this project.

Degradation or even failure of (elements of) the structure may occur as a result of loadings during the operational phase or in accidental situations. The principal failure mechanisms for the VT-GC dam are checked. Slope, crest and toe protections were designed to prevent failure. Two designs were made for the cross section of the dam: one with an outer slope revetment made of asphalt and one for an outer slope revetment made of rock. The biggest differences in these designs can be found in the crest height and layer thicknesses.

The construction of the dam should start with the construction of the bridge and work islands. Next, the discharge sluices and navigation locks will be made in building pits. Then the dam will be constructed. This starts with the foundation and scour protection which will be constructed using barges. Waterborne equipment is preferred because of the easier logistics and larger capacities. The core material, filter layers, underlayer, toe and armour layer will be constructed layer by layer to be able to raise the relatively steep slopes. Where water depths are too low, cranes operating from pontoons or rolling equipment can be used. After completion of the core and revetment, the crest is constructed and the dam can be finished with the construction of the road.

The closure of the final gap is a very important aspect within the construction of the dam. A storage area approach is used to determine the system behaviour during closure. High velocities in the final gap make
it impossible to close the gap with sand. A bottom protection is necessary in the final gap to prevent a scour hole, caused by high velocities, from developing too close to the dam. The gap is closed with coarser material than sand. A combination of vertical and horizontal closure will be used. First barges dump stones to a level of MSL -5 m. When the water is too shallow, dumping will continue from the sides of the closure gap. The final dam profile in the closure gap will be similar to the regular dam sections.

A general estimation of the cost for the total project, based on costs in reference projects, is 3.1 billion US dollar.

The Dong Thap Muoi region is a very important agricultural area for the production of rice. Due to the low river discharges and the large influence of the tide, the salt concentrations in the Vam Co rivers are high, causing problems for the farmers. An estimation of the salt intrusion length was made using a predictive steady-state salt intrusion model, according to Savenije. Unfortunately it was not possible to perform salinity measurements in the Vam Co River, so data from three fixed measurement stations was used. With this data, it was possible to fit the model roughly but more data is needed to make the model more accurate.
1 \hspace{1cm} \textbf{INTRODUCTION}

In this chapter the problem will be described, and the subject and research questions will be introduced. Furthermore the relation to other projects and a report outline will be presented.

1.1 \hspace{1cm} \textbf{Problem description}

The area in and around Ho Chi Minh City (HCMC) faces two major hydraulic problems:

1. Flooding of the city.
   Large parts of the city are built on areas with a ground level only a few meters above sea level.
2. Salt intrusion.
   This is especially a problem in the Dong Thap Muoi region, an important area for the rice production, but also for the drinking water production in HCMC.

An overview of the HCMC area is displayed in Figure 1.1.

The flooding of the city is caused by (a combination of) the following three causes.

a. Intensive rainfall in the city
b. High river discharges
c. High tides on the sea, which affects the water level in rivers even upstream of HCMC

In the map in Figure 1.2 the areas frequently flooded by tide and/or rain are indicated.

\begin{figure}[h]
\centering
\includegraphics[scale=0.5]{figure1_1.png}
\caption{Map of the Ho Chi Minh City delta region, with in red one of the possible trajectories of the structure}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[scale=0.5]{figure1_2.png}
\caption{Map of frequently flooding areas. The light blue colour indicates areas flooded by tidal influences and the red colour indicates areas flooded by rain}
\end{figure}
The flooding of HCMC is closely related to three other problems:

1. **The design of the urban drainage system**
   As a result of continuous expansion of the urban area, with a high percentage of paved area, the capacity of the urban drainage system is insufficient to discharge the rainfall. Moreover the drainage system is in open connection with the rivers. Therefore the city can flood when water levels in the rivers rise. Because of bad maintenance, the drainage capacity of the sewer system is low.

2. **Sea level rise**
   With large certainty a worldwide rise of the sea level is predicted. A rise of the sea level would mean that the city will flood more frequently.

3. **Subsidence of the city**
   Large parts of the city are founded on soft soils. Due to extensive groundwater withdrawal the soil settles and the city subsides.

In order to solve the flooding problem various plans have been drafted and partly executed to improve the flood situation. In 2008 ‘the master plan of water level control for flood protection in HCMC area’ by the Ministry of Agriculture and Rural Developments (MARD) was approved by Vietnam’s Prime Minister. Emphasis of this plan is on controlling the water levels in the open channels independent of those in the rivers. According to this Master Plan this can be done by constructing 12 tidal drainage gates, embankments, retention basins and supporting pumping stations.

Another solution, which solves both the flooding and siltation problem the vice-minister of MARD (Vietnamese Ministry of Agriculture and Rural Development), dr. Hoc, came up with the idea to search for a hydraulic structure downstream of HCMC. This hydraulic structure can be a good alternative for ‘the master plan of water level control for flood protection in HCMC area’.

One of these downstream possibilities is to construct a dam or barrier between Vung Tau and Go Cong. This structure, sometimes called the ‘superdike’ is the subject of this study. This large closure involves several difficulties. Besides investigating flood protection one has to take into account that:

1. **Shipping must remain possible, and delay should be limited**
   Near HCMC, in the Soi Rap and the Long Tau navigation up to 100,000 DWT takes place. During and after construction of the dam navigation should be possible without too much delay.

2. **The drainage of river discharge to the sea must be sufficient**
   The estuary of the Saigon, Dong Nai and Vam Co rivers will be closed off. Sufficient drainage capacity is needed to be able to discharge the river discharges to the sea.

3. **The Can Gio mangrove forest should be preserved**
   Salinity levels and natural tide range may not alter such that the Can Gio mangrove suffers from the consequences of the closing structure.

4. **Closing the final gap in an estuary is rather complex due to high velocities**
   During construction of the dam the velocity in the gap increases when the gap area decreases. This makes the final closure difficult, because sediment will be eroded.

### 1.2 Project outline

In this project a preliminary design for the Vung Tau-Go Cong dam (VT-GC dam) will be made. The main focus of this project is twofold:

1. **Designing the cross section of the dam**
An investigation into the situation and the available materials will lead to a choice for the dam type. For this dam calculation of the required crest height, foundation, protection, slopes and berms will result in a final design. Technical drawings will be made to present the design.

2. Investigating the building method
An investigation will be done into possible ways to construct the dam. Special attention will be paid to the closure of the final gap: different closure methods will be discussed and the velocities in the closure gap will be estimated. The required material and equipment for the closure will be presented. Finally an overview of the building sequence of the total project will be drawn up.

Furthermore the trajectory of the dam, the location of elements like discharge sluices and the influence of the dam on shipping will be discussed. Also an estimation of the total costs of the project will be made.

1.3 Relation to other projects
Multiple organisations are involved in this project. One of them is the Vietnamese Institute for Water and Environment Research (IWER), which is working on the design of the dam. Also, the Royal Netherlands Embassy of Vietnam requested Delft University of Technology (TU Delft) to perform a prefeasibility study on this hydraulic structure.

This project is performed for the IWER under supervision of the TU Delft. It will catch up with the graduation work of Ms. Olga Sturm (MSc. student TU Delft), who performed a prefeasibility study on a barrier downstream of HCMC.

1.4 Report outline
In chapter 1 an analysis of the problem will be made. This will include a situation analysis (the hydraulic system and the subsoil) as well as the stakeholders, the requirements and reference projects.

Once the problem has been analysed, in chapter 3 choices will be made for the trajectory of the dam and the location and types of special structures required, like navigation locks and discharge sluices. The design of the dam and the building method will be based on these choices.

The design of the cross section will be treated in chapter 4. First the required crest height will be determined step by step. Once this height is determined the optimal alternative for the dam will be determined by means of a multi-criteria analysis. Once the most favourable alternative is chosen the design of the foundation will be treated, followed by calculations of the failure mechanisms and the amount and type of protection needed. The chapter will be concluded with an overview of the design and some technical drawings.

An important aspect in the design of a closure dam in a river estuary is the design of the closure gap. This will be the topic of the fifth chapter. The maximum velocities in the closure gap will be calculated, and after this the different closure techniques will be covered and one will be chosen and elaborated.

Chapter 6 will continue with the building method. Both the construction method of the dam and the construction of the closure gap will be treated, as well as the construction sequence of the total project.

In chapter 0 an estimation of the costs of the dam will be made, and in chapter 0 some results of studies related to the project are discussed (salt intrusion and velocity measurements). The report will conclude with drawing conclusions and giving recommendations in chapter 9.
2 ANALYSIS

Before making a design an analysis of the problem is made. This includes an analysis of the current situation, the stakeholders, the requirements of the structure, as well as a study into reference projects.

2.1 Situation description

To be able to make a good design first the system in which the structure will function has to be analysed. This way special points of attention can be identified. In the following sections therefore the hydraulic system, the tide, frequency of storms and the subsoil will be looked into in more detail.

2.1.1 Hydraulic system

In this section the hydraulic system in which the project area is situated will be introduced. The hydraulic system consists of the Saigon - Dong Nai river system and the East Sea. The location where a flood defence structure will be constructed will be influenced by both aspects: rivers and sea, and can therefore be classified as an estuary. In this section the river system and the tidal characteristics are described. With theory from literature the delta system can be classified.

2.1.1.1 Saigon - Dong Nai river system

The Saigon - Dong Nai river system consists of several rivers all leading in roughly two river arms to the estuary where the flood defence structure is planned to be built, see Figure 2.1.

Figure 2.1: Saigon - Dong Nai river system
The water levels of the rivers in the system are the result of the tide and the river discharge. The river discharge is mainly controlled by the discharge from large storage basins and direct run off due to rain.

The river system consists of the main rivers Dong Nai, La Ngã, Be, Saigon, Vam Co Dong and Vam Co Tay. Upstream of HCMC there are three reservoirs, of which two reservoirs are for irrigation purposes (Dong Nai and Phuoc Hoa), and the third one is to produce energy (Tri An). The discharges of the Vam Co rivers are not controlled by a hydraulic structure, so they are more directly linked to rainfall.

To give an idea of the amount of discharges from the different rivers, the measured peak discharges of a period of 5 years and the calculated design discharges are shown in Table 2.1.

Table 2.1: Impression of river discharges in project area (Sturm, 2011)

<table>
<thead>
<tr>
<th>River</th>
<th>Reservoir</th>
<th>Measured discharge: Average; extreme (m$^3$/s)</th>
<th>Designed discharge (m$^3$/s)</th>
<th>Flood duration (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dong Nai</td>
<td>Tri An and Phuoc Ha</td>
<td>750 25000</td>
<td>19000</td>
<td>3</td>
</tr>
<tr>
<td>Saigon</td>
<td>Dau Tieng</td>
<td>200 600</td>
<td>1500</td>
<td>6</td>
</tr>
<tr>
<td>Vam Co</td>
<td>-</td>
<td>250 2000</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

2.1.1.2 Astronomical tide

The character of the tide in the project area will be further introduced in this section. The tide at Vung Tau will be used to characterize the tide in the project area. Although this is not exactly true it is a good approximation. From measurements it is concluded that the tidal amplitude in the project area is the same and the delay in the peak varies between 15 and 30 minutes.

The typical character of the tide at Vung Tau can be observed by looking at measured water levels, see Figure 2.2.

Figure 2.2: Measured water level at Vung Tau in January 2005

Based on the tidal constituents, the type of tide at Vung Tau can also be checked by calculating the $f$-number. The $f$-number is a ratio between the amplitude of the principal tidal constituents:

$$f = \frac{K_1 + O_1}{M_2 + S_2} = \frac{0.60 + 0.45}{0.79 + 0.31} = 0.95$$ (2.1)
According to Table 2.3, this f-number corresponds with a mixed tide which is mainly semi-diurnal. Values for the tidal constituents were obtained from the British Admiralty Chart (Sturm, 2011).

Table 2.2: Principal tidal constituents

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Tidal Character</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_1$</td>
<td>Diurnal: Lunar-solar declinational</td>
</tr>
<tr>
<td>$O_1$</td>
<td>Diurnal: Principal lunar</td>
</tr>
<tr>
<td>$M_2$</td>
<td>Semidiurnal: Principal lunar</td>
</tr>
<tr>
<td>$S_2$</td>
<td>Semidiurnal: Principal solar</td>
</tr>
</tbody>
</table>

Table 2.3: f-numbers of tide

<table>
<thead>
<tr>
<th>f-number</th>
<th>Tidal character</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f &lt; 0.25$</td>
<td>Semi-diurnal</td>
</tr>
<tr>
<td>$0.25 &lt; f &lt; 1.5$</td>
<td>Mixed, mainly semi-diurnal</td>
</tr>
<tr>
<td>$1.5 &lt; f &lt; 3$</td>
<td>Mixed, mainly diurnal</td>
</tr>
<tr>
<td>$f &gt; 3$</td>
<td>Diurnal</td>
</tr>
</tbody>
</table>

The mean water levels at Vung Tau are provided in Table 2.4.

Table 2.4: The mean water levels at Vung Tau (UKHO, 1993)

<table>
<thead>
<tr>
<th>Water level (m to MSL)</th>
<th>MHHW</th>
<th>MLHW</th>
<th>MHLW</th>
<th>MLLW</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>+1.0</td>
<td>+0.7</td>
<td>-0.3</td>
<td>-1.6</td>
</tr>
</tbody>
</table>

2.1.1.3 Classification of the delta system

The formation of a delta depends on the interaction between the river flow and sediment supply on the one hand and the distribution of the river sediment by waves and tidal currents on the other hand (Bosboom & Stive, 2010).
The Saigon - Dong Nai delta can be classified as a tide-dominated delta with influence from waves. There is a large range between the high and low tides, with strong tidal currents. The wave height is moderate and alongshore currents are weak. The delta resembles an estuary because of its embayed setting of tidal flats. Due to the wave action there are no bars at the river mouth.

2.1.2 Frequency of storms at the project area

In the South China Sea, there are two centres where storms frequently form and develop: the coastal north-east of the Philippines and the south-east of Hainam Island. According to Le Trong Dao, et al. (2000), the storms hitting Vietnam’s coast are non-uniformly distributed: the frequency of storms reduces form North to South. 58.4% of the storms hit the northern coast, 36.85% the central coast and only 4.8% the southern coast. HCMC is affected by approximately 10% of all storms that hit Vietnam.

These extreme winds are caused by northers, extratropical cyclones, thunder storms and typhoons. Northers are resulting from movement of high pressure polar air masses south or southwestward off the Asia mainland. Winds gusting to 31 m/s may develop. Extratropical cyclones can cause wind gusts of 21 m/s and thunderstorms may cause wind gusts up to 36 m/s.
The typhoon season in Vietnam generally takes place from 1 May to 1 January, with the largest probability of typhoons hitting the coast from July to October during the rainy season. Typhoons approaching the Asian mainland are far less frequent in latitudes south of 10°N and generally occur during November and December (Table 2.5).

Table 2.5: Total number of Typhoons with centre striking or coming within 100 miles of the Asia Mainland, South China Sea in 50 years. (Lyon Associates inc. Consulting Engineers, 1974)

<table>
<thead>
<tr>
<th>Latitude</th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 to 10° N</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>10 to 15°</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>14</td>
<td>16</td>
<td>2</td>
<td>37</td>
</tr>
<tr>
<td>15 to 20°</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>2</td>
<td>9</td>
<td>6</td>
<td>29</td>
<td>20</td>
<td>6</td>
<td>0</td>
<td>74</td>
</tr>
<tr>
<td>above 20° N</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>6</td>
<td>14</td>
<td>43</td>
<td>32</td>
<td>37</td>
<td>17</td>
<td>7</td>
<td>0</td>
<td>156</td>
</tr>
</tbody>
</table>

Storms that hit the Vietnamese coast are generally small and deep, which means that the storm-affected area is small (40 to 100 km) but the air pressure gradient between the centre and the outer skirt is large. Wind speeds may reach 56 m/s. To the south, the intensity of storms gradually decreases. At Vung Tau the wind speeds are therefore lower.

Extreme winds and their return period expected near Vung Tau are displayed in Table 2.6. These wind values were derived using a statistical technique, based upon distributions of past occurrences of storms and high winds in various parts of the world.

Table 2.6: Extreme wind speeds near Vung Tau as a function of Return Period (Lyon Associates inc. Consulting Engineers, 1974)

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sustained wind (m/s)</td>
<td>25</td>
<td>28</td>
<td>33</td>
<td>37</td>
</tr>
<tr>
<td>Instantaneous speeds (gusts) (m/s)</td>
<td>43</td>
<td>48</td>
<td>56</td>
<td>62</td>
</tr>
</tbody>
</table>

The last 15 to 20 years the number tropical storms hitting HCMC tends to increase; over the last 60 years 12 large tropical storms have affected the city. For the design of the VT-GC dam this tendency should be taken into account. Tropical storms were quite rare until recently. The projected warmer sea surface temperature in the South China Sea will intensify storms landing in the area of HCMC.
2.1.3 Subsoil conditions

Because there is no subsoil information available at the location of the dam the soil layering and parameters are estimated based on information in the vicinity of the dam. The following sources are used:

1. Thu Bo barrier data (for the deeper layering and parameters)
2. Information from the Southern Institute of Water Resources Planning (SIWRP)
   a. Boring until MSL -18 m at Go Cong (for the layering)
   b. Seismic data near Vung Tau (for the layering)
   c. Material type in the top 30 cm of the seabed
3. A report about a Vung Tau port study (Lyon Associates inc. Consulting Engineers, 1974) (for the layering on top layers and parameters)
4. For unknown soil parameters: the Dutch geotechnical code NEN 6740, where typical Dutch parameters are given for several soil types (Dutch Normalisation Institute NEN, 2006)

With this information, which can be found in APPENDIX B, three different soil profiles are drawn up, because of the big differences in layering at different locations. These three profiles are:

1. ‘Weak Vung Tau’: A profile based on the worst conditions observed in the data. This profile is found in Vung Tau port study. Data below MSL -20 m is based upon data from the Tho Bo barrier project.
2. ‘Strong Vung Tau’: A profile based on the strongest layering observed in the Vung Tau port study. Data below MSL -20 m is based upon data from the Tho Bo barrier project.
3. ‘Go Cong’: A profile based on the results of a boring in Go Cong. Thu Bo barrier soil parameters are used for similar layers.

The three profiles are drawn in Figure 2.5. All the profiles consist of soft soil top layers (mud or clay), and in the deeper layers sand is assumed to be present.

Assumed parameters for the three layerings are presented in Table 2.7, Table 2.8 and Table 2.9. An explanation for which parameters are based on what information is given in APPENDIX B.
### Table 2.7: Weak Vung Tau profile and parameters

<table>
<thead>
<tr>
<th>Category</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Layer 1</th>
<th>Layer 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description in the source of information</td>
<td></td>
<td></td>
<td></td>
<td>Soft marine clay</td>
<td>Sand, fine to medium, loose to medium compacted</td>
</tr>
<tr>
<td>Geological era</td>
<td></td>
<td></td>
<td></td>
<td>Holocene</td>
<td>Pleistocene</td>
</tr>
<tr>
<td>Geometry</td>
<td>Depth top of layer</td>
<td>d</td>
<td>[m]</td>
<td>0 to -12</td>
<td>-25</td>
</tr>
<tr>
<td></td>
<td>below mean sea level</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thickness</td>
<td>h</td>
<td>[m]</td>
<td>13 - 25</td>
<td>27</td>
</tr>
<tr>
<td>SPT blow count</td>
<td>N</td>
<td>[¥]</td>
<td></td>
<td>0</td>
<td>20</td>
</tr>
<tr>
<td>Consistency based on SPT</td>
<td></td>
<td></td>
<td></td>
<td>Very weak</td>
<td>(Range 11 – 29)</td>
</tr>
<tr>
<td>Soil type assumed to use Dutch code for unknown parameters</td>
<td></td>
<td></td>
<td></td>
<td>Clay, clean, soft</td>
<td>Sand, clean, medium</td>
</tr>
<tr>
<td>Unit weights</td>
<td>Saturated</td>
<td>γ_w</td>
<td>[kN/m³]</td>
<td>17.85</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Dry</td>
<td>γ_d</td>
<td>[kN/m³]</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Undrained</td>
<td>Cohesion</td>
<td>c_u</td>
<td>[kPa]</td>
<td>9.8</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Friction angle</td>
<td>φ</td>
<td>[deg]</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Effective</td>
<td>Cohesion</td>
<td>c’</td>
<td>[kPa]</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Friction angle</td>
<td>φ’</td>
<td>[deg]</td>
<td>17.5</td>
<td>32.5</td>
</tr>
<tr>
<td>Consolidation coefficient</td>
<td>c_v</td>
<td>[m²/s]</td>
<td>3.2·10⁻⁸</td>
<td>1.0·10⁻³</td>
<td></td>
</tr>
<tr>
<td>Bjerrum settlement parameters</td>
<td>Compression index</td>
<td>C_c</td>
<td>[-]</td>
<td>0.94</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>Creep index</td>
<td>C_a</td>
<td>[-]</td>
<td>0.0131</td>
<td>0</td>
</tr>
<tr>
<td>Permeability</td>
<td>k</td>
<td>[cm/s]</td>
<td>5.0·10⁻⁷</td>
<td>5.8·10⁻³</td>
<td></td>
</tr>
<tr>
<td>Void ratio</td>
<td>e₀</td>
<td>[-]</td>
<td>2.0</td>
<td>0.633</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2.8: Strong Vung Tau profile and parameters

<table>
<thead>
<tr>
<th>Category</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Layer 1</th>
<th>Layer 2</th>
<th>Layer 3</th>
<th>Layer 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description in the source of information</td>
<td></td>
<td></td>
<td></td>
<td>Soft marine clay</td>
<td>Clayey sand</td>
<td>Soft marine clay</td>
<td>Sand, fine to medium, loose to medium compacted</td>
</tr>
<tr>
<td>Geological era</td>
<td></td>
<td></td>
<td></td>
<td>Holocene</td>
<td>Holocene</td>
<td>Holocene</td>
<td>Pleistocene</td>
</tr>
<tr>
<td>Geometry</td>
<td>Depth top of layer</td>
<td>d</td>
<td>[m]</td>
<td>0 to -8</td>
<td>-8</td>
<td>-10</td>
<td>-25</td>
</tr>
<tr>
<td></td>
<td>below mean sea level</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Thickness</td>
<td>h</td>
<td>[m]</td>
<td>0 - 8</td>
<td>2</td>
<td>15</td>
<td>27</td>
</tr>
<tr>
<td>SPT blow count</td>
<td>N</td>
<td>[¥]</td>
<td></td>
<td>0</td>
<td>15</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>Consistency based on SPT</td>
<td></td>
<td></td>
<td></td>
<td>Very weak</td>
<td>Medium</td>
<td>Stiff</td>
<td>(Range 11 – 29)</td>
</tr>
<tr>
<td>Soil type assumed to use Dutch code for unknown parameters</td>
<td></td>
<td></td>
<td></td>
<td>Clay, clean, soft</td>
<td>Sand, very silty, clayey</td>
<td>Clay, clean, stiff</td>
<td>Medium Sand, clean, medium</td>
</tr>
<tr>
<td>Unit weights</td>
<td>Saturated</td>
<td>γ_w</td>
<td>[kN/m³]</td>
<td>17.85</td>
<td>20</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Dry</td>
<td>γ_d</td>
<td>[kN/m³]</td>
<td>18</td>
<td>19</td>
<td>19</td>
<td>18</td>
</tr>
<tr>
<td>Undrained</td>
<td>Cohesion</td>
<td>c_u</td>
<td>[kPa]</td>
<td>9.8</td>
<td>-</td>
<td>50</td>
<td>(Range 35.80 - 89.40)</td>
</tr>
<tr>
<td></td>
<td>Friction angle</td>
<td>φ</td>
<td>[deg]</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Effective</td>
<td>Cohesion</td>
<td>c’</td>
<td>[kPa]</td>
<td>0</td>
<td>0</td>
<td>13</td>
<td>32.5</td>
</tr>
<tr>
<td></td>
<td>Friction angle</td>
<td>φ’</td>
<td>[deg]</td>
<td>17.5</td>
<td>25</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Consolidation coefficient</td>
<td>c_v</td>
<td>[m²/s]</td>
<td>3.2·10⁻⁸</td>
<td>1.0·10⁻³</td>
<td>1.0·10⁻³</td>
<td>1.0·10⁻³</td>
<td></td>
</tr>
<tr>
<td>Bjerrum settlement parameters</td>
<td>Compression index</td>
<td>C_c</td>
<td>[-]</td>
<td>0.94</td>
<td>0.015</td>
<td>0.16</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>Creep index</td>
<td>C_a</td>
<td>[-]</td>
<td>0.008</td>
<td>0.080</td>
<td>0.080</td>
<td>0.038</td>
</tr>
<tr>
<td></td>
<td>C_a / (1+e₀)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeability</td>
<td>k</td>
<td>[cm/s]</td>
<td>5.0·10⁻⁷</td>
<td>5.0·10⁻⁷</td>
<td>5.0·10⁻⁷</td>
<td>5.8·10⁻³</td>
<td></td>
</tr>
<tr>
<td>Void ratio</td>
<td>e₀</td>
<td>[-]</td>
<td>2.0</td>
<td>0.7</td>
<td>1.0</td>
<td>0.633</td>
<td></td>
</tr>
</tbody>
</table>
### Table 2.9: Go Cong profile and parameters

<table>
<thead>
<tr>
<th>Category</th>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Layer 1</th>
<th>Layer 2</th>
<th>Layer 3</th>
<th>Layer 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description in the source of information</td>
<td></td>
<td></td>
<td></td>
<td>Mud</td>
<td>Sand, muddy</td>
<td>Clay, silty</td>
<td>Sand, fine to medium, loose to medium compacted</td>
</tr>
<tr>
<td>Geometry</td>
<td>Depth top of layer below mean sea level</td>
<td>d</td>
<td>[m]</td>
<td>-3</td>
<td>-12</td>
<td>-15</td>
<td>-18</td>
</tr>
<tr>
<td></td>
<td>Thickness</td>
<td>h</td>
<td>[m]</td>
<td>9</td>
<td>3</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>SPT blow count</td>
<td></td>
<td>N</td>
<td>[#]</td>
<td>&lt; 2</td>
<td>-</td>
<td>&lt; 10-19</td>
<td>11-29</td>
</tr>
<tr>
<td>Consistency based on SPT</td>
<td></td>
<td></td>
<td></td>
<td>Very soft</td>
<td>-</td>
<td>Stiff</td>
<td>Medium</td>
</tr>
<tr>
<td>Soil type assumed to use Dutch code for unknown parameters</td>
<td></td>
<td></td>
<td></td>
<td>Clay, clean, soft</td>
<td>Sand, very silty, clayey</td>
<td>Clay, clean, stiff</td>
<td>Sand, clean, medium</td>
</tr>
<tr>
<td>Unit weights</td>
<td>Saturated</td>
<td>γ_w</td>
<td>[kN/m^3]</td>
<td>14.6</td>
<td>20</td>
<td>19</td>
<td>19.2</td>
</tr>
<tr>
<td></td>
<td>Dry</td>
<td>γ_d</td>
<td>[kN/m^3]</td>
<td>14.6</td>
<td>18</td>
<td>19</td>
<td>15.8</td>
</tr>
<tr>
<td>Undrained</td>
<td>Cohesion</td>
<td>c_u</td>
<td>[kPa]</td>
<td>11.8</td>
<td>-</td>
<td>33.2</td>
<td>-</td>
</tr>
<tr>
<td>Friction angle</td>
<td></td>
<td>φ_u</td>
<td>[deg]</td>
<td>1.5</td>
<td>-</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>Effective</td>
<td>Cohesion</td>
<td>c'</td>
<td>[kPa]</td>
<td>8.8</td>
<td>0</td>
<td>25.8</td>
<td>5</td>
</tr>
<tr>
<td>Friction angle</td>
<td></td>
<td>φ'</td>
<td>[deg]</td>
<td>23</td>
<td>25</td>
<td>22.8</td>
<td>28.6</td>
</tr>
<tr>
<td>Consolidation coefficient</td>
<td></td>
<td>c_v</td>
<td>[m^2/s]</td>
<td>4.5x10^-9</td>
<td>1.0x10^-4</td>
<td>3.7x10^-7</td>
<td>1.0x10^-7</td>
</tr>
<tr>
<td>Bjerrum settlement parameters</td>
<td>Compression index</td>
<td>C_c</td>
<td>[-]</td>
<td>1.01</td>
<td>0.014</td>
<td>0.10</td>
<td>0.023</td>
</tr>
<tr>
<td></td>
<td>C_i/ (1+e_0)</td>
<td>C_i</td>
<td>[-]</td>
<td>0.009</td>
<td>0.009</td>
<td>0.038</td>
<td>0.038</td>
</tr>
<tr>
<td></td>
<td>Creep index</td>
<td>C_v</td>
<td>[-]</td>
<td>0.013</td>
<td>0</td>
<td>0.004</td>
<td>0</td>
</tr>
<tr>
<td>Permeability</td>
<td></td>
<td>k</td>
<td>[cm/s]</td>
<td>7.6x10^-5</td>
<td>5.8x10^-4</td>
<td>4.6x10^-7</td>
<td>5.8x10^-7</td>
</tr>
<tr>
<td>Void ratio</td>
<td></td>
<td>e_0</td>
<td>[-]</td>
<td>1.981</td>
<td>0.6</td>
<td>0.725</td>
<td>0.633</td>
</tr>
</tbody>
</table>

### 2.2 Stakeholders

Building a large dam will influence the surroundings. The dam will be built to prevent flooding in HCMC, but it will cause both positive and negative side effects. Thinking about the side effects of the dam and trying to come up with a good solution for all stakeholders will increase the chance the project will be executed.

In this section the most important stakeholders in this project are identified and described. Most of the text about the stakeholders is cited from the master thesis of Olga Sturm (Sturm, 2011). These stakeholders will be used in the multi criteria analysis for the trajectory of the dam in section 3.1.

#### 2.2.1 Ho Chi Minh City

One of the reasons HCMC is frequently flooded is the way the urban drainage system is designed; there is an open connection between the channels and the rivers. Due to this open connection the water levels in the channels are directly affected by the water levels in the rivers. If the water levels in the rivers increase due to high tide, storm surges or upstream river discharges affect the water levels in the open channels which may cause flooding.

In 2008 the ‘master plan of water level control for flood protection in HCMC area’ was approved. The emphasis of this plan is on controlling the water levels in the open channels independent of those in the rivers. According to this Master Plan this can be done by constructing 12 tidal drainage gates, embankments, retention basins and supporting pumping stations.

For this study it is assumed that the problems related to the drainage capacity of the urban drainage system are solved and the open connection between the channels and the rivers are intact. The effect of the Master Plan on the probability of flooding is not taken into account. It is assumed that there will be no
flood problems in HCMC as long as the water levels in the river at all locations in the city do not exceed MSL +1.0 m.

2.2.2 Dong Thap Muoi region

The Dong Thap Muoi region is a very important agricultural area for the production of rice in the South of Vietnam. The land is irrigated by an irrigation system which is fed by the Vam Co Tay and the Vam Co Dong rivers. The river discharges are in general very low, except for the rainy season when the discharges can be high for a short period. Due to these usually low river discharges the salt concentrations in the Vam Co rivers are high (up to 15 g Cl per litre), causing problems for the farmers. These problems can lasts for months and occur every year during the dry season. This is a problem for rice growth, as in the first growing stages the Cl concentration should not exceed 3 to 3.5 g/l.

The agricultural industry of the Dong Thap Muoi region has an interest in reducing the salt intrusion in the Vam Co Tay and Vam Co Dong.

2.2.3 Can Gio mangrove forest

The Can Gio mangrove forest is located 40 km southeast of HCMC. This forest has been listed as a biosphere reserve by UNESCO and the site is an important wildlife sanctuary in Vietnam. Mangroves are found in tropical and subtropical tidal areas in estuaries and marine shorelines. In order to survive in this environment, a plant must tolerate broad ranges of salinity, temperature, and moisture, as well as a number of other key environmental factors.

Depending on the location and the type of dam, the natural tidal and salinity range could be changed, which could have an effect on the mangrove forest. It is hard to determine a quantitative requirement for the conservation of the mangrove forest; however it is clear that for the mangrove forest the natural tidal and salinity range should change as less as possible.

2.2.4 Port activities

At different locations around HCMC port activities are going on, and also at some places extensions of port activities are planned. The dimensions of vessels calling at terminals in the area vary with the location. Most ships (80%) sail on the Long Tau, which has a greater depth compared to the Soi Rap. For a detailed description of the current port activities, see chapter 3.4.

For the competiveness of the terminals it is important that a downstream dam will not increase the handling time significantly. It is of the stakeholders’ interest that delay due to the construction of a dam is being avoided. Queues should be avoided and therefore the capacity of the navigational opening should be sufficient.

2.2.5 Infrastructure

The main highway in Vietnam runs from Hanoi along the coast via HCMC towards the Mekong Delta. The traffic intensity has increased in the past years and traffic congestion is a serious problem in HCMC. The fact that the main highway of Vietnam goes through HCMC, with its congestion, causes delays for the traffic. If a new road on the structure is included in the design this can reduce the congestion problems in the city, caused by traffic from the highway.
2.2.6 Stakeholder overview

In the sections above the stakeholders are described with their interest in the project. Table 2.10 gives an overview of the different stakeholders. This is also visualised in a map of the project area in Figure 2.6.

It is clear though that some interests are conflicting and concessions will have to be made.

Table 2.10: Stakeholders, interest and effect on location (Sturm, 2011)

<table>
<thead>
<tr>
<th>Stakeholder</th>
<th>Interest</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ho Chi Minh City</td>
<td>Water level $h &lt; \text{MSL} + 1.0$ m</td>
</tr>
<tr>
<td>Dong Thap Muoi region</td>
<td>Reduce salt intrusion in Vam Co rivers</td>
</tr>
<tr>
<td>Can Gio mangrove forest</td>
<td>Unchanged natural tidal and salinity range in Can Gio</td>
</tr>
<tr>
<td>Port activities</td>
<td>No delay in navigation</td>
</tr>
<tr>
<td>Infrastructure</td>
<td>Improve connection between Vung Tau and Go Cong areas</td>
</tr>
</tbody>
</table>

Figure 2.6: Stakeholders interests (Sturm, 2011)

2.3 Terms and conditions

In this section the list of requirements for the dam will be defined. Starting point for this list is a list of the different functions of the dam. The following functions are identified:

- Main function
  - Prevention of flooding of the hinterland

- Side functions
  - Control of salinity levels in the hinterland
  - Transportation connection between Go Cong and Vung Tau
These functions lead to functional requirements of the dam. Besides these functional requirements, structural, environmental and shipping requirements are defined. Some of these follow from the interests of the stakeholders. The full list of requirements is listed in the following sections. Based on these requirements the dam is designed.

### 2.3.1 Functional requirements
- The construction should be able to keep the water level in HCMC below MSL +1.0 m for events (e.g. waves, high river discharges, storm surges) with a return period of 1 in 200 years during the lifetime of the dam
- The construction should include a 2x2 road which connects Go Cong and Vung Tau

### 2.3.2 Structural requirements
- The construction should have a lifetime of at least 50 years
- The construction should be able to withstand events (e.g. storm surges, waves) with a return period less than 1 in 200 years during the lifetime of the dam
- Amount of required maintenance must be minimised during the lifetime

### 2.3.3 Environmental requirements
- The Can Gio mangrove may not suffer from the consequences of the structure
- Salinity levels in the Dong Thap Muoi region should not be too high for agricultural purposes

### 2.3.4 Transportation requirements
- Shipping delay must be kept to a minimum
- The construction process may not cause excessive hindrance to shipping

### 2.4 Reference projects
Reference projects are studied to obtain knowledge and to look for possible solutions for a design of the VT-GC dam. Data is collected about dimensions, construction type, foundations, water and wave heights, side functions, et cetera. The following projects are investigated:

1. Afsluitdijk, The Netherlands
2. Eastern Scheldt Barrier, The Netherlands
3. Haringvliet Dam, The Netherlands
4. IHNC barrier New Orleans, USA
5. Saemangeum Sea Wall, South Korea
6. Gulf of Khablhat Dam, India
7. Feni River Closure Dam, Bangladesh
8. MOSE project, Italy
9. Ems barrier, Germany

A full investigation can be found in appendix A. Points of interest were the foundation, cross section of the dam, construction method (especially for the last gap), discharge facilities and ship locks. These are discussed below.
2.4.1 Foundation

The VT-GC dam will be located in an estuary. Although there is not much information present about the subsoil, it is for sure that there will be soft soil at the location of the VT-GC dam present in the top layers. In all reference projects the barrier is located in an estuary and therefore usually built on soft soil. Barriers are heavy so a foundation has to be made. Different reference projects show different possibilities. Also the methods for the foundation of sluices and locks differ from the dam foundation. Dams which are open most of the time have different foundations as well.

2.4.1.1 Dams with a solid body

The reference projects of the Afsluitdijk, IHNC, Saemangeum, Khambhat and Feni River Closure all have a solid body. At the Afsluitdijk, the soft soil is reinforced with osier mats and dump stones. In Saemangeum the bottom is protect with mat paving on sandy soil/mud. For the Khambhat barrier, the plan is to make the bottom protection by using geotextile bags filled with sand. Bamboo sticks and filter material is used for the bottom protection of the Feni River Closure Dam. The IHNC barrier consists of almost 1400 concrete piles which are 42 to 49.5 m long and have a diameter of 1.7 m, and are an integral part of the construction itself (Figure 2.7).

2.4.1.2 Temporally dams

Three other projects are only closed when high water is expected: the Eastern Scheldt barrier, the MOSE and Ems barrier. At the Eastern Scheldt barrier the soil was first excavated by dredging, and replaced by sand. This sand was densified up to a depth of 15 meters using vibration piles (Figure 2.8). On the sand, matrasses filled with sand and gravel were placed to prevent scour of the sand under the pillars. On these matrasses the concrete pillars were placed. In the MOSE barrier project, the gates are founded on concrete piles. The German Ems barrier is founded on piles.

![Figure 2.7: The IHNC barrier (Brown, July 2009)](image1)

![Figure 2.8: Schematisation of foundation works of the Eastern Scheldt barrier](image2)

2.4.1.3 Sluices

Regarding the foundation for sluices almost all reference projects are relevant. The sluice foundation consists mostly of piles and soil improvement and a big concrete layer on top of it.

![Figure 2.7: The IHNC barrier (Brown, July 2009)](image1)

![Figure 2.8: Schematisation of foundation works of the Eastern Scheldt barrier](image2)
2.4.2 Dam cross section

When looking at the cross section of the dam and used materials, there are different types of dams. The dams that only close when high tide is predicted have a very special and unique cross section. The rigid dams have a kind of similar design.

At the IHNC barrier, the dam consists of big concrete tubes.

The common design of an earth dam is given in Figure 2.9. On the sides there are two clay or rock dams which form the toes. The space in between is filled with sand, rock or clay. The outer part of the dam is protected from the waves by boulders or an asphalt protection. On the toes there is also a rock-protection. Clay is used to prevent the dam from washing away.

![Typical cross section of an earth dam](image.png)

Figure 2.9: Typical cross section of an earth dam

2.4.3 Construction method

2.4.3.1 Earth dams

Several earth dams have been studied. These involve the Afsluitdijk, the Saemangeum Sea Wall, the Khambhat dam and the Feni River Closure Dam.

In all earth dam projects, first the bottom is levelled and reinforced using matrasses to prevent erosion of the soil on the bottom. Usually first one or two small clay or rock dams are placed, and hereafter the space in-between is filled with sand, rock or clay. On top of the dam a protection layer is then fabricated.

2.4.3.2 Barriers

Barriers studied are the Eastern Scheldt Dam, the Haringvliet Dam, IHNC barrier, the MOSE project and the Ems barrier.

For the Eastern Scheldt Barrier, which is founded on a shallow foundation, first the soil had to be improved. This is done by replacing soft soils by sand and compaction of sand. The other four projects are founded on piles, which are driven first.

On these foundations mostly pre-casted elements are used, but sometimes a construction is built in situ (i.e. the locks in the Afsluitdijk). On these elements a layer of protection is placed, usually stones, and in the last step the doors are placed.

2.4.3.3 Construction time

The construction time differs a lot between the different reference projects. The IHNC barrier for example is constructed very quickly (1 year), which made it very expensive, while construction of the Afsluitdijk
took 12 years to complete. The construction time depends largely on the applied building method. Innovative techniques usually take longer than proven techniques.

2.4.3.4 Closure of the final gap
Several closure methods are used in the reference projects. Most of the reference projects involve one final gap to be closed last. Exception to this is the dam in the Feni River in Bangladesh, which is constructed very quickly over the whole length of the closure using enormous amounts of manpower.

Closing techniques in the other reference projects involve dropping blocks from a cable crane (Haringvliet Dam, Figure 2.10), using of gated caissons (Khambhat, Figure 2.11), using of large geotextile bags (Khambhat) and the use of (large) stones (Saemangeum; Afsluitdijk, Figure 2.12).

![Figure 2.10: Closure of final gap using a cable crane (Haringvliet Dam)](image1)

![Figure 2.11: Closure of final gap using gated caissons (Khambhat Dam)](image2)

![Figure 2.12: Closure of final gap using stone dumping (Afsluitdijk)](image3)

In all reference projects bottom protection was needed to prevent the scour of the bottom due to increases in flow velocities caused by the narrowing of the gap during dam construction.

2.4.4 Discharge facilities
To provide capacity for the discharge of water from the Saigon - Dong Nai delta, gates or discharge sluices have to be constructed. The advantage of gates is that free navigation remains possible. Gate constructions are in general more expensive and have higher maintenance and management costs than regular discharge sluices.

2.4.4.1 Gate constructions
Gate constructions that have been studied are: IHNC barrier in New Orleans, USA, the MOSE project in Venice, Italy and the Ems barrier in Germany. In these projects sector gates and a lift gate, oscillating buoyancy flap gates and (rotary) segment gates have been used respectively. The gates provide a barrier against storm surges and in Venice also against high tides.
The VT-GC dam has to close off the estuary during high tides. During these high tides the gates will be shut. In Venice ship locks are installed to decrease port delays when the gates are closed. Also for the VT-GC dam the gates close off relatively often compared to for instance the IHNC barrier or Ems project. A ship lock additional to the gates could help to minimise port delays.

When the gates are open salt water can flow into the estuary behind the gates. This prevents the water in the estuary from becoming fresh, which would have negative environmental impacts, especially for mangrove forests. The oscillating buoyancy flap gates in Venice even transmit water when they are closed. For the design of the dam location made by the Southern Institute for Water Resources Planning (SIWRP), with a bridge of 5 km in length between the dam and Vung Tau, inflow of salt water into the estuary is of lesser importance for the mangrove forest as salt water flows in under the bridge. The location of the VT-GC dam can be seen in Figure 2.16.

In the IHNC barrier sector gates and a lift gate have been included in the closed barrier design. Gates to allow for shipping and salt water to enter the estuary may also be included in the design for a closed dam from Vung Tau to Go Cong.

The Ems barrier gives an example of segment gates. These close off the Ems River in case of a storm surge. By use of different segments a relatively long distance (476 m for the Ems barrier) is closed off, without requiring a lot of space. When the gates are open a lot of water can be discharged at the same
time. For the main shipping channel a rotary segment gate is used, which has the positive effect of having no height limit for navigation in the main channel.

2.4.4.2 Discharge sluices
Closure dams that include discharge sluices are: the Afsluitdijk, the Eastern Scheldt Barrier and the Haringvliet Dam in the Netherlands, the Saemangeum Sea Wall in South Korea and the Gulf of Khambhat Dam in India.

The amount of water that is discharged by these sluices and their dimensions can be of use when designing discharge sluices for the VT-GC dam. Other features, for instance the bottom protection and movable gates of the Haringvliet dam discharge sluices (Figure 2.17), can be used as an example for the design of gates and foundations.

![Cross section of Haringvliet sluice at base of pier (Ferguson et al., 1970)](image)

For the construction of the Haringvliet Dam the construction works started with the construction of the discharge sluices. The sluices were opened during the closure of the final gap of the dam. This reduced the currents and with that the forces on the construction during closure. In this sequence of building discharge sluices turn out to be useful not only for discharging of water but also during closure of the final gap.

The Eastern Scheldt barrier is designed as an open barrier, to maintain sufficient salinity levels in the Eastern Scheldt. This concept can be useful in the design of the VT-GC dam.

2.4.5 Ship locks
Most of the studied reference projects include ship locks to let ships pass the barrier or dam. The size of these ships locks can be compared with the maximum ship size using these locks, to get an impression of required space that ship locks will require in the VT-GC dam.
3 TRAJECTORY AND ELEMENTS

In this chapter the trajectory of the dam will be determined. The chosen trajectory will be used in the design of the dam and the closure. Also the type and location of special structures in the dam, like discharge sluices, will be discussed.

3.1 Trajectory

The trajectory and lay-out of the dam are of importance for the design of the dam. The route will result in parameters for the subsoil, water depth and input for the general dimensions of the structures. Those parameters are necessary to make the design of the dam.

First, two different options for the general functioning and route of the dam will be evaluated and one lay-out option will be chosen. Second, more detailed routes will be generated taking the different aspects influenced by the route into account. An advice is given on what route would be the best. Third, a basic design of the lay-out of this detailed route and the structures in it will be given.

3.1.1 General route

Two main options are taken into account for the general route of the dam. One is a dam from Go Cong to Vung Tau, the other a dam from Go Cong to Can Thanh, with a bridge to connect the dam with Vung Tau for the road connection.

To be able to make the best decision it is necessary to do a small study to the benefits of both options for the different stakeholders and weigh these advantages to the relative costs. With this study a choice can be made for the best alternative. For the analysis of the stakeholders, see section 2.2.

The general routes of the ‘Go Cong – Vung Tau’ and ‘Go Cong – Can Thanh’ dams are schematized in Figure 3.1 and Figure 3.2.

![Figure 3.1: Vung Tau - Go Cong](image1)

![Figure 3.2: Go Cong - Can Thanh](image2)
3.1.1.1  Go Cong – Vung Tau

The length of the dam will be approximately 30 km, depending on the routing and the locations where the
dam connects with the land. A road connection between Go Cong and Vung Tau can be built on the dam.
This alternative will need ship locks for all the ships sailing in and out the Saigon - Dong Nai delta. There
have to be discharge sluices for the river discharges from the Soi Rap and the Dong Nai.

The way the discharge sluices function will influence the climate in the river basin and surrounding areas.
For this reason, the following two options are taken into consideration: a salt water basin and a fresh
water basin. How these possible measures will affect the natural tidal and salinity range in the Can Gio
mangrove forest and the Vam Co rivers is unknown at the moment.

For the salt water basin the discharge sluices will work both ways letting salt sea water in with high water
and discharging river water and sea water with low water at sea. For the fresh water basin the discharge
sluices will only let water flow out of the basin, so the basin becomes fresh.

3.1.1.2  Go Cong – Can Thanh

This general route consists of a dam with a length depending very much on the route of the dam. This
length can be larger or smaller than the length of the dam in the Go Cong – Vung Tau alternative. This
alternative needs to have ship locks for ships sailing to or from the Soi Rap River. On the Vung Tau side,
sea water flows in and out of the delta uncontrolled. Ships do not have to pass ship locks to enter or leave
the delta. Extra measures for a road connection to Vung Tau have to be made. A bridge has to be high
enough so ships can sail under it. There will also be discharge sluices for the outflow of river discharges
and possibly the inlet of salt sea water, depending on the variant of the alternative: salt or fresh.

To prevent HCMC from flooding there has to be a flood defence structure downstream of HCMC in the
Dong Nai, and there have to be dikes built alongside the river to prevent flooding of other areas with high
water. The fresh water variant will need this dike to be on the West side of the Can Gio mangrove forest
to separate the fresh water basin and the natural estuary climate on the Dong Nai side of the river basin.

3.1.1.3  Construction costs

The differences in construction costs between the two general route options depend mainly on different
dimensions of the length of the dam, the locks and the discharge sluices. The building method, material
and profile of the different dams are the same and are therefore not taken into account.

<table>
<thead>
<tr>
<th>Cost aspect</th>
<th>Weighing factor</th>
<th>Go Cong – Vung Tau</th>
<th>Go Cong – Can Thanh</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of dam</td>
<td>31.44 M€/km</td>
<td>30 km</td>
<td>± 30 km</td>
</tr>
<tr>
<td>Ship lock(s)</td>
<td>Unknown</td>
<td>100%</td>
<td>20%</td>
</tr>
<tr>
<td>Discharge sluice</td>
<td>23,900 €/m³</td>
<td>5000 m²</td>
<td>2500 m²</td>
</tr>
<tr>
<td>Bridge</td>
<td>435 M€</td>
<td>None</td>
<td>5 km wide</td>
</tr>
</tbody>
</table>

From the comparison in Table 3.1 it is clear that the Go Cong – Vung Tau option has 5 times larger ship
locks and a double size discharge sluice than the Go Cong – Can Thanh option, where a high bridge with a
length of 5 kilometres has to be built. These rough estimates for the costs will be used in the multi criteria
analysis.
3.1.1.4  **Multi criteria analysis**

In this analysis the general route options are scored on how well they fulfil the demands of the stakeholders. These scores are then compared with the relative costs. Based on this analysis the decision for the best alternative is made.

*Table 3.2: Multi criteria analysis*

<table>
<thead>
<tr>
<th></th>
<th>Go Cong – Vung Tau</th>
<th>Go Cong – Canh Thanh</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>salt</td>
<td>fresh</td>
</tr>
<tr>
<td>HCMC: Prevent flooding</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Vam Co: Reduce salt intrusion</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>Can Gio: keep salty</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Navigation: no delay</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Infrastructure: Road connection</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Total score</td>
<td>3.45</td>
<td>3.0</td>
</tr>
</tbody>
</table>

The scores for the routes have to be divided by the costs. The alternative with the highest benefit/costs ratio is the best solution.

Taking 100% costs for the Vung Tau - Go Cong dam, the total costs of the Go Cong – Can Thanh dam can be approximately 113% and it will be the best option. The Go Cong – Can Thanh option is probably more expensive, but it is assumed not 1.13 times as expensive as the Go Cong – Vung Tau option. This means that for the general route of the dam the Go Cong – Can Thanh route is the best option, with a management to let the salt water intrude the basin as much as possible.

Further investigation is needed into the cost differences of the options, with that information the decision for the Go Cong – Canh Thanh dam with salt water can be checked.

3.1.2  **Route design - Aspects**

To determine the best route for the dam, different aspects have to be taken into account. The choice of a route will influence among others the basin area and the amount of material needed to build the dam.

In this section the different aspects having influence on the route of the dam will be discussed, resulting in a number of alternatives for the route. This section will conclude with the presentation of the chosen alternative.

3.1.2.1  **Connection with land**

The construction of a dam will influence the areas where the connection with the land is made. During construction there will be need for a building site at both ends. In the final situation a road will connect the areas.

The specific locations in Go Cong, Vung Tau and Can Thanh are not determined yet. To determine the route, several options are taken into account for the connection in Vung Tau and Go Cong. For the connection in Can Thanh the eastern most corner nearest to Vung Tau will be used.
Go Cong connection

For the connection in Go Cong there are two main possibilities, chosen for the current lay-out of the infrastructure. For the Northern option, option 1, it is necessary to build a road to the connecting point of the dam (striped line in Figure 3.3). In the connection area of option 1 there is more space for a building site, opposed to the connection area in option 2, where there is much development. Although there is a lot of development near the connection area in option 2, there is some agricultural land that could be used as building site if the building method requires this. See the detail of option 2 (Figure 3.4).

Vung Tau

For the connection in Vung Tau there are several options, regarding possibilities to link up with the current infrastructure and optimal locations regarding the minimal length of the dam. These options are pointed out with red squares in Figure 3.5.

For options 1, 2 and 5, a new road has to be built to make the Vung Tau - Go Cong dam a higher level road. Although these connection points will diminish the total length of the dam, a road has to be constructed over, past or under the hills there.
Option 3 and 4 are connecting the dam and road to the reasonably dense populated area of Vung Tau. In this area there are roads to connect to, but these roads go through the city centre and have to be upgraded in order to be able to use them as a connection to the dam. Due to the densely populated area, there is not much space for a construction site.

Connection option 6 lies more to the south than is probably necessary for the minimum required basin area, but is geographically a logical place to build the dam to. A connection to a high level road in the eastern direction can be made easily.

![Figure 3.5: Vung Tau connection possibilities (Google, 2010)](image)

**Can Thanh**

For the connection to Can Thanh in the Can Gio mangrove forest area, there is only one connection point taken into account. The dam to Can Thanh will not be a road connection, so the location is not limited by current infrastructure. The exact location of the connection with Can Thanh will be determined depending on what is the best location for the quantity of the basin area. The approximate location of this point is shown in Figure 3.6.
3.1.2.2 Basin area

The route of the dam will influence the created basin area on the land side of the dam. With a larger basin area, larger peak discharges of rivers can be collected, preventing flooding due to high river discharges. With a smaller basin however, the closure of the final gap of the dam will be easier, because the velocities in the gap will stay smaller.

The minimum required basin area to collect the peak river discharges with a return period of 200 years ($Q = 10250 \text{ m}^3/\text{s}$ (1 week peak discharge of Soi Rap River)), depends on the dimensions of the discharge sluice, see Table 3.3.

<table>
<thead>
<tr>
<th>$A_c$ (Cross section area of discharge sluice) [m$^2$]</th>
<th>$S$ (Storage area basin) [ha]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5000</td>
<td>10,000</td>
</tr>
<tr>
<td>2500</td>
<td>20,000</td>
</tr>
</tbody>
</table>

In order not to make the closing of the gap harder than necessary and to save construction material, it is best not to make the basin area larger than necessary for the designed river discharge.

3.1.2.3 Amount of material

The route of the dam will have a lot of influence on the amount of material needed for the construction of the dam. The length of the chosen dam, together with the bottom depth and the amount of subsidence due to the subsoil affects the amount of material necessary for construction.

Not much is known about the subsoil in the area where the dam will be constructed, which makes it impossible to make a fact-based decision on where to build or not to build the dam.

There is some information about the depth of the Holocene layers in a part of the area where the dam could be constructed, see Figure 3.7. There is a ‘well’ visible North West of Vung Tau with Holocene layers up to a depth of 30 m. A Holocene layer consists of fine sediments, constructing the dam on a thick Holocene layer will be unfavourable for the settlements and bearing capacity. Therefore these thick Holocene layers must be avoided.
The alternative with the shortest length and smallest water depths, avoiding areas with known bad subsoil will be preferred.

3.1.2.4 Morphological effects
This study does not focus on the morphological effects of the dam. The morphological effects of the different routes will not be taken into account for the choice of the route of the dam.

3.1.2.5 Currents
Closing a large area of an estuary will cause the current pattern of the tidal currents to change locally. Also closing the river discharge outlet (partially) will create locally smaller and far larger currents than in the current situation.

The route of the dam will influence the current pattern. The largest difference in current pattern exists between the straight dam and the hooked dam alternatives. For the hooked dam it is of importance that navigation through the undammed area near Vung Tau is able to pass the narrowest section without too high velocities.

3.1.3 Route design – Alternatives
With the above aspects in mind, different alternatives were developed. In this section the different alternatives will be introduced. In the next section the alternatives will be judged using a Multi Criteria Analysis and the best route will be chosen.

For all the alternatives several sub alternatives for the connection in Vung Tau are available. As this connection will be made with a bridge, and this project focuses on the design of the dam, the exact location of the bridge and connection to Vung Tau will not be looked in any further.
3.1.3.1 Alternative 1: North Straight

The Go Cong North alternative has a connection point north of Go Cong. A consequence of this northern connection point is that the basin area is small compared to the other alternatives. The dam will connect with Vung Tau in a straight line, crossing the first shipping lane at a narrow section and crossing shallow areas. This alternative therefore seems economic in its use of material.

Figure 3.8: Route alternative 1: North Straight

3.1.3.2 Alternative 2: South Straight

The South Straight alternative connects the Southern connection point of Go Cong to Vung Tau in a straight line, creating a large basin area. The route of the dam is not based upon the bathymetry: it crosses the first shipping lane at a wide section for example. There will be more material needed for this dam than for the first alternative.

Figure 3.9: Route alternative 2: North Straight

3.1.3.3 Alternative 3: South Curved

The South Curved alternative has the same connection point to Go Cong as the South Straight alternative, but the route of the dam is curved. The main reason for the curve in the route of the dam is that it will be more economic because less material is needed, as the dam will be built on shallower ground. The route of the dam is longer than the route of alternative 1, so it is not the most economic option, but it will be
more economic than alternative 2. Another difference with alternative 2 is the size of the basin area; the size of the basin area in this alternative is considerably lower than the size of the basin area in alternative 2. The size of the basin area of this alternative is larger than the basin area of alternative 1 though.

![Figure 3.10: Route alternative 3](image)

**3.1.4 Route design – Decision**

The most important criterion for the route of the dam is that it fulfils its main function: preventing HCMC from flooding. This criterion results in a requirement for a certain minimum basin area. This minimum value for the basin area is not known at this moment. In a later stage the minimum value of the basin area has to be put besides the values of the above alternatives. The alternative with a basin area just above the minimum will probably be the best option.

The approximate basin areas of the different alternatives have been determined by calculating the surface area of the basin with a computer program. This means the possible storage in the rest of the river delta is not taken into account. The results of the basin areas of the different alternatives are shown in Table 3.4.

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Basin Area (ha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative 1</td>
<td>16,600</td>
</tr>
<tr>
<td>Alternative 2</td>
<td>24,500</td>
</tr>
<tr>
<td>Alternative 3</td>
<td>20,000</td>
</tr>
</tbody>
</table>

These values are compared with the indicative values in Table 3.3 about the relation between the discharge capacity of the discharge sluices and the storage area of the basin. From the comparison the conclusion is drawn that all above alternatives are possible, with matching values for the cross sectional area of the discharge sluices.

The other criterion in which there is a difference between the alternatives is the costs. The differences in costs between these alternatives are based upon the differences in the amount of material necessary to construct the dam and the foundation needed.

It is difficult to draw a fact based conclusion without knowledge about the ideal basin area. The ultimate decision should be made in a later stage, when this information is known. In the rest of this study, a dam will be designed for the route of alternative 2.
3.2 Movable barrier versus discharge sluices

Construction of a dam between Vung Tau and Go Cong will block the navigation channel for ships. Ship locks will be applied to allow for ships to pass the dam. Discharge sluices are necessary to discharge the water that enters the estuary behind the dam from the Soi Rap River. Instead of discharge sluices, a movable barrier could be constructed that closes at high sea water levels, but allows for ships to navigate unhindered during normal conditions. In this section these solutions will be considered.

3.2.1 Movable barrier

The biggest advantage of a movable barrier is that it allows for continued use of waterways for navigation during normal conditions. Salt water can pass through the gates of the movable barrier both ways.

Special structures are relatively expensive. Especially structures with movable gates require a high capital investment. (Linham & Nicholls, 2010) found unit costs for storm surge movable barrier construction between US$ 0.7 and 3.5 million per unit meter width. Annual maintenance costs have been estimated at approximately 5 - 10% of the capital. A proper flood warning system is necessary that decides when to close the gates.

The possibilities for traffic on the dam are reduced when a movable barrier is constructed instead of discharge sluices. A road cannot continue in case of an open movable barrier. If a bridge is constructed over the movable barrier opening or gates, costs will be even higher and the height for navigation is reduced.

3.2.2 Discharge sluice

Navigation is not possible through the discharge sluices. Ships will have to use locks at all times. Salt water can pass through the gates of the discharge sluices both ways. Traffic is possible on top of the discharge sluices.

3.2.3 Navigation through narrow openings

The width of a barrier has a minimum for traffic purposes (ship size) and to make sure velocities won’t be too high for navigation.

To determine the minimum width for traffic navigation issues, reference projects are studied. The Maeslant barrier in the Netherlands has used the minimum width for traffic purposes for the ships passing the barrier, resulting in a width of 370 m. The Ems barrier in Germany has a passage for shipping with a width of 60 meter, but this passage is not often used for large ships.

The width might better be determined using simple guidelines, like in (Fisheries and Oceans, n.d.). With knowledge of the conditions coefficients for the width can be found, see Table 3.5 and Table 3.6. A large difference in the required width will be caused by choosing for 1 line of 2 lines (2-way) traffic, see Figure 3.11.
Figure 3.11: Channel width for one- and two-way traffic (Fisheries and Oceans, n.d.)

<table>
<thead>
<tr>
<th></th>
<th>One-way traffic</th>
<th>Two-way traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maneuvering Lane</td>
<td>$1.5B^4$</td>
<td></td>
</tr>
<tr>
<td>Ship Clearance</td>
<td>-</td>
<td>$1.0B + 0.2B^2$</td>
</tr>
<tr>
<td>Bank Clearance</td>
<td>$1.0B^3$</td>
<td></td>
</tr>
<tr>
<td>Wind and Current effects</td>
<td>$0.5B^4$</td>
<td></td>
</tr>
<tr>
<td>Depth/Draught ratio</td>
<td>$0.4B^5$</td>
<td></td>
</tr>
</tbody>
</table>

1 Maneuverability Coefficient for tankers
2 Ship Clearance + additional width requirement for moderate traffic density (1-3 vessel/hour)
3 Medium width requirement - severity
4 Combined value for wind and current effects on required width of channel
5 $D/d \leq 1.15$
The maximum velocity at which a vessel is still manoeuvrable is approximately 2 m/s, for currents in the length direction of the ship. From the previous sections a minimum cross-sectional area for the shipping passage follows: 20 meters depth and 188 or 304 meters wide, depending on whether there will be one- or two-way traffic. With this information the maximum current velocity in these passage can be calculated, using the storage area approach. For the explanation of the storage area approach see section 5.2. The outcome of these calculations are:

<table>
<thead>
<tr>
<th>One-way traffic</th>
<th>Two-way traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>$4.4B = 188 \text{ m}^2$</td>
<td>$7.1B = 304 \text{ m}^2$</td>
</tr>
</tbody>
</table>

From Table 3.7 it is clear that the maximum velocities in these passages with minimum dimensions are too large to make shipping possible.

The storage area approach was used again to estimate the velocities that occur at the location of the discharge sluices or gates for a certain discharge area. During an average tidal cycle the velocity can become 3 m/s for a discharge area of 20,000 m$^2$ (Figure 3.12). On most days velocities will exceed 2 m/s and shipping through the gates is not possible for a while. If the movable barrier is located between the Soi Rap navigation channel and the bridge to Vung Tau, where the average depth is MSL -5 m, the gates would need to have a length of 4 km. In the navigation channel, with a depth of MSL -8.5 m this would still be 2.4 km. A combination of a movable barrier and discharge sluices is possible. But also in that case the current velocities in the movable barrier will be too high for navigation during part of the day.

For a discharge area of 30,000 m$^3$ velocities remain below 2 m/s (Figure 3.13). In that case about 3.5 to 6 km of gates have to be constructed.

---

6 The width of the normative ship is $B = 42.8 \text{ m}$
3.2.4 Conclusion

Because discharge sluices and/or a movable barrier are very expensive elements it is advised to keep them as small as possible, but in small barriers velocities can get very high and this will hinder navigation. Construction and maintenance is more difficult for a movable barrier structure, compared to discharge sluices. Transport possibilities are better for discharge sluices.

Therefore it is advised to use discharge sluices and no movable barrier structure.

3.3 Location of different dam elements

The total layout of the dam is visualized in Figure 3.14. The trajectory of the dam is determined in section 3.1. An explanation for the chosen location of the locks, the discharge sluices and the final closure gap is given in the following sections.
3.3.1 Location navigation locks

For the construction and location of the navigation locks the following considerations are taken into account for the location of these structures:

1. The ship locks must be constructed in or near the main shipping channel, so that the existing navigation channel can be used when the dam is finished.
2. Shipping on the Soi Rap river must continue during construction of the dam

For these considerations the best location for the ship locks is next to the existing navigation channel. Then during construction shipping is possible. After construction dredging must be done to connect the ship locks to the existing channel.

3.3.2 Location discharge sluices

Consideration for the location of the discharge sluices:

1. The location of the discharge sluices will influence the flow pattern of the water and the movement of sediment
2. If the sluices are built in deeper water the required width is smaller for the same discharge area. Thus the width is minimised and the required amount of bottom protection smallest

The discharge sluices are best constructed next to the existing navigation channel. It is not expected that the discharge sluices have a large influence the flow velocities in the navigation channel, although further research on this topic will be needed in a later stage of the design, because it is important for shipping.

3.3.3 Location closure gap

For the location of the closure gap the following considerations are taken into account:

1. A closure gap located in deep water makes for the smallest width of the closure. Thus a greater dam length can be constructed by ‘standard’ construction methods, and less bottom protection is needed below the final closure gap
2. Free navigation through the channel is possible during the largest part of the construction of the dam.

For these reasons the final closure is best located in the navigation channel. When closing the gap the construction of the navigation locks should be finished, so shipping remains possible at all times.

3.4 Consequences for shipping

The construction of a dam in the mouth of the estuary of the Saigon-Dong Nai river basin will influence the shipping in the area. First the current situation and the planned flood defence structure are described, then the influence of a shipping lock on the delay of shipping and the preconditions for a potential gate following from shipping manoeuvrability.

A shipping lock is included in the design for ships sailing up or down the Soi Rap. This shipping lock will create delays for shipping. The time needed for the levelling of the water in the ship lock is calculated.

Although the use of gates is not advised, see section 3.2, the possibilities of ships sailing through gates in the dam is looked into. A section is devoted to the minimum width and maximum velocity for the manoeuvrability of ships and what this will mean for the design of gates, should they be used.
3.4.1 Current situation

Ms. Sturm (Sturm, 2011) made the following analysis of the current situation of the navigation in the area: at different locations around HCMC port activities are going on, also at some places port extensions are planned. The dimensions of vessels calling at terminals in the area vary with the location.

The largest vessels visit the terminal east of the Can Gio mangrove forest (100,000 DWT). The main approach channel for the HCMC port is the Long Tau (30000 DWT). Currently vessels up to 30,000 DWT can sail upon the Soi Rap, however there are plans to dredge the river in order to facilitate larger vessels to visit newly built terminals along the river (70,000 DWT).

Upstream of HCMC there are also some port activities, but those terminals are only visited by vessels smaller than 10,000 DWT. There are plans to dredge the Dong Nai in the future so that vessels up to 30,000 DWT can visit those terminals. Furthermore there are port activities along the Vam Co (10,000 – 20,000 DWT), Vam Co Dong (<5,000 DWT) and Vam Co Tay (< 5,000 DWT).

See Figure 3.15 for the locations of the port activities (Sturm, 2011).

![Figure 3.15: Location of port activities (Sturm, 2011)](image)

3.4.2 Planned flood defence structure

In Figure 3.16 the shipping routes are pictured. About 80 % of the ships is taking the eastern route to a harbour east of the mangrove. The total number of ships sailing ‘in and out’ of the basin area is unknown.

It can be seen that when a VT-GC dam is build the ships has to cross this dam. The interest of the shipping industry is to have no/only a small delay due to the VT-GC dam.
3.4.3 Delay due to ship locks

This section is about the time it takes a ship to pass the ship locks. The main component taking time is the filling or emptying of the ship lock to adapt to the demanded water level, called the levelling time. Another component is the time it takes a ship to sail in a ship lock and moor, moor and sail away. This last component is not taken into account in this study because it is not significant compared to the levelling time.

It is expressed clearly that the waiting time is not taken into account in this study. The waiting time will have a significant part in the total time it will take a ship to pass the ship locks, so this aspect needs sorting out in another study. The waiting time is not taken into account here because too much information is missing about the distribution of ships who want to pass the lock.

3.4.3.1 Levelling time

The main component taking time is the emptying or filling of the basin, this is called the levelling time. The levelling time depends on: the lock dimensions, the water level difference and the maximum allowed flow velocity with respect to the towed ships.

The choice in levelling times is an economic choice in which a comparative assessment is made between the interests of navigation, which would benefit from a shorter time, and the investments in the intake and discharging systems for which it is true that shorter times could lead to an expensive system. The following levelling times are usual in the Netherlands:

8 to 10 minutes for inland navigation locks and small sea locks with gate openings and a rise/fall of 2 to 6 m. For extra-long locks or a larger rise/fall, this could increase to 12 minutes; a smaller rise/fall requires less than 8 minutes. For lock culverts with an energy dissipating chamber (rise/fall between 6 and 12 m) the filling time is about 10 minutes; 11 to 15 minutes for large sea locks with a rise/fall between 1.5 and 5 m. (Ministry of Transport, Public Works and Water Management (NL), 2000)

Lock dimensions

The dimensions of the ships that will use the ship lock are varying from 20,000 DWT to 100,000 DWT. The Dead Weight Tonnage of the ship leads to the general dimensions of the ship depending on the type of ship: Container or Cargo, see Table 3.8 and Table 3.9. The general dimensions of the lock will have to be large enough to contain at least one ship of 100,000 DWT, the normative vessel for the lock. The dimensions on the lock should also depend on the intensity of navigation volume (navigation supply and pattern) and the fleet composition (Ministry of Transport, Public Works and Water Management (NL), 2000). Those aspects are not taken into account in this study.

Table 3.8: General dimensions container ships (NILIM, 2009)

<table>
<thead>
<tr>
<th>DWT (t)</th>
<th>Type</th>
<th>Length Overall (m)</th>
<th>Breadth (m)</th>
<th>Full Load Draft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20,000</td>
<td>Under Panamax</td>
<td>177</td>
<td>27</td>
<td>10</td>
</tr>
</tbody>
</table>
### Table 3.9: General dimensions cargo ships (NILIM, 2009)

<table>
<thead>
<tr>
<th>DWT (t)</th>
<th>Type</th>
<th>Length Overall (m)</th>
<th>Breadth (m)</th>
<th>Full Load Draft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50,000</td>
<td>Panamax</td>
<td>270</td>
<td>32.3</td>
<td>12.7</td>
</tr>
<tr>
<td>100,000</td>
<td>Super Large Container Ship</td>
<td>320</td>
<td>42.8</td>
<td>14.5</td>
</tr>
</tbody>
</table>

From the dimensions given in Table 3.8 and Table 3.9 the following dimension follow for the general dimensions of the ship lock:

### Table 3.10: Normative ship dimensions and estimated dimensions ship lock

<table>
<thead>
<tr>
<th></th>
<th>Length Overall (m)/ Length (m)</th>
<th>Breadth (m)/ Width (m)</th>
<th>Full Load Draft (m)/ depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normative Dimensions Ships</td>
<td>320</td>
<td>42.8</td>
<td>15.5</td>
</tr>
<tr>
<td>Estimated dimensions ship lock</td>
<td>400</td>
<td>50</td>
<td>20</td>
</tr>
</tbody>
</table>

### Water level difference

The water level difference at the Vung Tau - Go Cong dam will be the highest with (low) low water, as this creates the largest difference with the level in the basin. A (low) low water level will create a water level difference with the basin of approximately 1.5 meters. This low water level occurs only in a short period of time. With high water levels the water level difference is approximately 1 meter, see Figure 3.17. The water level difference (WLD) for the situation with low water at sea is therefore normative.

### Figure 3.17: Tidal elevation in Vung Tau and Water Level Differences (WLD) for Low and High water

#### Maximum allowed flow

Moored ships in the ship lock have to stay moored securely during the emptying or filling of the basin. Due to the flushing of water, flows could occur in the lock approaches that are bothersome or dangerous to navigation. For this reason, there has to be a maximum flow velocity in the ship lock.
In the Panama Canal Third set of locks, one of the design criteria is: “Limitation of average water velocities in culverts and ports to 7 m/s for all filling and emptying operating scenarios.” (Roumieu, 2008)

The maximum velocity also depends on the strength of the mooring facilities in the lock. The assumption of the maximum velocity in the ship lock is based upon horse sense, the design criteria used in the Third set of the locks in the Panama canal and the knowledge that a higher maximum flow velocity will bring about extra costs to build the intake and discharging system. The assumption is made that the maximum flow velocity in the lock openings can be 5 m/s at the maximum.

The area of the openings in the gate or others is assumed to be 1 % of the total surface area of the gates. With a full load draft of the normative ship of 15.5 meters and a water level difference of 2 meter, a depth of 20 meters or bigger for the sluice complex is probably on the small side, so the 1 % value for the filling doors will be on the conservative side.

**Calculation of levelling time**

To calculate the levelling time, the values from the sections above will be used. It is clear that there have been made some assumptions and that the outcome of the calculation will be an indicative value.

The water volume that has to go through the filling doors:

\[ V = \Delta h \times W \times L \]  

(3.1)

The maximum discharge through the filling doors:

\[ Q_{max} = v_{max} \times A_{fill} \]  

(3.2)

The levelling time for filling the basin will be:

\[ t_{fill} = \frac{V}{Q_{max}} \]  

(3.3)

<table>
<thead>
<tr>
<th>( \Delta h )</th>
<th>2</th>
<th>m</th>
<th>water level difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( W )</td>
<td>50</td>
<td>m</td>
<td>width of ship lock</td>
</tr>
<tr>
<td>( L )</td>
<td>400</td>
<td>m</td>
<td>length of ship lock</td>
</tr>
<tr>
<td>( V )</td>
<td>( 40 \times 10^3 )</td>
<td>m(^3)</td>
<td>volume of water through ship locks</td>
</tr>
<tr>
<td>( v_{max} )</td>
<td>5</td>
<td>m/s</td>
<td>maximum velocity in ship lock</td>
</tr>
<tr>
<td>( A_{fill} )</td>
<td>10</td>
<td>m(^2)</td>
<td>surface area of filling doors</td>
</tr>
<tr>
<td>( Q_{max} )</td>
<td>50</td>
<td>m(^3)/s</td>
<td>maximum discharge</td>
</tr>
<tr>
<td>( t_{fill} )</td>
<td>800</td>
<td>s</td>
<td>filling time</td>
</tr>
</tbody>
</table>

These calculations lead to a filling time of 800 seconds (= 13 minutes). A filling time of 13 minutes seems reasonable. This value is in the same order of magnitude as the value of the Dutch Ministry of Transport, Public Works and Water Management. The Dutch Ministry of Transport, Public Works and Water Management estimate a value of 11 to 15 minutes for large sea locks with a rise/fall between 1.5 and 5.0 m (Ministry of Transport, Public Works and Water Management (NL), 2000).
4 Design of the dam

In this chapter a preliminary design of the cross section will be made. This will include a calculation of the required crest level, the foundation, the materials that can be applied and some drawings.

4.1 Crest level

The height of the crest is an important parameter of a dam. The crest level can be deduced from several factors, some independent and some depending on each other. The crest level for the dam in this design has to be able to withstand conditions with a return period of 200 years. For this return period a design still water level and a design wave can be calculated. With the design wave the run-up and overtopping on the dam can be calculated. For determining the crest level also settlements and sea level rise will be taken into account.

4.1.1 Design life and level of protection

The design life time for flood defences in the Netherlands is 50 years. Structures in urban areas, where it is more difficult to upgrade a flood defence, are designed for a lifetime of 100 years. For very special structures with high capital costs like the Maeslantkering and the Eastern Scheldt storm surge barrier the design life is 200 years.

Risk levels (probability x consequence) that can be tolerated for a structure depend on local circumstances, local and national guidelines, the balance between risk and benefits, and the level of overall exposure. Lightly used areas may be designed to experience a certain level of hazard for a shorter return period than heavily trafficked areas. Example protection levels and return periods, that were suggested in (EurOttop, 2007) can be found in Table 4.1.

In Europe several national guidelines have recommended lower risk levels. In the United Kingdom, a medium probability of sea flooding between 0.5% and 0.1% (1:200 to 1:1000 year return period) was determined.

<table>
<thead>
<tr>
<th>Hazard type and reason</th>
<th>Design life (years)</th>
<th>Level of protection (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temporary or short term measures</td>
<td>1 - 20</td>
<td>5 - 50</td>
</tr>
<tr>
<td>Majority of coast protection or sea defence walls</td>
<td>30 - 70</td>
<td>50 - 100</td>
</tr>
<tr>
<td>Flood defences protecting large areas at risk</td>
<td>50 - 100</td>
<td>100 - 10,000</td>
</tr>
<tr>
<td>Special structure, high capital cost</td>
<td>200</td>
<td>Up to 10,000</td>
</tr>
<tr>
<td>Nuclear power stations etc.</td>
<td>-</td>
<td>10,000</td>
</tr>
</tbody>
</table>

For the VT-GC dam, maintenance will be relatively easy when it is constructed from sand or rock fill material. A design life time of 50 years is applied.

Not all failure modes will directly result in major damage. Overtopping water for example will discharge into the basin area behind the dam, while dam breach may cause more severe damage. The VT-GC dam is somewhere between a coast protection and a flood defence protecting large areas at risk. A flood risk with a 1:200 year return period is used for the design of the dam.
4.1.2 Design Still Water Level

The Still Water Level (SWL) is the elevation of the water surface (in the theoretical case) if wave action is absent. The design Still Water Level (design SWL) is the maximum SWL that can occur for the applied return period and within the lifetime of the structure. The Still Water Level (SWL) varies due to tides and storm conditions.

The tidal elevation can be predicted when the tidal constituents in the area are known. The tide depends on the movements of the sun, the moon and the earth. These movements can be predicted and therefore the tide can be predicted very accurately. Storm surges are caused by a low pressure weather system. Strong winds blowing over a shallow water body will cause an onshore rise of water. This storm surge increases the still water level.

In this section the design SWL for the dam is determined for conditions with a return period of 200 years. First, a literature study is done to find values for the design SWL used for another project in the area. Second, the available measurement data of the water level elevation is used to find the design SWL. Third, simplified calculations are done to calculate the design storm surge. Finally, a conclusion is drawn and a design Still Water Level is determined.

4.1.2.1 Design Still Water Level from literature study

A literature study is done to find out more about the design SWL and its components: the tidal component and the storm surge component.

Tidal component

The Hydrographic Office of the United Kingdom is a respected source of information for providing nautical maps and predicting tides. On nautical maps information is published about the tidal elevations in the area.

The nautical map of the Saigon River gives the following tidal elevation for Vung Tau: a Mean Higher High Water (MHHW) value of 1.0 m above Mean Sea Level (MSL) (UKHO, 1993). The value for Mean Higher High Water is used to calculate the design Still Water Level and not the Highest High Water level, as it is assumed that the chance of occurrence of a design storm and spring tide at the same time is relatively small.

The definition of MHHW is: “MHHW is the average of the elevations of the higher high waters of a mixed tide. A higher high water is the highest value of a tidal day. A tidal day in a semi-diurnal tide is the period of two low and two high waters. In a diurnal tide it is the period of a low and a high water and the higher high water is the only high water” (1972).

Storm surge component

The Vietnamese code advises a water level elevation of 1.5 m for storm surge for type 1 (primary defence) structures and 1.0 m for type 2 to 4 (secondary or lower defence) structures to calculate the design SWL. A return period is not used in this method.

Tidal and storm surge components

An American company of consulting engineers has done an analysis of the technical problems and approaches in the development of a port in Vung Tau in 1974. Although the report is more than 45 years old, the data about storm surges and tides are still relevant for the present situation.

Using the storm data from the Vung Tau Port Project, the (CIRIA, 2007) estimated the maximum storm surge for a return period of 40, 80 and 200 years. The value for the Mean Higher High Water (MHHW) was
estimated to be 1.0 m for the various return periods. In Table 4.2 the estimated maximum still water level above MSL can be found for different return periods.

*Table 4.2: Maximum Still Water Level Above MSL (CIRIA, 2007)*

<table>
<thead>
<tr>
<th>Return period (years)</th>
<th>40</th>
<th>80</th>
<th>200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm surge (m)</td>
<td>1.2</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>Astronomical Tide (MHHW) (m)</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Total SWL (m)</td>
<td>2.2</td>
<td>2.4</td>
<td>2.7</td>
</tr>
</tbody>
</table>

The return periods in this table represent the probability that a tide level of MHHW and a storm surge occur at the same time. The storm surge probabilities were based on the probability of a storm creating that surge. From an analysis of astronomical tide tables over several years it was concluded that the greatest frequency of high tides coincides with the late fall and winter seasons. The highest frequency of typhoons hit the coast from October to December. These two components partially coincide. This table therefore represents conditions during the fall of a typical year and may therefore be considered on the conservative side.

Vietnamese experts used water level data along the entire coast of Vietnam and Monte Carlo simulations including typhoon data to generate a graph of the water levels near Vung Tau with their chance of occurrence, see Figure 4.1. With this graph a design still water level of MSL +2.8 m was found for a return period of 200 years.

*Figure 4.1: Water level and frequency of occurrence per year in Vung Tau*

**4.1.2.2 Design Still Water Level from measurement data**

With hourly measurement data of the water level in Vung Tau from 1980 to 2007, a study was done to determine the design Still Water Level for a return period of 200 years. Included in the water level measurements are tidal elevation and the set-up due to storms, because the mean water level is measured.
A frequency study of the highest water levels shows a design still water level of MSL +1.6 m for the return period of 200 years. The study focused on the occurrence and height of peak values. First the peaks were isolated from the data and put in groups of 5 cm, ranging from 0 to 150 cm. A graph was plotted for the exceedance frequency of a peak over a certain level. This graph was extrapolated for a 1 in 200 years exceedance frequency. This resulted in a water level of MSL +1.6 m. This is illustrated in Figure 4.2.

Figure 4.2: Extrapolation of measurement values to an exceedance frequency of 1/200 per year

A design SWL of MSL +1.6 m is low compared to the value of the Lyon Associates, who worked with a design SWL of MSL +2.7 m for the same return period. It is therefore assumed that the measurement data did not administrate large storm surges. The following example illustrates this.

The highest water level (MSL +1.46 m) in the measured period of 28 years was compared with the predicted water level (MSL +1.25 m) due to tidal elevation only on the same hour. The difference between the predicted water level and the measured water level (0.21 m) was probably caused by a storm surge. The highest recorded water level only exists for 0.21 m of storm surge. This amount of storm surge is really small, compared to the value of 1.7 m of the Lyons Associates.

It can be concluded that a design SWL of MSL +1.6 m, extrapolated from measurement data, is not a reliable value for the design SWL.

4.1.2.3 Design Storm surge from calculation

The storm surge component of the design SWL can be determined using calculations. The input parameters include among others the wind velocity and the fetch (effective distance over which this wind blows) for the design circumstances.

In this section the principal of storm surge is explained, the parameters are determined, and several schematisations and corresponding calculations are made.
**Storm surge: Theory**

The definition of storm surge by (1972) is: “A rise above normal water level on the open coast due to the action of wind stress on the water surface.”

In shallow seas, deltas, closed off creeks and lakes, wind fields can influence the water level quite considerably by damming up the water (wind set-up). Figure 4.3 shows a model to approximate the wind set-up.

![Diagram of wind set-up](image)

*Figure 4.3: Balance of forces in case of wind set-up (Vrijling, revision 2011)*

The height of the water level due to storm surge can be calculated using a simplified calculation of wind blowing over a fetch of water and creating wind set-up.

The wind set-up in the equilibrium state is approximated by:

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

(4.1)

For a schematized situation with a constant depth, the formula can be rewritten to:

\[
S = C_2 \frac{u^2}{gd} \cdot x_1
\]  

(4.2)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>(S)</td>
<td>m</td>
<td>total wind set-up</td>
</tr>
<tr>
<td>(C_2)</td>
<td>-</td>
<td>constant</td>
</tr>
<tr>
<td>(u)</td>
<td>m/s</td>
<td>wind velocity</td>
</tr>
<tr>
<td>(g)</td>
<td>m/s²</td>
<td>gravitational acceleration</td>
</tr>
<tr>
<td>(d)</td>
<td>m</td>
<td>water depth (function of (x) or constant)</td>
</tr>
<tr>
<td>(x_1)</td>
<td>m</td>
<td>fetch length</td>
</tr>
</tbody>
</table>

The formula shows that the wind set-up increases with increasing wind velocity and fetch and decreasing water depth.

In a closed basin or a lake, the total amount of water cannot change. This means that, provided the slope may be assumed constant, the surface of the water (by approximation) will tilt around the gravity line of the basin surface, perpendicular to the wind direction. The water in the area away from the wind is subjected to wind set-down, see Figure 4.4. (Vrijling, revision 2011)
Storm surge: Parameters
The parameters of the storm surge formula mentioned in equation (4.1) are determined in this section.

Wind velocity
The wind velocity of a storm with a return period of 200 years can be found through extrapolating the data from the Vung Tau Port Project (CIRIA, 2007), see Figure 4.5. A value of 42 m/s for the wind speed is found.

Fetch length
The stretch of water over which the maximum wind velocity (see section ‘wind velocity’) blows is taken as the fetch length. The first estimation of the maximum length of the area where the maximum wind speed occurs is taken as 100 km, a conservative estimate. This value is compared with the measured values for the fetch length of typhoon Linda. The measured values of the wind speed and the radius where the maximum wind speed takes place are pictured in Figure 4.6. The maximum measured radius for this typhoon is 57 km.
The wind direction in a typhoon moves around the so called ‘eye’, therefore to obtain the fetch length, the typhoon is schematized as can be seen in Figure 4.7. The fetch length is chosen in a way that the vector of the wind speed has a larger onshore than alongshore component. From this schematisation, a fetch length of $R \sqrt{2}$ (radius times the square root of 2) is found. In practice, this means that the maximum measured fetch length of the Linda typhoon was approximately 80 km (based on the measured radius of 57 km) and the radius of the assumed typhoon would be approximately 70 km (based on the assumed value of the fetch).

![Figure 4.6: Graph 'Typhoon Linda' - Measured wind speed and radius (Nguyen, n.d.)](image)

The assumed radius length together with the velocity (see section ‘wind velocity’) are compared with measured values of typhoons of the last 50 years in the Gulf of Thailand. This is illustrated in Figure 4.8, with the red dot representing the assumed design situation. From this graph it is clear that there is a large variety in the radius of the typhoons and not so much in the maximum velocity. From this graph no
conclusion can be drawn about a relation between the fetch length and the maximum velocity in a typical typhoon, because there is no logical relationship. This same conclusion is drawn in different studies: (e.g., (Mallen et al., 2005)) found that there is no simple correlation between the radius with maximum wind speed and other dynamical parameters (such as maximum wind speed, minimum central pressure, or environmental conditions such as sea surface temperature) within the realm of measured data.

![Historical typhoon characteristics](image)

*Figure 4.8: Historical typhoon characteristics (Vongvisessomjai, 2008) compared with assumed design situation*

The wind speed in a typhoon changes with the distance to the eye of the typhoon. So for the same typhoon there are several relations between the velocity and the distance to the eye (radius). The velocity is the largest near the eye of the typhoon, so for smaller velocities the radius is larger. A lower wind velocity will thus blow over a larger area and will result in a different wind set-up than the combination of the maximum wind velocity and the corresponding fetch. Depending on the relation between the wind speed and the distance from the typhoon centre, different combinations of wind speed and fetch could be used in calculating the storm surge to be sure that the governing parameters for the storm surge are used.

However, without a correlation between the maximum wind velocity and the radius of a typhoon (and therefore the fetch length) this is not possible, so the parameters of the maximum wind velocity and corresponding fetch length for the return period of 200 years will be used for the storm surge calculations.

*Depth: constant*

The situation in the area where the dam will be constructed can be schematized as a basin with an average depth of 20 m (conservative value), see Figure 4.9.
Figure 4.9: General bathymetry South China Sea (National Geographic Magazine)

Depth: profiles
To take the near shore topography into account, a schematisation of the near shore profile is made. The inclination of the shoreline varies greatly along the route of the dam. Roughly a distinction in two profiles can be made, see Figure 4.10.

Figure 4.10: Location of depth profiles (map from UKHO, 1993)
The profiles can be schematized as linear profiles with an inclination of 0.001 and 0.0004 or as block profiles. The two different schematized situations with inclined profiles and block profiles (in red) are schematised in Figure 4.11.

**Storm surge: Calculation for constant depth**

The easiest calculation for the wind set-up is for an area with a constant depth. This will however be an approximation because in reality the geometry near the coast will have a gradient and the depths will be smaller than the assumed constant depth of 20 meter.

The following formula for the storm surge for an area with a constant depth:

\[
S = C_2 \frac{u^2}{g \cdot d} \cdot x_1
\]

| \(C_2\) | \(3.5 \times 10^4\) | constant |
| \(u\)    | \(42\) m/s       | wind velocity |
| \(g\)    | \(10\) m/s²      | gravitational acceleration |
| \(d\)    | \(20\) m         | water depth (function of \(x\) or constant) |
| \(x_1\)  | \(100 \times 10^3\) m | fetch length |
| \(S\)    | \(3.09\) m       | total wind set-up |

The outcome of this calculation is a total storm surge of 3.09 m, which means a resulting wind set up of 1.5 m, see Figure 4.12.
Storm surge: Calculation for depth profiles

With the depth profiles given in section ‘depth: profiles’, the total set-up of the water level due to storm surge can be calculated. With the block profile schematisation the same methodology as for a total fetch with a constant depth can be used. For the sloping profile it is not possible to use the simplified formula, so an integral has to be solved.

Block profile

The total set up is the sum of the deep water set-up and the shallow water set-up. The equation for the set up in shallow water is the same as in deep water, except that the total storm surge is equal to the wind set up. An illustration of the process of wind setup in a schematized blocked depth profile near shore is given in Figure 4.13.

The calculation of the wind set up with this method can be found in Appendix C. The wind set up that follows from these calculations is higher than was calculated for the block profile.

Sloping profile

The approach for calculating the storm surge for a sloping profile is similar to the approach for the block profile. The total wind set-up is the wind set-up of section 1 plus the storm surge in section 2.

The calculation of the wind set-up with this method can be found in Appendix C. The wind set-up that follows from these calculations is higher than was calculated for the block profile.
Profile 1: 2.2 m
Profile 2: 3.1 m

4.1.2.4 Design still water level
With the knowledge of the sections above, a decision is made for the storm surge component and the tidal component of the design still water level.

Storm surge component
The results of the literature study and the different methods to calculate the height of the storm surge differ between 1.5 m and 3.1 m. This is a very wide range, with the higher end being twice as high as the lower end. This wide range is caused by the differences in approach (first a rough calculation, and after that taking the bed inclination into account) and different slopes (0.001 and 0.0004).

The differences between the block elevated profile and the inclined profile are small: 2.1 m and 2.2 m for the first profile; 2.9 m and 3.1 m for the second (less steep) profile.

It is clear that the storm surge is higher for the less steep profile, though it is not certain that the formula works well for shallow areas.

The calculated values are higher than the values from the literature study: 1.5 m for primary defence structures according to the Vietnamese code, and 1.7 m for the Vung Tau study with a return period of 200 years according to the Lyons Associates.

The dam is situated in a relatively shallow area, where large set-ups due to storms can develop. From the Vietnamese code and the study of the Lyons Associates it is unclear for what circumstances (and return period, for the Vietnamese code) the values are valid. It is therefore a storm surge of 2.2 meters for the whole dam is suggested, and further investigation into storm surges in shallow areas will be needed.

Tidal component
The tidal component of the design still water level is set to 1.0 m above Mean Sea Level, according to the values from the Admiralty Chart and the Lyons Associates report.

Total Design Still Water Level
The design still water level for the dam with a return period of 200 years is the storm surge component plus the tidal component, so equal to MSL +3.2 m.

This level is somewhat higher than the design level of the Lyons Associates (MSL +2.7 m) and the Vietnamese experts (MSL +2.8 m). The difference in design level is most likely caused by the differences in storm surge component. The quantity of wind set-up due to storm surge depends on uncertain factors like the fetch of the design storm.

For the design of the dam, a design still water level value of MSL +3.2 m will be used. More research into factors with a high uncertainty is advised.

4.1.3 Reference Level: Mean Sea Level
Engineers usually use Mean Sea Level (MSL) as their reference level. Any value referring to another reference point is converted to the MSL reference level.

The definition of MSL is: “The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings” (1972).
The average of the height of the water level measurements in Vung Tau is VRL -0.25 m (VRL = Vietnamese Reference Level). The Vietnamese Reference Level is equivalent to the mean sea level in the north of Vietnam. For Vung Tau, the translation between Vietnamese Reference Level (of the measurements) and Mean Sea Level is: MSL = VRL - 0.25 m.

4.1.4 Waves: Run up & overtopping

Waves have an important influence on the crest height of the dam. Wave run-up and overtopping can cause damage and instability to the dam. In this section the wave run-up height that is exceeded by 2% of the 1/200 year incoming waves at the toe of the structure is calculated. This run-up can be used to calculate overtopping volumes. In the past decade the design for safety assessment has been changed to allowable overtopping instead of wave run-up. It turned out that dams with a proper revetment can withstand certain overtopping volumes, so designing to prevent wave overtopping could be a waste of money.

4.1.4.1 Influence factors

The run-up height, amount and character of overtopping are determined by waves, wind and properties of the cross section of the dam. These properties are:

- Geometry of the structure
- Roughness of the slope
- Slope angle
- Angle of wave attack
- Berm
- Vertical wall on top of the dam

Wave characteristics that determine the run-up and overtopping are:

- Wave height
- Wave steepness

Wind effects may influence overtopping by increasing overtopping discharges up to 4 times (EurOtop, 2007).
4.1.4.2 Wave data

Simplified calculations

The design significant wave height ($H_{m0}$) and the design period ($T_{m-1.0}$) at the toe of the dam were determined using SwanOne software and boundary data from the Vietnamese design code (appendix D). In these calculations the water level was assumed at MSL +3.2m (design still water level), to simulate an extreme situation. For a 200 year return period the significant wave height at a distance of 110 km offshore of the dam ($H_{sig}$) is 10 m and the wave period at the same location ($T$) is 12 s. The results of the SwanOne computation are displayed in Table 4.3. A design significant wave height ($H_{m0}$) of 3.2 m was determined for calculations for the largest part of the dam, located between the two navigation channels, with a design wave period of 8 seconds. In the shallow area near Go Cong $H_{m0}$ is not much lower; in the deeper zone near the navigation channel a higher value for $H_{m0}$ can be expected.

<table>
<thead>
<tr>
<th>Location</th>
<th>Design significant wave height ($H_{m0}$ [m])</th>
<th>Design wave period ($T_{m-1.0}$ [s])</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow area near Go Cong</td>
<td>Around 3m</td>
<td>Around 6s</td>
</tr>
<tr>
<td>Near and in navigation channel</td>
<td>Around 4.5m</td>
<td>7.1 to 7.5s</td>
</tr>
<tr>
<td>Between the two navigation channels</td>
<td>2.8-3.5</td>
<td>7-8.5</td>
</tr>
</tbody>
</table>

A wave surge level, this mean a water level elevation due to wave action, of 0.25 m was found for the cross section between the two navigation channels.
Wave height, period and wave surge for calculation
A design significant wave height of 3.2 m was determined for the location of the dam, but multiple assumptions and estimates were made to obtain this value, for example for the bottom profile and wind data. More research to the design significant wave height is advised for future stages of the project.

A design wave period $T_{m1,0}$ of 8 s will be used for calculations, as well as a wave surge level of 0.25 m.

4.1.4.3 Wave Run-up, slope angle and berm width
The wave run-up height is defined in the overtopping manual (EurOtop, 2007) as the vertical difference between the design Still Water level and the highest point on a slope that is reached by water running up the slope (Figure 4.15).

In the Netherlands many dike heights have been designed for a wave run-up height which is exceeded by 2% of the incoming waves at the toe of the structure ($R_{u2%}$). If only 2% of the waves reaches the crest of a dike or embankment during design conditions, the crest and inner slope do not need special protection measures other than clay with grass.

The inclination of the outer slope and the length of the berm are factors that have a large influence on the run-up height. In this section the wave run-up level for different slope angles and berm lengths is calculated. Also the material that is used for the revetment on the outer slope determines the run-up height for a great deal. Asphalt and quarry stone revetments are considered in these calculations.

Using this run-up level, the optimum slope angle and berm width is determined for the design of the VT-GC dam.

Nowadays more dikes are designed to withstand a certain overtopping volume rather than have a crest height on the 2% wave-run-up level. This will be explained further in section 4.1.4.4. The wave run-up is the basic input for calculating overtopping volumes.

Calculation results
In appendix E the calculation method for the 2% wave run-up level is described. For slope angles of 1:3, 1:4, 1:5 and 1:6 and a berm length of 0 up to 20m in steps of 2m, the run-up level was calculated. This was
done for a slope with asphalt revetment as well as one with a quarry stone revetment. From these calculations follows that a more gentle slope and a longer berm reduce the wave run-up.

Because using more material results in higher costs, the volume of the dam per m$^3$ length is calculated for different slope angles and berm widths. From these calculations the optimum berm width and slope angle are determined for a dam with asphalt and a dam with quarry stone as outer slope revetment.

For an asphalt slope a slope angle of 1:4 gives the lowest volume per m$^3$ of the dam. The optimum berm length is 16 m. For further design these values for the slope and berm length are used. The calculated 2% wave-run-up level $R_{2\%}$ is 4.8 m.

For a slope with quarry stone revetment a slope of 1:3 gives the lowest volume per m$^3$. Because this steep slope angle has a big chance of being instable, and because steeper slopes result in larger stone sizes for the revetment, a slope of 1:4 is chosen for further design. The content of the dam per m$^3$ turn only slightly differ for a berm with a length of 12, 14 or 16 m. A lower dam results in lower settlements in the subsoil, because the load will be lower. Because concluded is that a longer berm results in a lower dam ($R_{2\%}$), a berm length of 16 m is chosen for the quarry stone slope. The calculated 2% wave-run-up level $R_{2\%}$ is 3.1 m.

4.1.4.4 Wave overtopping

Wave overtopping is the mean discharge of water per linear meter of width over the dam. The amount of overtopping that is tolerable depends on the protection of the dam crest and inner slope. Also the use of the dam (traffic, pedestrians) has influence on the tolerable overtopping discharge.

In this section first the tolerable amount of overtopping is determined for different situations. These estimates are based upon model tests and available measurements. After that a deterministic calculation method for the overtopping discharge is given. With this discharge an estimate for the necessary freeboard of the VT-GC dam is made.

Overtopping limits

From overtopping research turns out that it is not possible to give unambiguous or precise limits to tolerable overtopping for all conditions. The following guidelines are taken from the Wave Overtopping manual (EurOtop, 2007):

Pedestrians

A discharge limit of 0.1 l/s/m is recommended when pedestrians on the dam crest have a clear view of the sea and are not easily upset or frightened and/or dressed to tolerate getting wet. A wider walkway is applied in this situation. A further precautionary limit of $q = 0.03$ l/s/m might apply for unusual conditions where pedestrians have no clear view of incoming waves; may be easily upset or frightened or are not dressed to get wet. A narrow walkway or close proximity to a trip or fall hazard apply in this situation. Research studies have shown however that this limit is only applicable for the conditions above and that this discharge should not be used as a general limit.

In case pedestrians are allowed to walk on the VT-GC dam crest a situation somewhere in between the above mentioned cases holds as pedestrians have a clear view over the sea. When a storm is predicted people standing on the crest may expect to get wet. The walkway can be closed off when a storm is predicted.
Traffic
Vehicles will drive on the VT-GC dam and therefore stricter overtopping limits than in a situation without traffic on the dam are required. Possibilities to close the dam for (specific or all) traffic in case of a storm can be considered to increase the overtopping limits.

For vehicles two very different situations are considered and therefore the overtopping limits are widely spaced. A limit of 10 - 50 l/m/s holds for roads where wave overtopping generates pulsating flows at the roadway level. Drivers have to reduce their speed. A lower overtopping limit of 0.01 up to 0.05 l/m/s is derived from considering more impulsive flows, overtopping at some height above the roadway, with overtopping volumes being projected at speed and with some suddenness. These lower limits may be pessimistic as they are based on only few tests and site data.

The hazardous effect of overtopping water reduces with the distance from the defence line. A rule of thumb for the hazard effect of an overtopping discharge at a point $x$ meters back from the seawall crest (over a range of 5 - 25m) is given by: $q_{\text{effective}} = q_{\text{seawall}} / x$. When the road is located on the inner side of the VT-GC dam higher overtopping limits can be applied. The Enclosure Dam (Afsluitdijk) in the Netherlands is an example of a dam with the road on the inner side (Figure 4.16).

![Figure 4.16: Cross section Enclosure Dam (Afsluitdijk) in The Netherlands](image)

If the road on the VT-GC dam is located say 10 m away from the crest, an overtopping discharge of 0.1 up to 0.5 l/m/s is acceptable.

In case no vehicles or pedestrians are allowed on the dam the overtopping limits depend on the protection of the crest and inner slope:

- Well protected: 50 - 200 l/m/s
- Grass covered embankment of clay: 1 - 10 l/m/s
- Not protected: 0.1 l/m/s

The required freeboard heights (height of the crest above design SWL, see appendix F) will be calculated for the following overtopping limits:

- Pedestrians: 0.1 l/m/s
- Road with continuing traffic: 0.5 l/m/s
- Lower driving limit: 10 l/m/s
- Upper driving limit: 50 l/m/s

Calculation results
The calculation method used to determine the required freeboard height for the dam can be found in appendix F. The freeboard height is the elevation of the crest above design Still Water Level (Figure 4.15).
If the freeboard is lower, more overtopping will occur. The necessary freeboard height is calculated for a maximum overtopping discharge volume of 0.1, 0.5, 10 and 50 l/m/s. This calculation was done for a 1:4 slope and a berm length of 16 m (as was determined in the previous section), for a revetment with asphalt and a revetment with quarry stone. Results can be seen in Figure 4.17.

![Figure 4.17: Required freeboard for different overtopping discharge and different revetment type](image)

Allowing a larger overtopping discharge can reduce the necessary freeboard height significantly. The required freeboard height for quarry stone reduces with almost 50% compared to the 2% run-up level when an overtopping discharge of 50 l/m/s is allowed. For an asphalt revetment a reduction of even more than 50% is found.

For small values of the overtopping discharge the calculation is not very accurate. For an overtopping discharge of 0.1 and 0.5 l/m/s the same freeboard heights are calculated.

For the design of the dam a discharge of 50 l/m/s will be acceptable if the revetment on the inner slope is built to withstand this discharge and if it is allowed to close the road on the dam for traffic when much overtopping is expected, e.g. when a storm is predicted. Closure of the road for part of the time during storm conditions is considered worthwhile as the design crest height reduces significantly.

For a dam with an outer slope revetment with asphalt this results in a freeboard height of 2.4 m; for quarry stone the required freeboard height is 1.6 m.

**Further research**

In a further stage of the project probabilistic calculations can be made for overflow and overtopping to include the effect of uncertainties in all parameters, or to find optimum levels including wind, wave and surge statistics.

### 4.1.5 Sea Level Rise

Because the dam needs sufficient height during the full lifetime sea level rise is taken into account in the determination of the required crest height. Global sea level rise predictions vary between 0.18 and 1.00 m between now and 2100. Further research is needed to make more accurate predictions.
For the design of the VT-GC dam a sea level rise of 0.75 m in 2100 is used as input for the design. This is the common value that is used for the design of flood defences in Vietnam.

More information on sea level rise can be found in appendix G.

4.1.6 Settlemnets
The settlements of the dam can be divided into 2 parts:

1. The settlement of the dam body
2. The settlements of the subsoil

4.1.6.1 The settlement of the dam body
The dam will settle due to the self-weight of the soil. Because the soil in the core is freshly applied, settlements are to be expected. To calculate this settlement a rule of thumb is used for the settlements of a sand core dam. This rule states that the dam body will settle 5% of the height, although these settlements can be neglected when the sand core is sufficiently densified (Stichting Deltawerken online, 2004). Because the sand will most likely be bottom dumped or rainbowed it will be loosely packed, so a percentage of 5% is used in the determination of the crest height. When a dam height of 10 m is assumed the settlement will be 0.5 m.

4.1.6.2 The settlements of the subsoil
In section 4.2.1.3 the settlements of the subsoil are calculated. These lie in the order of magnitude of several meters. However, soil improvement is needed to increase the bearing capacity, and this influences the settlements as well. In section 4.2.3.2 for the three assumed soil profiles the settlements are calculated when soil replacement or preloading and drainage are applied before construction of the dam. Then the settlements of the subsoil lie in the range of 0.3 – 0.55 m in 50 years, depending on the profile and soil improvement method.

4.1.7 Total crest height
The total crest height above MSL can be calculated as:

\[ h_{\text{crest}} = \Delta h_{\text{SWL}} + \Delta h_{\text{SWLs}} + \Delta h_{\text{wo}} + \Delta h_{\text{ws}} + \Delta h_{\text{SLR}} + \Delta h_{\text{sub}} + \Delta h_{\text{sd}} \]

In the table below the parameters are explained and the calculated values are given:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Definition</th>
<th>Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_{\text{crest}} )</td>
<td>m</td>
<td>Crest height above MSL</td>
<td>7.6 for asphalt slope</td>
</tr>
<tr>
<td>( \Delta h_{\text{SWL}} )</td>
<td>m</td>
<td>Tidal elevation of the Still Water Level</td>
<td>1.0</td>
</tr>
<tr>
<td>( \Delta h_{\text{SWLs}} )</td>
<td>m</td>
<td>Storm surge elevation of the Still Water Level</td>
<td>2.2</td>
</tr>
<tr>
<td>( \Delta h_{\text{wo}} )</td>
<td>m</td>
<td>Necessary freeboard height for wave overtopping</td>
<td>2.4 for asphalt slope</td>
</tr>
<tr>
<td>( \Delta h_{\text{ws}} )</td>
<td>m</td>
<td>Wave surge</td>
<td>1.6 for quarry stone slope</td>
</tr>
<tr>
<td>( \Delta h_{\text{SLR}} )</td>
<td>m</td>
<td>Sea Level Rise</td>
<td>0.25</td>
</tr>
<tr>
<td>( \Delta h_{\text{sub}} )</td>
<td>m</td>
<td>Settlements in the subsoil</td>
<td>0.5</td>
</tr>
<tr>
<td>( \Delta h_{\text{sd}} )</td>
<td>m</td>
<td>Settlements in the dam body</td>
<td>0.5</td>
</tr>
</tbody>
</table>

For a dam section located between the two navigation channels the design crest height for an asphalt revetment on the outer slope is MSL +7.6 m, and for a quarry stone revetment on the outer slope the design crest height is MSL +6.8 m. In both situations the outer slope has a slope of 1:4 and the berm on
the outer slope has a length of 16 m. For a dam section near the navigation channel the required crest height is higher, in the shallower parts the crest can be lower as the wave action is less severe in those areas.

In Figure 4.18 the final outer slope profile of the dam is drawn for a dam with an asphalt revetment

![Diagram](image)

Figure 4.18: Determined outer slope profile for a dam with an asphalt revetment (figure not drawn on scale)

The berm will be constructed one meter above design SWL. The berm has most effect when it is located on design SWL. Settlements and Sea Level Rise will reduce the vertical distance between the design Still Water Level and the berm. Settlements in the subsoil and the dam body below the berm are about 0.75 m in the lifetime of the structure. Sea level rise is determined to be 0.75 m. Settlements are more severe when the dam has just been constructed. Therefore the location of the berm above SWL is chosen a little bit above the average value of the settlements.

### 4.2 Foundation

#### 4.2.1 Geotechnical calculations without soil improvement

##### 4.2.1.1 Bearing capacity

Bearing capacity is one of the important aspects in the foundation engineering: will the soil fail or will it keep its properties? First is looked whether the bearing capacity is large enough when the dam is built directly on the soil. The dam is therefore schematised as a strip load, for which the bearing capacity is calculated with the program D-Foundations. This program is based on the Dutch version of Eurocode 7 (EC7-NL), which is based on the formula of Brinch Hansen. Although the dam induces a more or less triangular force on the subsoil, to simplify the loading it is schematized as a uniform load.

The bearing capacity has to be checked for the undrained as well the drained situation. Undrained calculations are needed directly after placing the embankment, as the additional stresses are then carried by the water. After a while, the excess water pressure will dissipate, so a drained calculation has to be performed.

All calculations will be done for a cross section of the dam as given in Figure 4.19 with 1 m width. Used parameters for the three different soil types can be found in appendix B. An explanation of the method, hand calculation and computer calculations can be found in appendix H.
In Table 4.4 and Table 4.5 the calculated design values of the bearing capacity and loads with different water depths are given. In the undrained situation, the bearing capacity is not enough (Total design bearing capacity $P_d < \text{total design load in the cross section } F_d$). The drained situation has enough capacity. Also resistance against punch ($R_d$ punch) (a load is sinking through a non-cohesive layer) and squeeze ($R_d$) (a cohesive layer is pushed away between two non-cohesive layers) are checked. $F_{\text{pull}}$ is the occurring pull force due to horizontal forces, and when punch is taken into account, also new load due to punch calculated ($V_d$). In the design of the dam, punch and squeeze are not allowed.

**Table 4.4 Computer calculation results bearing capacity and load, profile 1: Weak Vung Tau Undrained**

<table>
<thead>
<tr>
<th>Bottom at [MSL m]</th>
<th>$P_d$ undrained</th>
<th>$F_d$</th>
<th>$R_d$ squeeze</th>
<th>$F_{\text{pull}}$ [kN]</th>
<th>Undrained check</th>
<th>Undrained squeeze check</th>
</tr>
</thead>
<tbody>
<tr>
<td>-2</td>
<td>3714 kN (37.4 kN/m)</td>
<td>8738 kN (87.8 kN/m)</td>
<td>6837 kN (68.7 kN/m)</td>
<td>361 kN</td>
<td>Fail</td>
<td>Fail</td>
</tr>
<tr>
<td>-5</td>
<td>4498 kN (37.4 kN/m)</td>
<td>12368 kN (102.6 kN/m)</td>
<td>9766 kN (81.0 kN/m)</td>
<td>437 kN</td>
<td>Fail</td>
<td>Fail</td>
</tr>
<tr>
<td>-10</td>
<td>5804 kN (37.4 kN/m)</td>
<td>19958 kN (128.3 kN/m)</td>
<td>17504 kN (112.6 kN/m)</td>
<td>564 kN</td>
<td>Fail</td>
<td>Fail</td>
</tr>
</tbody>
</table>

**Table 4.5 Computer calculation results bearing capacity and load, profile 1: Weak Vung Tau Drained**

<table>
<thead>
<tr>
<th>Bottom at [MSL m]</th>
<th>$P_d$ drained</th>
<th>$F_d$</th>
<th>$R_d$ punch</th>
<th>$V_d$ Punch</th>
<th>Drained check</th>
</tr>
</thead>
<tbody>
<tr>
<td>-2</td>
<td>320018 kN (3216 kN/m)</td>
<td>8738 kN (87.8 kN/m)</td>
<td>320018 kN (3216 kN/m)</td>
<td>8738 kN (87.8 kN/m)</td>
<td>Pass</td>
</tr>
<tr>
<td>-5</td>
<td>587104 kN (4872 kN/m)</td>
<td>12368 kN (102.6 kN/m)</td>
<td>587104 kN (4872 kN/m)</td>
<td>12368 kN (102.6 kN/m)</td>
<td>Pass</td>
</tr>
<tr>
<td>-10</td>
<td>1236484 kN (7952 kN/m)</td>
<td>19958 kN (128.3 kN/m)</td>
<td>1236484 kN (7952 kN/m)</td>
<td>19958 kN (128.3 kN/m)</td>
<td>Pass</td>
</tr>
</tbody>
</table>

For the profiles *Strong Vung Tau* and *Go Cong* the same calculations are done. The bearing capacity in the undrained situation is not enough in all profiles, except in the situation of the profile *Strong Vung Tau*, with the bottom on a depth of 10 m below MSL. Here, the undrained situation is sufficient because the dam is then founded on sand and not on the soft clay layer any more. In the drained situation, all profiles pass, see Table 4.6 and Table 4.7. Soil improvement is needed there for all profiles.
Table 4.6: Computer calculation results bearing capacity and load, profile 2: Strong Vung Tau

<table>
<thead>
<tr>
<th>Bottom at [MSL m]</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-2 m</td>
<td>8738 kN (87.8 kN/m)</td>
<td>3714 kN (37.4 kN/m)</td>
<td>Fail</td>
<td>374918 kN (3768 kN/m)</td>
</tr>
<tr>
<td>-5 m</td>
<td>12368 kN (102.6 kN/m)</td>
<td>4498 kN (37.4 kN/m)</td>
<td>Fail</td>
<td>665486 kN (5523 kN/m)</td>
</tr>
<tr>
<td>-10 m</td>
<td>19958 kN (128.3 kN/m)</td>
<td>5804 kN (37.4 kN/m)</td>
<td>Pass</td>
<td>1323342 kN (8510 kN/m)</td>
</tr>
</tbody>
</table>

Table 4.7 Computer calculation results bearing capacity and load, profile 3: Go Cong

<table>
<thead>
<tr>
<th>Bottom at [MSL m]</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-2</td>
<td>8738 kN (87.8 kN/m)</td>
<td>4472 kN (44.9 kN/m)</td>
<td>Fail</td>
<td>279415 kN (2808 kN/m)</td>
</tr>
<tr>
<td>-5</td>
<td>12368 kN (102.6 kN/m)</td>
<td>5415 kN (44.9 kN/m)</td>
<td>Fail</td>
<td>447561 kN (3714 kN/m)</td>
</tr>
<tr>
<td>-10</td>
<td>19958 kN (128.3 kN/m)</td>
<td>6988 kN (44.9 kN/m)</td>
<td>Fail</td>
<td>820182 kN (5274 kN/m)</td>
</tr>
</tbody>
</table>

4.2.1.2 Remarks on the hand and computer calculation of the bearing capacity

There are some remarks on the assumptions that are made for the calculations and check on the bearing capacity of the dam. The first remark is on the schematisation of the dam as a strip load, to be able to use Eurocode 7. Because of the size and the more or less triangular shape of the dam (and therefore not uniform force and a big depth of influence that has to be taken in account for the strength-calculation and soil improvement) more investigation is recommended for the final design. Furthermore, in the preliminary design the undrained and drained calculations are done. But when building such a big dam there is not only the undrained or only the drained situation, but it might be in between. Finally, in the performed calculations, extra weight of needed soil due to settlements is not taken into account. All remarks are explained in the appendix B.

4.2.1.3 Settlements

The assumed dam profile for settlement calculations is presented in Figure 4.20. The bottom is chosen at a level of MSL -5 m, because this is the characteristic depth for most of the trajectory of the dam.

![Figure 4.20: Dam profile for settlement calculation](image)

For the three soil profiles drawn up (Weak Vung Tau, Strong Vung Tau and Co Gong, see appendix B) settlement calculations are made with hand calculations as well as computer software (the computer
programme D-Settlement, developed by Deltares, is used for this). For both methods the Bjerrum settlement formula is used. See appendix I for a further explanation of the calculation method.

The results of these calculations are presented in Table 4.8. Figures of settlements against time and settlements against depth are given in appendix I. Settlements in the order of magnitude of several meters are likely to occur during the lifetime of the dam!

**Table 4.8: Settlement calculations for the three assumed soil profiles**

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Hand calculations</th>
<th>D-Settlement calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Estimated full consolidation time [years]</td>
<td>Settlement after 50 years [m]</td>
</tr>
<tr>
<td>weak Vung Tau</td>
<td>228</td>
<td>3.3</td>
</tr>
<tr>
<td>strong Vung Tau</td>
<td>410</td>
<td>2.3</td>
</tr>
<tr>
<td>Go Cong</td>
<td>198</td>
<td>2.6</td>
</tr>
</tbody>
</table>

Calculations are performed for the settlements, assuming that the dam is continuously maintained at the same height. The option ‘maintain height’ in the computer programme D-Settlement is used for this purpose. The increase of settlements is imposed by the extra settlements caused by the settlement compensation. This extra settlement has to be taken into account when calculating the required height necessary to compensate the settlements. Results are presented in Table 4.9. Maintaining the height leads to a maximum additional settlement of 0.9 m after infinite time.

**Table 4.9: Maintain height calculations for the settlements and increase relative to the calculations without maintaining the profile height**

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Settlement after 50 years [m]</th>
<th>Increase [m]</th>
<th>Settlement after 100 years [m]</th>
<th>Increase [m]</th>
<th>Infinite settlement [m]</th>
<th>Increase [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak Vung Tau</td>
<td>4.0</td>
<td>0.7</td>
<td>4.9</td>
<td>0.8</td>
<td>5.4</td>
<td>0.9</td>
</tr>
<tr>
<td>Strong Vung Tau</td>
<td>1.9</td>
<td>0.1</td>
<td>2.1</td>
<td>0.2</td>
<td>2.3</td>
<td>0.2</td>
</tr>
<tr>
<td>Go Cong</td>
<td>2.9</td>
<td>0.3</td>
<td>3.4</td>
<td>0.3</td>
<td>3.6</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Because the precise layering of the soil is unknown, a sensitivity study is performed to analyse the influence of both the sea bottom depth (which influences the dam height and thus the loading) and the depth of the stiff layers (which are modelled as infinitive stiff) on the settlements of the soft top layer. Parameters used for these soft top layers are the parameters from the Go Cong top layer mud.

A figure for the settlements after 50 years is given in Figure 4.22, settlements after infinite time in Figure 4.21 and the settlements reached after 50 years as a percentage of the settlements after infinite time in Figure 4.23.
Figure 4.21: Total settlements (after 2500 years) of the weak top layer as function of the sea bottom depth and the bottom depth of the soft soil layer

Figure 4.22: Settlements of the weak top layer as function of the sea bottom depth and the bottom depth of the soft soil layer after 50 years

Figure 4.23: Percentage of total consolidation reached after 50 years

It can be concluded that the thicker the layer, the more settlements occur, but the slower these settlements develop. This is in line with theory, which says that consolidation takes longer with increasing drainage lengths.

It is possible to accelerate the settlements. This is useful to decrease the total settlements during the lifetime of the dam and to increase the effective strength of the soil due to dissipation of excess pore
water pressures. With such methods before or during the construction process, the settlements are accelerated such that the residual settlement gets lower and the effective strength higher. This can be useful when continuous heightening of the dam is unwanted. The optional methods are described in section 4.2.2.

4.2.2 Soil improvement

Settlements, bearing capacity and squeezing have a direct relation: the strength of the soil is leading. To create enough bearing capacity for the dam and reduce the settlements the following methods can be used.

4.2.2.1 Piles

At the location of the VT-GC dam the soil consist of a soft layer with a stronger layer beneath it. This stronger layer can be used for the foundation of the dam: piles can be placed in the soil. The load of the construction will be carried by the pile to the stronger layer. Therefore also fewer settlements are expected.

Foundation piles are available in various lengths, shapes and materials (wood, concrete and steel). It is possible to place piles from surface level and in the water. The number of piles needed for a foundation depends on a lot of variables. The most important one is the condition of the soil, as it determines the shaft friction and the tip resistance.

For the VT-GC dam, when the dam will be founded on piles a lot of piles are needed. Roughly the soil consists of 20 meter soft materials and under that fine to medium sand, loose to medium compacted, can be found. The soft layers will settle, and therefore causing negative friction on the piles: the pile will be loaded by the soil and settle (Figure 4.24). The tip of the pile has to be placed in a soil layer with sufficient bearing capacity to carry not only the weight of the dam but also the forces caused by the negative friction. Due to the big settlements this negative shaft friction is expected to be very high. The sand layer is not very strong, this means that only to overcome the force due to the negative friction the pile has to be very deep in the sand layer to generate (positive) shaft friction and point resistance. Only when there is enough bearing capacity to overcome the negative shaft friction and the force due the dam, the pile will not settle.

4.2.2.2 Soil improvement

To strengthen the soil and to reduce settlements, it is possible to compact the soil or to mix it with chemicals or grout, or to make columns of rock or sand. Figure 4.25 shows different types of soil improvement by mixing the soil. An explanation and relevance for the VT-GC dam is given below. Also compaction is added in the list of possibilities.
Figure 4.25: Possible application of several injection techniques (Van Tol & Everts, 2009)

- Permeation Grouting: pores in the soil are filled by injecting grout. This method can be applied in different kinds of soils, but will work the best for soils with rough structures (high permeability). By filling the holes with the injection fluid, a lower permeability is created. Also the bearing capacity is increased and settlements will decrease. At the location of the VT-GC dam the soil that has to be strengthened is clay, which has fine particles and a low permeability. Therefore this method might be difficult to use.

- Jet Grouting; a jet is bored in the soil. When taking the bore out, grout is jetted under high pressure into the soil. Due to the high pressures, which have to be different in different soil conditions, the soil/grout/water mixture is pumped away. The jet-grout method gives a soil with a high bearing capacity, and decreases settlement.

- Fracturing; create fractures in the soil by applying large amounts of injection fluid (grout) under a high pressure. When the horizontal effective stresses are larger than the vertical effective stresses, the fluid will form horizontal lenses in the soil. The soil then fractures horizontally. This is an uncontrolled process. Fracturing is usually used for reconstruction. The difference between fracturing and permeation grouting is that the aim is not to infiltrate the grain skeleton but to
push the soil aside and create higher stresses. Application of this method will be difficult for the dam because of the high pressures that are needed.

- **Compaction Grouting;** the soil is compacted and stressed due to injections with grout, which displaces the soil. This method is often used in coarse soils to uplift the soil due to settlements (reconstruction or precaution). For the VT-GC dam this might be a difficult method to use.

- **Mechanical Mix in Place;** mixing the soil and adding some grout, to create a ‘soil-concrete’ column. The strength is comparable with permeation grouting or jet grouting. Soil mixing is used to control settlements and to stabilize critical slopes. It is not clear if this method of soil strengthening works well for soils that mainly consist of clay. Investigation has to be done.

- **Compaction:** to create more strength by compaction is only possible when the soil consists of loose material. It is only possible for coarse material, and since the mud is a mix of mostly fine particles, this method cannot be used for the top layer on the VT-GC location.

### 4.2.2.3 Soil replacement

It might be the case that the soil is too bad to improve by means of mixing. Another way to make the soil more suitable to found a structure on is by replacing the soft soil (all or partly) by other materials.

- **Partly replacement can be done when making sand or stone columns (e.g. in a pattern of 2 x 2 m).** First a steel tube is put in the soil, then the mud is removed and finally the column is filled up with material. This creates bearing capacity like piles do, but can also work as vertical drain to speed up the settlements. For the dam, this can be an option, but it might be very expensive.

- **Fully replacing by sand (Figure 4.28).** Replacement material can be sand which has to be densified after placement to create sufficient bearing capacity. There is sand available in the neighbourhood of the project location. While sand might also be used for the core of the dam and therefore a lot of sand is needed in total, this might be a cheap and relatively easy solution to create bearing capacity and also to reduce the total settlements.

- **Fully replacing, partly by lightweight materials like EPS and partly by sand.** Installing EPS in sea is very difficult and light weight material can be very expensive, so this method is not favourable for the VT-GC dam.

*Figure 4.28: Soil improvement by fully replacing by sand*

The use of sand columns is used often in Asian and Pacific Rim region to increase the strength of the soil (measured by SPT blow count). (Bowles, 1997) The use of sand columns is mostly a trial and error-experience. Sand columns are usually drilled at diameters between 600 and 800 mm, with depths usually ranging from 3 to 8 m. There are calculation methods available, but the real check for the sand column diameter and spacing has to be tested with an SPT before and after installing some columns.

Stone columns are less economical than sand columns, but can also be applied in soft cohesive soils. The granular material has usually a gradation of 6 to 40 mm, a diameter ranging from 600 to 800 mm and a length of 5 to 8 m. The centre to centre distance is usually 1.2 to 3 m. The maximum length of a rock
column depends on the rock material and the surrounding soil and has to be tested in practice. It is assumed that all the load of the construction will be taken by the rock columns.

Due to the thick soft layer and the amount of material needed, sand/stone columns are not the most favourite option to strengthen the soil.

4.2.2.4  

Forcing acceleration of the consolidation process

Various methods are available to accelerate the consolidation process. This could be useful to accelerate the settlements and therefore increase the effective strength of the soil due to dissipation of excess pore water pressures. With such methods before or during the construction process the settlements are accelerated such that the residual settlement gets lower. This can be useful when continuous heightening of the dam (and therefore replacing the revetment) is unwanted. Also the soil is stronger. Several methods are:

- **Preloading**
  Before construction the subsoil is loaded by a load higher than the load induced by the final structure. Because of the higher weight the effective stresses in the soil are higher and therefore the consolidation accelerates. This extra load is removed when the final structure is constructed. This method is not suitable, because of the difficulties when placing this load, the low undrained bearing capacity of the subsoil and the low effectiveness (effective weight of soil under water is lower).

- **The use of drains**
  Vertical or horizontal drains can be implemented in the subsoil. Thus the maximum distance the water has to ‘travel’ (the drainage length) decreases and the consolidation speed increases. The basic is that a pipe is driven into the soil, and the space in the pipe is replaced by sand. Then the pipe is withdrawn.

- **Vacuum consolidation**
  A vacuum pressure is applied to lower the water pressure in the soil, thus increasing the effective pressure and therefore accelerating the consolidation process. This method is the most expensive one.

![Figure 4.29: Soil improvement by preloading and placing drains](image)

The use of vertical drains and preloading are the most useful to apply. For the use of vertical drains the following formula can be used (Barends, 2010):

\[
\frac{p}{p_0} = e^{-\frac{4c_v}{D^2}t}
\]

(4.3)

In which:

- \(p/p_0\) gives the consolidation phase, the part that already consolidated [-]
- \(c_v\) consolidation coefficient [\(m^2/s\)]
- \(D\) spacing between the wells [m]
Also preloading can be used, alone or in combination with drainage. In section 4.2.1.1 is calculated that the undrained bearing capacity of the weak top soils is too low to bear the weight of the dam.

From literature it is known that the undrained shear strength is related to the water content: the lower the water content the higher the undrained strength. Rule of thumb is that the undrained strength of a material with a water content equal to the plastic limit is 100 times larger than the undrained shear strength with a water content equal to the liquid limit (Davidson & Springman, 2000), so

$$c_{u,W=PL} = 100c_{u,W=LL}$$ (4.4)

By lowering the water content with preloading and/or drainage the undrained shear strength will be enough to ensure the bearing capacity in the undrained situation. It must be taken into account that this only increases the bearing capacity. Large settlements can still occur, this in contrast with soil replacement.

4.2.2.5 Conclusion

The options for constructing a good foundation for the dam are discussed. In a multi criteria analysis the best method for a foundation will be chosen. Only the reliable options are taken into account. Scores range from 1 (not favourable) to 5 (favourable). From Table 4.10 follows that replacement by sand seems to be the best option.

<table>
<thead>
<tr>
<th>Method</th>
<th>Weight factor</th>
<th>Piles</th>
<th>Mechanical Mix in Place</th>
<th>Partly replacing (stone or sand columns)</th>
<th>Fully replacing by sand</th>
<th>Drainage</th>
<th>Drainage and preloading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Costs material</td>
<td>15%</td>
<td>2</td>
<td>1</td>
<td>4</td>
<td>5</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Costs building the foundation</td>
<td>20%</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>5</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Building method</td>
<td>15%</td>
<td>3</td>
<td>2</td>
<td>3</td>
<td>5</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Reduce or speed up settlements</td>
<td>20%</td>
<td>4</td>
<td>2</td>
<td>4</td>
<td>3</td>
<td>5</td>
<td>1</td>
</tr>
<tr>
<td>Increases bearing capacity</td>
<td>30%</td>
<td>4</td>
<td>3</td>
<td>3</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Final score</td>
<td>100%</td>
<td>315</td>
<td>195</td>
<td>295</td>
<td>460</td>
<td>420</td>
<td>325</td>
</tr>
</tbody>
</table>

4.2.3 Geotechnical calculations with soil improvement

4.2.3.1 Bearing capacity

In almost all situations the undrained bearing capacity is not sufficient to bear the weight of the dam. Therefore the soil has to be improved. In section 4.2.2 different method are discussed for foundation/soil improving. The most favourable method for improving the bearing capacity and reduction of the settlements is replacement of soil by sand. Less favourable but also a good option is using vertical drainage with or without preloading. For the calculations the cross section of Figure 4.19 is used.

Profile 1, *Weak Vung Tau*, consist of a soft marine clay layer until a depth of 25 m below MSL. When replacing the clay by sand, a lot of sand is needed to increase the bearing capacity. Drainage in combination with pre-loading is schematized in D-Settlement and seems to be the best solution. In
section 4.2.1.1 is already determined that in the undrained phase the clay has a bearing capacity of 37.4 kN/m (per m width). This means that a preloading of 3.5 m sand (under water) is possible without failure of the soil. When drains are applied as well, the required consolidation time to create enough bearing capacity to carry the load of the full dam is decreased. In the section before it is explained that not the whole layer has to be consolidated to create enough bearing capacity, only minor drainage can be enough to create enough bearing capacity. The drainage distance not only depends on the waiting time to construct the whole dam, but also on the total settlement curve: in the lifetime of the dam only small settlements are allowed. Section ‘settlements’ (4.2.3.2) elaborates more on the drainage distance.

For the second profile, Strong Vung Tau, the problems with bearing capacity are much smaller: the soft marine clay layer is only present until MSL -8 m, and there is a strong clay layer between the sand layers. Less clay has to be replaced by sand, but the preloading method will also be a good solution. It takes only 5 days of preloading to create enough bearing capacity in the soil. Placing drains is therefore not needed. However, after constructing the dam the settlements will still be large, see 4.2.3.2, therefore replacement by sand will be favourable.

Profile 3, the Go Cong profile, has the same layer material sequence as Strong Vung Tau profile. The difference is that this middle clay layer is not as strong as the clay layer in Strong Vung Tau. When replacing the clay by sand, a lot of sand is needed, but the sand from the first sand layer can be re-used. Preloading with a load of 3.5 m sand will increase the bearing capacity in both layers, but then still a lot of settlement will occur during the lifetime. Therefore the foundation of this profile will combine the two methods: replacing sand in the upper soft layer and after that use preloading for consolidation of the lower clay layer.

An overview and conclusion of the possible methods is given in Table 4.11.

### Table 4.11 Overview and conclusion soil improvement

<table>
<thead>
<tr>
<th>Profile</th>
<th>Replacing sand [m³]</th>
<th>Drainage and preloading [days]</th>
<th>Combination</th>
<th>Best option</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak Vung Tau</td>
<td>9650</td>
<td>70 days year with D = 3 m and preloading</td>
<td>-</td>
<td>Drainage + preloading</td>
</tr>
<tr>
<td>Strong Vung Tau</td>
<td>1448</td>
<td>5 days with no drain</td>
<td>-</td>
<td>Sand</td>
</tr>
<tr>
<td>Go Cong</td>
<td>6273</td>
<td>Only preloading, mud layer = 5 days</td>
<td>3374 m³ of sand and preloading</td>
<td>Sand and preloading</td>
</tr>
</tbody>
</table>

4.2.3.2 **Settlements**

When soil improvement will be used, the settlements during the lifetime of the dam will decrease, either by accelerated settlements on beforehand, or by replacing the soft soil.

An estimation for the influence of replacing the top layer with incompressible permeable soil for the three assumed soil profiles is given in Figure 4.30.
One might see that especially replacing the upper few meters is favourable. Increasing the thickness of the soil improvement leads to decreasing efficiency for settlement reduction. Also only replacing the soft top layer is favourable, because this layer causes most of the settlements.

In section 4.2.3.1 the best way to improve the strength of the soil is calculated. In this chapter the influence on the settlements is determined. For the Strong Vung Tau and the Go Cong profiles the settlements after soil replacement are calculated, and for the Weak Vung Tau profile the required drain distance and preloading is determined.

Replacing the top weak layer(s) by a loose sand layer with a saturated volumetric weight of \( \gamma_s = 20 \text{ kN/m}^3 \), to increase the bearing capacity, and with a sea bottom level on MSL -5 m gives results as given in Table 4.12.

<table>
<thead>
<tr>
<th>Profile</th>
<th>Thickness of replaced soft layer [m]</th>
<th>Total settlements after 50 years [m]</th>
<th>Total settlements after infinite time [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vung Tau Strong</td>
<td>3.0 (until MSL -8 m)</td>
<td>0.49</td>
<td>0.79</td>
</tr>
<tr>
<td>Go Cong</td>
<td>13.0 (until MSL -12 m)</td>
<td>0.30</td>
<td>0.31</td>
</tr>
</tbody>
</table>

For several vertical drain distances and preloading times the effect on the settlements before and during the lifetime of the structure are calculated for the Weak Vung Tau profile in appendix I. A vertical drain centre-to-centre distance of 2 m is chosen, in combination with 3 years of preloading with 10 m of sand (sea bottom assumed on MSL -5 m). After 3 years, 3.81 m of settlements will be reached. Now the sand preload can be built out to the required dam profile, and during the following 50 years lifetime of the structure an additional settlement of approximately 0.55 m will occur. Thus settlements of the structure are minimised during the lifetime.

Thus for the three profiles, with either soil replacement or drainage and preloading the settlements during the lifetime of the dam can be minimised to 0.3 to 0.55 m. However, in these calculations the additional settlement caused by the extra material required to compensate for the settlements is not taken into account.
4.2.3.3 Stability

The stability of a construction in or on soft soil is very important in the design. When not taken into account, sliding might be a critical failure mechanism of a structure. Various methods are available to calculate the stability of slopes. Most of them assume a circular slip surface. The stability is expressed in a stability factor $F$ (strength/load). A lot of calculations for different possible slip circles are done, the smallest $F$ is the critical stability factor.

![Figure 4.31: Cross section dam profile for stability check](image)

With the computer program D-Geo Stability (based on the method of Bishop) the profile of Figure 4.31 is checked for stability. The soil profiles are adjusted with the required soil improvement (replacing the upper layer by sand for profiles Strong Vung Tau and Go Cong). The schematisation of the Weak Vung Tau profile is given in Figure 4.32. The phreatic line is at mean sea level and the tide influences only the first meters. This follows from the design inner berm, see section 4.3.4. Situations are checked for high water with storm surge and with sea level rise taken into account. The critical slip circles can be seen in Figure 4.33 and Figure 4.34.

![Figure 4.32: Input profile Weak Vung Tau in D-Geo Stability with low water level](image)
Due to the width and shape of the dam the two sides does not correlate with each other for the stability, therefore they are treated seperately. The critical sitation seems to be a low water level when the water in the dam is still high. The water in the dam can rise due to a long storm, rain and sea level rise.

In al situations, the inner slope is the most critical. This is due to the steepness of the slope. For profile Weak Vung Tau this slope is too steep, the inner slope has to be 1:4 instead of 1:3.5. Now a stability factor of 1.38 is calculated, which is sufficient.

The other two profiles do meet the requirements when soil improvement is taken into account. The (lowest) stability factors are 1.39 and 1.23 respectively. The full calculations are given in appendix J.

4.3 Cross section

4.3.1 Materials

For building the dam, the use of local material is a cheap and easy solution. On the location of the dam the subsoil consists mostly of soft soils: soft marine clay, loose and medium compacted sand and moderate clays (appendix B). For the dam, building materials like sand, rocks, good clay and a filter material is looked for. Because a lot of mud is assumed to be present on the project location, the possibilities for using this as a building material are studied in more detail.

An overview of the locations of the possible building materials is given in Figure 4.35.
4.3.1.1 Sand

Sand can be used as material in the core of the dam or as soil improvement for the foundation. So a lot of sand can be used in the design of the dam, which makes it an important building material.

In the Can Gio Mangrove area a couple of dredgers were observed at the Long Tau River (see Figure 4.36). The material that was obtained from the river seemed to be sand. In the local newspaper (Viet Nam News, 2009) it was found that sand can indeed be found in the Long Tau River.

In Figure 4.37 the location of sand in two cross sections are given. To the south-west of Vung Tau a sand layer can be seen. Also, the Vietnamese government has published the locations where sand may be dredged from sea (Figure 4.38). Finally, when looking to a sea map anchorages are signed. Here the subsoil must be strong enough for the anchor of the ships, so probably sand can be found here as well.
For all locations, parameters about the (quality of the) sand are not available. Because of the different sources there might be enough sand for building the dam.

![Figure 4.37: Location of sand (Southern Institute for Water Resources Planning, 2011)](image)

The sand used for the building of the dam (foundation and sand core) will be local sand. There is no information about the parameters of the sand, therefore some assumptions are made. For the sand specified in the Vung Tau profiles, the dredged sand is assumed to be medium dense. Curve C from Figure 4.39 is assumed for the sieve curve of this sand. An overview of the parameters of the ‘building sand’ is given in Table 4.13.
Figure 4.39 Grain size medium dense sand (curve C)

Table 4.13 Assumed parameters for local medium dense sand

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{\text{dry}}$</td>
<td>18 kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_{\text{wet}}$</td>
<td>20 kN/m$^3$</td>
</tr>
<tr>
<td>$c'$</td>
<td>0 kPa</td>
</tr>
<tr>
<td>$k$</td>
<td>$5 \times 10^{-4}$ m/s</td>
</tr>
<tr>
<td>$D_{15}$</td>
<td>0.15 mm</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>0.65 mm</td>
</tr>
<tr>
<td>$D_{85}$</td>
<td>0.60 mm</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>30 degrees</td>
</tr>
</tbody>
</table>

4.3.1.2 Rock

In the neighbourhood of Vung Tau there are some mountains (Figure 4.41). The mountains closest to Vung Tau are protected because of the nature, but mountain nr. 1 is already used as a stone quarry. Further away there are also mountains (2 and 3) with rocks which are not protected. The quality of the rocks is unknown. In Figure 4.40 it can be seen, that these mountains consist of “complex of mountain soils, yellow and red podzolic soils and lithosolic soils”. The quality of rock is not determined. Since there is already a stone quarry at mountain 1 this must be good rock relatively to the surroundings.

There is no information about the parameters of the rock, therefore the density is assumed to be 2600 kN/m$^3$ (Province of British Columbia, Ministry of Environment, Lands and Parks, 2000).
Figure 4.40: General Soil map (Moermann, n.d.)
4.3.1.3 Bamboo

In Figure 4.42 a geomorphological map of South Vietnam is given. From this the land-use can be observed. For the construction of a protection filter in the dam, bamboo or rush plant can be used. Bamboo (18, rùng tre nứa) can be found on a distance of 200 km of Ho Chi Minh City or in the area of Ta Lai. Rush plant (6, cỏi) is produced a lot, but more far away in the north of Vietnam.
4.3.1.4 Basalt

A material that is used a lot as dam protection is basalt. In Figure 4.40 the red/pink areas contain basalt. It is not clear on what depth the basalt can be found and what the quality is. Most regions where basalt is found are located further away than the potential rock quarries.

4.3.1.5 Mud

In this section the possibilities for the use of mud will be investigated. In the Saigon/Dong Nai river delta the top layers (±10 m) consist largely of this material, which makes it interesting for dam construction, because it is present in abundance near the construction area. Mud is categorized as a weak mixture of sands, silts, clays and organic material.

Two descriptions based on the grain size distribution, water content and Atterberg limits are summarized in Table 4.14.

<table>
<thead>
<tr>
<th>Location</th>
<th>Depth layer [m below surface]</th>
<th>Sand [%]</th>
<th>Silt [%]</th>
<th>Clay [%]</th>
<th>Liquid Limit (w_L) [%]</th>
<th>Plastic limit (w_p) [%]</th>
<th>Plasticity Index (PI) [%]</th>
<th>Permeability k [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thu Bo barrier data</td>
<td>0 – 9</td>
<td>16.4</td>
<td>30.4</td>
<td>53.2</td>
<td>61</td>
<td>31</td>
<td>30</td>
<td>7.6 \cdot 10^{-6}</td>
</tr>
<tr>
<td>Go Cong boring (LK 2000-1 188m)</td>
<td>3 - 12</td>
<td>17-41</td>
<td>23-48</td>
<td>20-63</td>
<td>?</td>
<td>?</td>
<td>?</td>
<td>?</td>
</tr>
</tbody>
</table>

In the Dutch construction guideline CUR report 162 'Construction with soils: soil constructions on and in very compressible soil with a low bearing capacity' (in Dutch: 'Construeren met grond: grondconstructies op en in weinig draagkrachtige en sterk samendrukbare ondergrond') (CUR, 2002) guidelines are given for the use of soft soils for construction works. A distinction into two categories is made:

1. Use of material as dam cover
2. Use of material as dam fill

Both aspects will be treated in more detail.

Use of material as dam cover

For dikes in The Netherlands the clay cover has to meet the requirements as given in Table 4.15. (CUR, 2002).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Outer slope and crest</th>
<th>Inner slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage lutum (grains &lt; 2 μm)</td>
<td>[%]</td>
<td>&gt; 20 &amp;</td>
<td>&gt; 15 &amp;</td>
</tr>
<tr>
<td>Percentage sand (grains &lt; 2 mm)</td>
<td>[%]</td>
<td>&lt; 40</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>Percentage organic material</td>
<td>[%]</td>
<td>&lt; 35</td>
<td>&lt; 40</td>
</tr>
<tr>
<td>Amounts of CaCO₃</td>
<td>[%]</td>
<td>&lt; 4</td>
<td>&lt; 4</td>
</tr>
<tr>
<td>Salinity (NaCl) in pore water</td>
<td>[gr/l]</td>
<td>&lt; 25</td>
<td>&lt; 25</td>
</tr>
<tr>
<td>Exchangeable Sodium Percentage (ESP)</td>
<td>[%]</td>
<td>&lt; 15</td>
<td>&lt; 4</td>
</tr>
<tr>
<td>Liquid limit (w_L)</td>
<td>[%]</td>
<td>&gt; 45</td>
<td>No requirements</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>[%]</td>
<td>&gt; 0.73 \cdot (w_L - 20%) &amp;</td>
<td>&gt; 18%</td>
</tr>
</tbody>
</table>
These criteria are checked for the Co Gong and Thu Bo mud in Table 4.16.

Table 4.16: Check of the CUR-requirements for Thu Bo and Go Cong soils. Red means criterion not met, green means criterion met. '?' means unknown

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Outer slope and crest Thu Bo</th>
<th>Inner slope Thu Bo</th>
<th>Outer slope and crest Co Gong</th>
<th>Inner slope Go Cong</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage lutum (grains &lt; 2 μm)</td>
<td>53.2</td>
<td>53.2</td>
<td>20 - 63</td>
<td>20 - 63</td>
</tr>
<tr>
<td>Percentage sand (grains &lt; 2 mm)</td>
<td>16.4</td>
<td>16.4</td>
<td>17 - 41</td>
<td>17 - 41</td>
</tr>
<tr>
<td>Percentage organic material</td>
<td>?</td>
<td>?</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>Amounts of CaCO₃</td>
<td>?</td>
<td>?</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>Salinity (NaCl) in pore water</td>
<td>32 g/l (sea water)</td>
<td>32 g/l (sea water)</td>
<td>32 g/l (sea water)</td>
<td>32 g/l (sea water)</td>
</tr>
<tr>
<td>Exchangeable Sodium</td>
<td>?</td>
<td>?</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>Liquid limit (w_L)</td>
<td>61</td>
<td>No requirements</td>
<td>?</td>
<td>No requirements</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>31</td>
<td>31 (&gt; 29.9)</td>
<td>?</td>
<td>?</td>
</tr>
</tbody>
</table>

Both types of mud do not meet these requirements, because of the saline sea water conditions and the large amount of lutum present. Therefore it can be concluded that mud is not suitable to use as dam cover material.

Use of material as dam fill
In The Netherlands clay is almost never used for constructive applications, only for dam cover material. Therefore no guidelines are present for the use of clay as fill material.

The use of the dredged alluvial deposits seems not feasible without the use of some sort of containment. The material has a low strength, especially under undrained loading conditions. With the use of containment it could be feasible however. One could think of:

1. Geotextiles
2. Rigid structures

In some countries clay dredging material is sometimes used as a fill material for dikes in combination with geotextiles. For example, for land reclamation projects along the coast of Tianjin (China) flat geotextile bags were filled with locally dredged clay slurry, see Figure 4.43 and Figure 4.44. The dredged slurry is immediately pumped into the bags, where it can consolidate.
Other techniques involving geotextiles are the use of geosynthetic tubes or big geosynthetic bags. Although courser material like sands or silts are more favourable as a fill material for geosynthetics, clay is also usable. Some points need special attention:

- Geosynthetic gap size: it must be small enough to prevent particles from escaping the bag, but big enough to allow water to pass to enable consolidation.
- Consolidation of the bags: the bags filled with dredged material need time to consolidate. The accompanying settlements need to be compensated.
- Atterberg limits: the dredged material should be easily processable (one must pump it into the bags).

Another possibility is the use of a rigid containment structure, like a caisson or sheet pile walls. Thus lateral movement of the material is restricted, and the bearing capacity of the soil is increased compared to the case where no lateral restraining is applied. However still attention has to be given to:

- Consolidation of the fill: consolidation will go slowly due to the impermeable structure. The excess pore pressure will decrease slowly, and thus the effective stress governing the strength will remain low for a long time. Special measures need to be taken therefore to increase the consolidation.
- Atterberg limits: the dredged material should be easily processable (one must be able to pump it).

**Conclusion**

It can be concluded that mud is not likely to be a suitable construction material when not contained, because of the low strength parameters. It could be useful however as a fill material when contained in bags or a rigid structure. The high water content makes it easily processable. Special care has to be given to the consolidation.

However, more research is needed on the behaviour and composition on the mud. Because this is not available at the moment, alternatives for the dam involving mud are not likely to be chosen because of the uncertainties and technical difficulties involved.
4.3.2 MCA of Different Alternatives

4.3.2.1 Introduction
To determine the best alternative for the cross section of the dam several alternatives are prepared and compared by means of a Multi Criteria Analysis (MCA). In this section the alternatives will be presented, the criteria by which they are compared will be explained and the analysis and results will be given.

4.3.2.2 Alternatives
Nineteen alternatives are sketched during brainstorm sessions. These profiles are based on:

1. Dam profiles encountered in reference projects (see section 2.4 or Appendix A)
2. Material that is used for the major part of the dam. These are divided into two classes:
   - Local materials
     - Local alluvial deposits Although weak abundantly present in the surroundings.
     - Sand Can presumably be dredged north of Vung Tau and offshore, although quantities are unsure.
   - Other materials
     - Rocks Could possibly be obtained from quarries near Bà Rịa. Amounts are unsure.
     - Steel Source unknown
     - Concrete Source unknown
     - Concrete blocks Source unknown
     - Rubber Source unknown

The alternatives are categorised by differences in fill material and concrete or steel works. Note that protection materials are not yet taken into account, as they are considered to be too detailed for this stage of the project.

In Table 4.17 a short description of the different alternatives is given. A schematic cross section of each alternative can be found in Figure 4.45.
### Table 4.17: Description of the alternatives for the cross section

<table>
<thead>
<tr>
<th>#</th>
<th>Category</th>
<th>Short description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Single fill material</td>
<td>Dam with sand core</td>
</tr>
<tr>
<td>2</td>
<td>Single fill material</td>
<td>Dam with rock core</td>
</tr>
<tr>
<td>3</td>
<td>Single fill material</td>
<td>Dam with core local alluvial deposits (e.g. in geosynthetic bags)</td>
</tr>
<tr>
<td>4</td>
<td>Single fill material</td>
<td>Dam with concrete block core</td>
</tr>
<tr>
<td>5</td>
<td>Multiple fill materials</td>
<td>Combined dam with rock and sand</td>
</tr>
<tr>
<td>6</td>
<td>Multiple fill materials</td>
<td>Combined dam with rock and local alluvial deposits (e.g. in geosynthetic bags)</td>
</tr>
<tr>
<td>7</td>
<td>Multiple fill materials</td>
<td>Combined dam of concrete blocks and sand</td>
</tr>
<tr>
<td>8</td>
<td>Concrete wall</td>
<td>Vertical concrete wall founded on piles</td>
</tr>
<tr>
<td>9</td>
<td>Concrete wall</td>
<td>Combined vertical concrete wall with sand</td>
</tr>
<tr>
<td>10</td>
<td>Caissons</td>
<td>Concrete caisson dam</td>
</tr>
<tr>
<td>11</td>
<td>Caissons</td>
<td>Concrete caisson dam, completely covered with sand</td>
</tr>
<tr>
<td>12</td>
<td>Concrete wall</td>
<td>Concrete caisson with open top filled with sand</td>
</tr>
<tr>
<td>13</td>
<td>Concrete wall</td>
<td>Concrete caisson with open top filled with local alluvial deposits</td>
</tr>
<tr>
<td>14</td>
<td>Concrete wall</td>
<td>Concrete caisson with open top, completely covered with sand</td>
</tr>
<tr>
<td>15</td>
<td>Sheet pile walls</td>
<td>Double sheet pile wall filled with sand</td>
</tr>
<tr>
<td>16</td>
<td>Sheet pile walls</td>
<td>Double sheet pile wall filled with local alluvial deposits</td>
</tr>
<tr>
<td>17</td>
<td>Sheet pile walls</td>
<td>Dam with sand core and sheet pile</td>
</tr>
<tr>
<td>18</td>
<td>Dunes</td>
<td>Sand beach and dune profile</td>
</tr>
<tr>
<td>19</td>
<td>Inflatable</td>
<td>Inflatable barrier</td>
</tr>
</tbody>
</table>

![Diagram](image)

**Figure 4.45**: Schematic cross sections of alternatives. The left side of each drawing is the seaside, the right side is the basin side.
4.3.2.3 Criteria

The criteria for the MCA are presented in Table 4.18. For each criterion an explanation and weight factor (measure of importance) is given.

The criterion ‘reliability’ is not taken into account in the MCA, because reliability of a dam depends more on the detailed design than on the concept of the alternative. Each alternative will have to be designed such that it will meet the predetermined reliability level. (e.g. a clay dam can be as reliable as a concrete dam, if the slopes are gentle and sufficient protection is used).

Table 4.18: Explanation of criteria used in the MCA

<table>
<thead>
<tr>
<th>#</th>
<th>Category</th>
<th>Criterion</th>
<th>Explanation</th>
<th>Weight %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Construction techniques &amp; application</td>
<td>Technical difficulty &amp; Experience</td>
<td>Takes into account the technical difficulty of the design, the experience with similar construction techniques and projects and the possibilities for quality management. Easier constructions where a lot of experience is gathered in similar projects result in higher scores.</td>
<td>20.0</td>
</tr>
<tr>
<td>2a</td>
<td>Building costs</td>
<td>Fill material (amount and local availability)</td>
<td>Takes into account the distance to the source of the material and also the amounts and costs. A closer source and cheaper material means a higher score.</td>
<td>12.5</td>
</tr>
<tr>
<td>2b</td>
<td>Building costs</td>
<td>Dam protection works</td>
<td>Takes into account the measures required to protect the fill material. More or more difficult protection means larger costs and failure chances. Less protection works required result in a higher score.</td>
<td>2.5</td>
</tr>
<tr>
<td>2c</td>
<td>Building costs</td>
<td>Foundation</td>
<td>Different cross sections need different foundations. E.g. a concrete dam needs a rigid foundation to prevent cracking, whereas a soil dam can be built on a more flexible foundation. Need of rigidity means more complicated foundation works. Less foundation works results in lower costs, thus in a higher score.</td>
<td>5.0</td>
</tr>
<tr>
<td>2d</td>
<td>Building costs</td>
<td>Equipment</td>
<td>Takes into account the amount and costs of equipment needed for construction. Less or less expensive equipment results in a higher score.</td>
<td>5.0</td>
</tr>
<tr>
<td>2e</td>
<td>Building costs</td>
<td>Environmental costs</td>
<td>The cost involving the compensation of loss of environmental value. Higher scores mean lower costs.</td>
<td>2.5</td>
</tr>
<tr>
<td>2f</td>
<td>Building costs</td>
<td>Rehabilitation costs</td>
<td>The cost required to acquire construction space and to compensate loss of housing and jobs. Lower costs result in higher scores</td>
<td>2.5</td>
</tr>
<tr>
<td>3</td>
<td>Operation &amp; Maintenance</td>
<td>Maintenance costs &amp; possibilities</td>
<td>The amount of maintenance needed during the lifetime and the ease of performing these reparations (e.g. soil dams are easy to maintain (just replace/refill), whereas for concrete and steel (corrosion) maintenance is more difficult). Options that are easier to maintain receive a higher score.</td>
<td>15.0</td>
</tr>
<tr>
<td>4</td>
<td>Traffic</td>
<td>Possibilities for traffic and future expansion</td>
<td>The possibilities for construction of a road on top of the structure and the possibility to expand the road in the future, will this be needed. Higher scores mean more possibilities for traffic.</td>
<td>15.0</td>
</tr>
<tr>
<td>5</td>
<td>Environment</td>
<td>Influences on nature</td>
<td>The influence of the construction of the environment (e.g. pollution, vegetation, landscape). Higher scores mean less negative impacts.</td>
<td>10.0</td>
</tr>
<tr>
<td>6</td>
<td>Aesthetics</td>
<td>Design</td>
<td>The aesthetic value of the design, and the possibilities to become an icon. Higher scores mean higher aesthetic value.</td>
<td>10.0</td>
</tr>
</tbody>
</table>
4.3.2.4 **MCA calculation**

Each alternative gets a score for each criterion. This score is expressed on a scale from 1 to 5, where ‘1’ stands for unfavourable outcomes (e.g. ‘expensive’ or ‘difficult’, whereas ‘5’ on the opposite stands for favourable outcomes (e.g. ‘cheap’ or ‘easy’).

The score for each criterion is multiplied by the weight factor of that criterion, and all these values are summed up for each alternative to find the total score. The higher the total score the better the alternative is.

Scores are given in Appendix K. The total score for each alternative is presented in Figure 4.46.

![Figure 4.46: MCA total score for each alternative](image)

4.3.2.5 **MCA Results**

The five best scoring alternatives from Figure 4.46 are, in descending order:

1. Alternative 1: Dam with sand core
2. Alternative 5: Dam with rock and sand
3. Alternative 2: Dam with rock core
4. Alternative 6: Combined dam with rock and local alluvial deposits
5. Alternative 4: Dam with concrete block core

The three best scoring alternatives will be elaborated further.

From the results it can also be concluded that the best categories of options are is descending order:

1. Single fill material / multiple fill material
2. Dunes
3. Sheet pile walls
4. Caissons
5. Inflatable dam
6. Concrete wall

The use of local weak alluvial deposits will also be investigated further. The usability of this material is uncertain, because not much is known about the quality. It can be very rewarding however, because of the abundance of this material in the vicinity of the dam.

4.3.3 **Three alternative cross sections**

In the previous section a MCA was made for the alternatives of the cross section for the VT-GC dam. The three alternatives which scored best will be elaborated in this section. This will include the following elements:
1. Cross section of the dam
2. General dimensions
3. Material
4. Foundation
5. Building method
6. Location

Hereby the following assumptions are made:

- Crest level dam: MSL +6 m
- Sea bottom: MSL -5 m (the average depth for the majority of the dam trajectory)

### 4.3.3.1 Dam with sand core

#### Cross section, general dimensions and materials

This dam cross section (Figure 4.47) consists of a sand core. Protection layers are applied to assure the stability. Application of an inner berm improves the macro stability of the dam. A road is located on this inner berm. The traffic on the road is less exposed to wind gusts and incidental wave overtopping compared with traffic on the crest of the dam. A berm on the outer slope can be applied to reduce wave run-up and has the strongest effect when located on the Still Water Level line. The outer slope will have an inclination around 1:4 or 1:5. The inner slope is steeper, because loadings on this side of the dam are less severe. Dimensions in the figure are indications.

**Foundation**

The construction will be founded on a shallow foundation. Because the bearing capacity of the current subsoil is probably not strong enough, soil improvement will be needed to strengthen the soil (e.g. soil replacement, grouting or soil mixing).

Also bottom protection will be needed to prevent scour of the subsoil in front of and behind the dam.

**Building method**

The construction method will consist of the following steps:

1. Soil improvement
2. Application of the bottom protection
3. Construction of the core and protection layers
4. Construction of the crest
5. Finishing (e.g. construction of the road)

For step 3, the construction of the core and protection layers, two work sequences can be followed. On the left side of Figure 4.48 the traditional method is shown, where the core material is applied first and the protection layers are applied afterward. The right side of Figure 4.48 shows the alternative method...
where protective bunds are provided. This method results in a better protection of the core material during construction. It requires however more protection material and the construction method is a little more complicated.

<table>
<thead>
<tr>
<th>Traditional method</th>
<th>Alternative method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Filters</td>
<td>a. Filters</td>
</tr>
<tr>
<td>2. Core</td>
<td>b. First under-layer (part)</td>
</tr>
<tr>
<td>3. First under-layer</td>
<td>c. Core</td>
</tr>
<tr>
<td>4. Armour layer</td>
<td>d. First under-layer (part)</td>
</tr>
<tr>
<td>5. Crest (working back)</td>
<td>e. Core</td>
</tr>
<tr>
<td></td>
<td>f. Armour layer (part)</td>
</tr>
<tr>
<td></td>
<td>g. First under-layer (part)</td>
</tr>
<tr>
<td></td>
<td>h. Armour layer (part)</td>
</tr>
<tr>
<td></td>
<td>i. Crest (working back)</td>
</tr>
</tbody>
</table>

Figure 4.48: Varying construction sequences (Verhagen et al., 2009)

**Location**

This cross section can be used for most of the length of the VT-GC dam. Current velocities at the final gap closure are probably too high for the application of sand. Therefore at the location of the final gap a cross section which consists of heavier material must be applied.

### 4.3.3.2 Dam with rock and sand

**Cross section, general dimensions and materials**

For this dam cross section (Figure 4.49) the core consists of both sand and rock. The part made of rock has a height just above high water and is located on the sea side of the dam. The size of the rock material depends on current velocities in the final gap. Protection layers and an inner and outer berm can be applied for the same reasons as for the sand dam (section 4.3.3.1). The outer slope will have an inclination of around 1:4 for the rock core part and around 1:4 or 1:5 for the sandy part. A lower slope angle decreases the wave run-up and therefore the required crest height. The inner slope is again steeper, because loadings on this side of the dam are less severe. Dimensions in the figure are indications.
Foundation
See ‘Dam with sand core’, section 4.3.3.1.

Building method
The construction method will consist of the following steps:

1. Soil improvement
2. Application of the bottom protection
3. Construction of the rock core
4. Construction of the sand core and protection layers
5. Construction of the crest
6. Finishing (e.g. construction of the road)

Location
This cross section is especially suitable for the location of the final gap. The rock core can be constructed while current velocities at the final gap closure are probably too high for the application of sand. When the final gap has been closed the rest of the dam can be finished with sand and protection material.

4.3.3.3 Dam with rock core

Cross section, general dimensions and materials

For this dam cross section (Figure 4.50) the core consists of rock. Protection layers and an inner and outer berm can be applied for the same reasons as for the sand dam (section 4.3.3.1). The diameter of the core rocks depends on current velocities in the final gap. The rock material that may be used for protection layers is coarser than that of the core. Rock has a higher angle of internal friction compared to sand. A steeper slope can be applied to create a stable profile. The outer slope will have an inclination of around 1:4. The inner slope is again steeper, because loadings on this side of the dam are less severe. Dimensions in the figure are indications.

Foundation
See ‘Dam with sand core’, section 4.3.3.1.

Building method
The construction method will consist of the following steps:

1. Soil improvement
2. Application of the bottom protection
3. Construction of the core and protection layers
4. Construction of the crest
5. Finishing (e.g. construction of the road)
Location
Because the slopes of this cross section are steeper compared to the slopes of the sand dam, a smaller amount of material is required for this dam than for a sand dam. Especially at the deeper parts the cost reduction for the smaller amount of material may be higher than the additional transport costs.

This cross section is also suitable for the location of the final gap. The rock core can be constructed while current velocities at the final gap closure are probably too high for the application of sand.

4.3.4 Design inner slope

4.3.4.1 Inner berm

In this section the design of the inner slope will be discussed. This largely depends on the use of an inner berm and the positioning of the road on the dam. The road can be placed on the crest of the dam, or on an inner berm, see Figure 4.51.

![Figure 4.51: Schematic view of the dam with and without inner berm, and the positioning of the road](image)

For the design of the dam it is chosen to construct the road on an inner berm. This has the following advantages:

1. The road is more sheltered from winds and wave overtopping, so it can be used in more severe weather conditions.
2. Construction of an inner berm is advantageous for the macro stability of the inner slope.
3. Construction of an inner berm makes the dam wider. Therefore the hydraulic gradient in the dam is smaller, so less water will flow through the dam and the risk of micro instability is lower.
4. Construction of an inner berm makes the dam wider. Therefore the risk of shearing (lateral displacement of the dam) is decreased.

There are however disadvantages involved. These are:

1. The asphalt of the road cannot be used as crest protection.
2. The total surface area of the dam increases, so more protection will be needed.

Depending on the dimensions of the dam the amount of material needed for an inner berm can be an advantage or a disadvantage. This can be estimated by calculating the difference in volume between the dam without an inner berm and a dam without one: \( \Delta V = w_r h_r - (w_r - w_c)(h_c - h_r) \) [m³/m'].

In the design it is chosen to construct a dam with an inner berm, because the advantages are considered more important than the disadvantages.

4.3.4.2 The level of the road

In this section the level of the road will be determined. A lower level means less material needed, and therefore lower construction costs. However, the road must have a minimum level, based on:
1. Traffic must be guaranteed, so the road must not flood due to the water level and waves on the inner side of the dam.
2. Water pressures below the road asphalt layer must be minimized to prevent burst.
3. The phreatic line must stay below the road. This way seepage in the dam is minimized.

The third aspect will be the strictest. Therefore this will be used in the calculation. On the basin side a maximum water level of MSL +1 m is assumed (maximum level in HCMC) and in the sea side a high average water of MSL +2.5 m, and a tidal amplitude of 1 m.

The influence length of the tide is equal to (Barends & Uffink, n.d.)

$$\lambda = \sqrt{kDT/\pi n} = \sqrt{5 \cdot 10^{-4} \cdot (5 + 2.5) \cdot 43200/\pi \cdot 0.4 = 11.4 \text{ m}}$$ (4.5)

Where:
- \(k\) = the permeability [m/s],
- \(D\) = the average water depth above sea bottom [m],
- \(T\) = the period of the wave [s] (assumed to be 12 hours)
- \(n\) = the porosity of the material [-]

The influence length of wind waves will be much shorter, as the period is much shorter, and will be neglected.

The maximum increase in phreatic height due to the tide is now given by:

$$\Delta h = H \cdot e^{-x/\lambda}$$ (4.6)

Where:
- \(H\) = the amplitude of the tide, = 1 [m]
- \(x\) = the distance from the outer slope [m]

This can be added up at the formula for the phreatic head in the dam, which is given by

$$h(x) = \sqrt{\frac{h_2^2 - h_1^2}{L} (L - x) + h_1^2}$$ (4.7)

Where:
- \(h(x)\) = the phreatic head at distance \(x\) from the outer slope [m]
- \(h_1\) = the phreatic head in the basin = 4 m (reference level is bottom level)
- \(h_2\) = the phreatic head at sea = 8 m (reference level is bottom level)
- \(L\) = the drainage length, in [m]

So the total phreatic height in the dam is given by

$$h(x) = \sqrt{\frac{h_2^2 - h_1^2}{L} (L - x) + h_1^2 + H \cdot e^{-x/\lambda}}$$ (4.8)

The result of this formula is given in Figure 4.52. Assumed dimensions can be observed in Figure 4.53.
When this graph is used to find the minimum level of the road level of MSL +2.1 m is obtained to prevent water seeping out of the inner side of the crest, next to the road. When a safety of 0.3 m is used the level of the inner berm and the road is chosen as MSL +2.4 m.

This level is supposed to be high enough to prevent wind waves on the basin side from overtopping on the road on the inner berm.
4.3.5 Failure mechanisms

A number of failure mechanisms has to be considered in the design of hydraulic structures. Degradation or even failure of (elements of) the structure may occur as a result of loadings during the operational phase or in accidental situations. In this section the principal failure mechanisms for the VT-GC dam are described.

4.3.5.1 Overflow and overtopping

A combination of waves and water levels or a high water level alone can lead to overtopping or overflow. The resistance to erosion of the crest and inner slope is important for the amount of overflow and overtopping that can be resisted. The overflow phenomenon often involves high-velocity flow over the crest. In section 4.1.4.4 the tolerable amount of overtopping is determined to be 50l/m/s. The crest height of the dam is designed for overtopping with a return period of 200 years. Both for the design of the inner and the outer slope overtopping has been taken into account.

The material and shape of the outer slope determine the amount of overtopping that occurs for certain water level and wave conditions. The outer slope material has to withstand flows that occur due to waves. The dimensions of the outer slope material have been calculated in section 4.3.6.1.

Overtopping or overflow will lead to flow velocities on the inner slope of the dam. The outer layer of the inner slope consists of an armour layer to protect the fill material from erosion during wave overtopping. The dimensions of this armour layer have to be large enough to withstand the overflow velocity. The density of this armour layer and the permeability will also affect the stability of the armour layer. Calculations on the dimensions of the inner slope are made in section 4.3.6.1.

4.3.5.2 Sliding inner and outer slope

Lack of stability in the inner or outer slope can lead to sliding. In this case the shear force developed on the sliding surface and the self-weight of the dam at the toe are not sufficient to counteract the force induced by the self-weight of the dam. Especially this can be a problem with steep slopes, high water tables and soft soils. Application of berms will reduce the risk of sliding. Calculations on the stability of the inner and outer slope are performed in section 4.2.3.3
4.3.5.3 Shearing

When the residual force on the dam due to the water height difference is larger than the maximum shear force underneath the dam lateral displacement of the dam (shearing) can occur and the structure will fail. When a sea level of MSL +3.2 m and a basin level of MSL -1 m are assumed with simple geotechnical relations and the Eurocode the resistance against shearing can be calculated, see appendix L. From these calculations it follows that the dam has sufficient resistance, in undrained as well as in drained loading conditions.

4.3.5.4 Micro instability

Micro instability is caused by seepage water that reaches the inner slope of a dam. The smaller particles in the dam can erode due to water pressures from the inside, or the inner slope could lose stability and shear. This danger is largest when the hydraulic gradient in the dam is steepest (high water at sea, low water in the basin.

With an assumed sea level of MSL +3 m and a basin water level of MSL -1 m a discharge through the dam of 12.5 m$^3$/day/m$^2$ is calculated. The maximum velocity in the dam is estimated as 5.3 m/day, based on a hand-drawn flow net. Details can be found in appendix M.

With this seepage flow it is calculated that the dam with an inner slope of 1:3 has enough safety against washing out of the sand particles: the inner slope is shallow and the hydraulic gradient low. However, the resistance against local shearing of the inner slope is too low, and therefore an inner slope of 1:3.5 is required. Calculations can be found in appendix M.

4.3.5.5 Piping

Project Flood Defence HCMC | Design of the dam
Pipes can be formed in the subsoil beneath a cohesive layer, due to seepage water. If the dam core is made of sand only, piping will not be a danger. For the ship locks and discharge sluices this mechanism must be taken into account.

4.3.5.6  
Erosion outer slope

![Erosion outer slope](image)

*Figure 4.61: Erosion outer slope (Weijers & Tonneijck, 2009)*

The outer layer of the outer slope consists of an armour layer to prevent erosion and an underlayer to provide stability to the armour layer and several filter layers to prevent the washing of material.

Waves, currents and differences in water levels determine the lift and drag forces acting on the stones in the cover layer. The inertial forces are also determined by the stone characteristics. The stone weight and forces due to friction and interlocking are stabilising factors. The calculation of the necessary stone dimensions to prevent instability of the protection layer can be found in section 4.3.6.1.

The dynamic loss of balance of all these forces may cause stone movements. Movement of rock cover may also occur on the rear-side face of the dam due to overtopping, see section 4.3.5.1. These responses may be allowed for in the design but care is needed to avoid reactions large enough to initiate other degradation or failure modes such as damage to the filter layer.

4.3.5.7  
Settlement

![Settlement](image)

*Figure 4.62: Settlement (Weijers & Tonneijck, 2009)*

When the dam material and the subsoil settle too much the crest height will become insufficient and the dam will ‘fail’, it will not be able to function properly. Therefore in the design of the foundation and the crest level measures are taken to make sure the crest level maintains sufficient height during the lifetime of the structure, see section 4.2.3.2 and section 4.1.6.

However, still a risk of too large settlements exists because of lack of sufficient subsoil information. Careful monitoring during the lifetime is needed. Because settlement is a slow process additional measures can be taken when the problem is recognised in time.
4.3.5.8  Erosion first bank

The erosion of the first bank can cause instability of the slopes of the dam. This process can start if there is a steep under water slope in front of the dam. Especially with loose packed sand in the subsoil this becomes a problem.

During the closure of the dam, a scour hole occurs due to increased flow velocities on both sides of the dam. A bottom protection is applied to keep this scour hole away from the dam and prevent it from ‘falling’ into the scour hole. Calculation results for the dimensions of the bottom protection can be found in section 5.3.2.

This failure mechanism can also occur due to erosion holes near the ship locks and discharge sluices. A bottom protection has to be applied here.

At locations with a steep foreshore this mechanism should also be checked.

4.3.5.9  Vessel collision

The collision of a vessel could damage the dam and cause collapse. There is no limit state defined in practice. Ships will not navigate very close to the dam or in ultimate conditions. If a ship damages the dam during service conditions the dam can be repaired. The chance of collision during ultimate conditions is low.

For the ship locks the chance of vessel collision is larger.

4.3.6  Protection

4.3.6.1  Slope protection

In this section the slope protection for the basic design of the dam profile will be designed. This includes the protection of the outer (seaward) slope, the inner (landward) slope as well as the crest protection. The slope protection protects these surfaces from wave attack.

First the design parameters are given. Most parameters are resulting from other sections in this report. Then a design for the slope protection is made: first a design for a rock armour layer, followed with the
necessary underlayer and filter layers. Also a design for the crest and inner slope is made for a rock armour layer. Hereafter a design is made for an asphalt armour layer. The section will conclude with an overview of the designed slope protection.

**Design parameters**

*Hydraulic Loading: Waves*

The design waves are waves from the wave conditions of a storm with a return period of 200 years. The significant wave height at the toe of the dam is calculated with the SWAN software, using the wave height and period in deep water \((H_{m0} = 10 \text{ m}, T_p = 12 \text{ s})\) for conditions with a return period of 200 year (appendix D). The output from SWAN for these conditions is given in Table 4.19.

<table>
<thead>
<tr>
<th>(H_s)</th>
<th>3.5</th>
<th>m</th>
<th>Significant wave height at toe of the dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_{2%})</td>
<td>4.6</td>
<td>m</td>
<td>2% Wave height at toe of the dam</td>
</tr>
<tr>
<td>(T_{m-1.0})</td>
<td>8.0</td>
<td>s</td>
<td>Mean wave period from wave spectrum</td>
</tr>
</tbody>
</table>

*Hydraulic parameter: relative water depth*

The relative water depth at the location of the dam says something about the wave conditions and will influence the design of the protection of the dam. The transition from deep to shallow water lies around \(h = 3H_{s-toe}\). The transition from shallow to very shallow water, when: \(H_{s-toe} < 0.70H_{s0}\) (van Gent, 2005, in (CIRIA, 2007)). With Table 4.20 this leads to the conclusion that the toe of the dam and the dam are built in very shallow water.

<table>
<thead>
<tr>
<th>(H_{s-toe})</th>
<th>3.5</th>
<th>m</th>
<th>Significant wave height at toe of the dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>(H_{s0})</td>
<td>10</td>
<td>m</td>
<td>Significant wave height in deep water</td>
</tr>
</tbody>
</table>

*Material*

For the VT-GC dam, two types of material that can be appropriate for the revetment of the outer slope are quarried rocks and asphalt. Quarried rock is a local material and will probably have the right dimensions to be able to apply the largest fraction of rock in the armour layer and the smaller fractions as filter layer material. Asphalt could be a good alternative for the armour layer instead of rock. Asphalt has the advantages that the material is somewhat plastic and can adjust to small settlements, and that probably less material is needed to build the dam. A disadvantage of using asphalt instead of rock is that the crest height elevation should be higher because of higher run up levels.

Concrete blocks or tetrapods have been taken into account in a preliminary study, but are thought to be an uneconomic alternative for quarried rock. If it appears to be the case that the necessary volume of rock is not available at nearby quarries, the concrete options have to be investigated.

*Cross section*

The general cross sections have been determined in section 4.4. The inclination of the slope of the dam is 1:4. The fill material of the dam is sand. Cross sections are presented in Figure 4.65 and Figure 4.66.

Rock and asphalt revetment have different material characteristics (like the roughness coefficient) and will lead to different run up levels (see section 4.1.4). That is the reason why there is a difference in crest height for the different armour layers.
Figure 4.65: Schematic view of the rock dam

Figure 4.66: Schematic view of the asphalt dam

Layers
The typical layering a dam profile with rock will have an armour layer (often a double layer), one or more thin granular underlayers or filters, and a core of rather finer material. The underlayer has to stabilise the armour layer, and the filter layers have to prevent the washing away of finer material. The way to design the underlayer and filter layers of a dam with a rock armour layer are described in literature.

The way to design the underlayer and filter layer for a dam with an asphalt revetment is unclear from literature. To be able to design an appropriate underlayer and filter layer for the asphalt armour layer, it would be necessary to do a literature study. For this reason, the underlayer and filter layer of asphalt are not looked into in this report.

Structure classification
The dam structure is a non-overtopped or marginally overtopped structure, with a high crest elevation only overtopped under severe wave conditions. The wave attack on the seaward slope is higher than for low-crested structures. Under design conditions some wave overtopping may occur.

Armour layer - Rock
The basic approach to determine the stability of rock armour layers is to assess the stability a slope covered with rough angular shaped armour stone, placed in a double layer on filter layers also consisting of armour stone.

The approach used here is the approach described in the Rock Manual (CIRIA, 2007). First an estimate of the necessary stone size is calculated with a very simple approach, and hereafter the modified Van der Meer formula for shallow water is applied to calculate the stone size. With this stone size the necessary amount of material is calculated.

First estimate
Stability analyses of structures under wave attack are commonly based on the stability number, $N_s$. The stability number gives a relationship between the structure and the wave conditions (CIRIA, 2007).

$$N_s = \frac{H_s}{\Delta D_{n50}}$$
The values of the parameters are listed below, as well as the outcome of the formula. The origin of the values of the input parameters can be found in appendix N.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_s$</td>
<td>1 to 4</td>
<td>-</td>
<td>Stability number</td>
</tr>
<tr>
<td>$H_s$</td>
<td>3.5</td>
<td>m</td>
<td>Significant wave height</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>1.5</td>
<td>-</td>
<td>Relative buoyant density</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>0.6 to 2.3</td>
<td>m</td>
<td>Characteristic stone size or diameter</td>
</tr>
</tbody>
</table>

The outcome of the first estimate for the characteristic stone size is ranging from 0.6 to 2.3 m. The range of this output is too large to be useful for the design of the dam. More detailed calculations are needed on the wave–slope interaction and the range of the stability number could be adjusted.

**Calculation method**

The methods available to evaluate the stability of rock armour layers on non-overtopped hydraulic structures (see section 'Structure classification') are dependent upon the specific hydraulic conditions and structural parameters. The basic approach is to assess the stability of slopes covered with rough angular shaped armour stone, placed in a double layer on an underlayer and filter layers also consisting of armour stone, but smaller fractions.

The decision for what formula to use depends mostly on the ‘relative water depth’ at the toe of the structure and the permeability of the structure:

- Non-breaking waves on the foreshore (deep water) \( \Rightarrow \) Hudson
- Breaking waves on foreshore (depth-limited waves) \( \Rightarrow \) Hudson
- Very shallow foreshores \( \Rightarrow \) Van Gent et al.
- Deep water (non-depth-limited waves) \( \Rightarrow \) Van der Meer
- Shallow water and gently sloping foreshores \( \Rightarrow \) modified Van der Meer

The formulas of Hudson can only be used for permeable structures, and the formula of Van Gent works best for structures with a permeable core. As the dam structure will not be that permeable, the Van der Meer formula will be used.

As the dam is constructed in ‘very shallow’ water (see section ‘Hydraulic parameter: relative water depth’), the Van der Meer formula for very shallow water will be applied.

**Van der Meer formula – Input parameters**

To apply the van der Meer formula, first a number of parameters have to be determined. The determination of these parameters can be found in appendix N. The parameters and their values are listed in Table 4.21.

**Table 4.21: Van der Meer formula: input parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{2%}$</td>
<td>4.6</td>
<td>m</td>
<td>2 per cent wave height at toe of structure</td>
</tr>
<tr>
<td>$T_{m-1,0}$</td>
<td>8.0</td>
<td>s</td>
<td>Spectral wave period at toe of structure</td>
</tr>
<tr>
<td>$S_d$</td>
<td>4</td>
<td>-</td>
<td>Damage level parameter</td>
</tr>
<tr>
<td>$N$</td>
<td>3000</td>
<td>-</td>
<td>Number of waves</td>
</tr>
<tr>
<td>$\xi_{s-1.0}$</td>
<td>1.2</td>
<td>-</td>
<td>Surf similarity parameter</td>
</tr>
<tr>
<td>$c_{pl}$</td>
<td>8.7</td>
<td>-</td>
<td>Plunging coefficient, shallow water calculation</td>
</tr>
<tr>
<td>$P$</td>
<td>0.4</td>
<td>-</td>
<td>Notional permeability</td>
</tr>
<tr>
<td>$\xi_{CR}$</td>
<td>2.6</td>
<td>-</td>
<td>Critical value of the surf similarity parameter</td>
</tr>
</tbody>
</table>
Van der Meer formula - Calculation

Comparing the surf similarity parameter with the critical value of the surf similarity parameter:

\[ \xi_{s-1.0} \leq \xi_{cr} \]  \hspace{1cm} (4.10)

This means that plunging conditions are fulfilled, and that the following formula (for very shallow water conditions) holds:

\[ \frac{H_s}{\Delta D_{n50}} = c_{pl}P^{0.18} \left( \frac{S_d}{\sqrt{N}} \right)^{0.2} \left( \frac{H_s}{H_{2\%}} \right)^{0.2} (\xi_{s-1.0})^{-0.5} \]  \hspace{1cm} (4.11)

Table 4.22: Van der Meer formula in very shallow water: total of input parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_{pl} )</td>
<td>8.7</td>
<td>- Plunging coefficient, shallow water calculation</td>
</tr>
<tr>
<td>( P )</td>
<td>0.4</td>
<td>- Notional permeability</td>
</tr>
<tr>
<td>( S_d )</td>
<td>4</td>
<td>- Damage level parameter</td>
</tr>
<tr>
<td>( N )</td>
<td>3000</td>
<td>- Number of waves</td>
</tr>
<tr>
<td>( H_s )</td>
<td>3.5 m</td>
<td>- Significant wave height at toe of structure</td>
</tr>
<tr>
<td>( H_{2%} )</td>
<td>4.6 m</td>
<td>- 2 per cent wave height at toe of structure</td>
</tr>
<tr>
<td>( \xi_{s-1.0} )</td>
<td>1.2</td>
<td>- Surf similarity parameter</td>
</tr>
<tr>
<td>( \Delta )</td>
<td>1.5</td>
<td>- Relative buoyant density</td>
</tr>
<tr>
<td>( D_{n50} )</td>
<td>0.76 m</td>
<td>- Armour stone size</td>
</tr>
</tbody>
</table>

The outcome of the Van der Meer formula is a characteristic diameter for the armour stones of 0.76 m. This is a realistic value for the armour layer of a construction in relative open sea. The value also lies in the range of the first estimated diameter in section ‘First estimate’: 0.6 to 2.3 m. 0.76 m is a reasonable outcome when taking into account:

- The higher the damage level that is chosen, the smaller the required size of the diameter of the armour stones. The damage level is chosen to be 4, so the armour stone size will be on the small side of the range from the first estimate outcome.
- The shallower the location of the armour layer, the more breaking occurs in the zone of the armour layer, resulting in larger required armour stones. The location of the dam and armour layer is shallower than ‘assumed’ in the first estimate, so the armour stone size is larger than the minimum value in the range of the first estimate outcome.

The following remarks must be made for what is not taken into account in this calculation. The most important issues that are not taken into account in this calculation are: the porosity and packing density of the armour layer, the influence of shallow and gently sloping foreshore and the effect of oblique wave approach. The outcome from this equation is however sufficient for the first design. When a more detailed design is required the issues above should be taken into account.

Amount of material needed

From section ‘Van der Meer formula - Calculation’ it follows that quarried rock with a \( D_{n50} \) of 0.76 m is needed for the armour layer. With the general dimensions of the armour layer and the packing, it is possible to determine the amount of rocks needed for the armour layer per meter dam.

In general, the layer thickness for any armour stone layer is a minimum of at least two layers of rocks, with a factor \( k_t \) in-between:

\[ t_d = 2k_t D_{n50} \]  \hspace{1cm} (4.12)
The volume of rock has to be calculated to know the amount of rock to buy at the quarry. First the bulk volume is calculated (the rock volume plus void volume) using the thickness of the layer. With this bulk volume and the porosity the volume of rock can be calculated. The porosity of the rock layer is estimated on 37% (Kamphuis, 2000).

![Armour layer geometry](image)

*Figure 4.67: Armour layer geometry*

The design bulk volume of armour stone is given by:

\[
V_{b,d} = A \times t_d
\]  
(4.13)

<table>
<thead>
<tr>
<th>(A)</th>
<th>50 m²/m</th>
<th>Surface area of armour layer parallel to the slope (per meter dam length)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(t_d)</td>
<td>1.5 m</td>
<td>Layer thickness</td>
</tr>
<tr>
<td>(V_{b,d})</td>
<td>75 m³/m</td>
<td>Design bulk volume armour stone (per meter dam length)</td>
</tr>
</tbody>
</table>

The volume of rock is calculated with:

\[
V_r = V_b (1 - n_v)
\]  
(4.14)

| \(V_b\) | 75 m³/m | Design bulk volume armour stone (per meter dam length) |
| \(n_v\) | 37 % | (Volumetric) layer porosity |
| \(V_r\) | 47 m³/m | Volume of rock (per meter dam length) |

The volume of rock needed to build 1 m of the armour layer of the outer slope is 47 m³/m³.

---

7 A layer thickness of 2 times the diameter might be considered conservative, for example the Rock Manual will go as far as a \(k_z\) of 0.87 for a double standard layer of irregular rock. For this stage of the design process a layer thickness of 2 times the diameter is a good base to work from.
Underlayers and filter layers – Rock

Rock structures are normally constructed with an armour layer, one or more thin granular underlayers or filters, and a core of rather finer material like sand.

Underlayer

The underlayer has the following functions according to (CIRIA, 2007):

- Act as filter
- Protect subsoil and preventing core erosion
- Provide in-plane drainage
- Regulate or level layer that provides appropriate surface for armour layer placement
- Separate armour from smaller sized materials and reduces hydraulic gradient into subsoil/core

The Shore Protection manual (SPM) (U.S. Army Engineer Waterways Experiment Station, 1984) recommends a certain ratio between the stone mass of the underlayer and that of the armour layer. This ratio results in this ratio for the nominal diameters:

\[
\frac{D_{n50a}}{D_{n50u}} = 2.2 \text{ to } 2.5
\]  

<table>
<thead>
<tr>
<th>(D_{n50a})</th>
<th>(D_{n50u})</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 m</td>
<td>0.40 to 0.45 m</td>
<td>Nominal diameter armour layer</td>
</tr>
<tr>
<td>0.40 to 0.45 m</td>
<td></td>
<td>Nominal diameter underlayer</td>
</tr>
</tbody>
</table>

The nominal diameter for the stone size of the underlayer lies in the range of 0.40 to 0.45 m. For greater stability of the armour layer, the high end of the range is chosen: a nominal diameter of the underlayer of 0.45 m.

For determining the layer thickness of the underlayer, the same formula can be used as for the armour layer. With the determined nominal diameter this will lead to a layer thickness of 0.9 m. This thickness meets the requirement that any armour stone layer should have a layer thickness of at least 300 - 500 mm to overcome placement irregularities.

Filter layer

The main function of protective filters is to prevent the washing away of the underlying material (Schierack, 2001). That means that the underlying grains (usually referred to as the base layer) should not pass the pores of the upper layer (the filter layer). This can be prevented either by one or more layers of grains of varying diameter, a granular filter, or a geotextile.

A granular filter is thought to be the most probable solution, because rock from a quarry is used and smaller fractions from the quarry could be used for the granular filter. There are two main types of granular filters: geometrically closed filters and geometrically open filters. With a geometrically closed filter, the diameter of the filter layer grains is much larger than the diameter of the grains of the fill material, thus preventing the fill material to move. In a geometrically open filter, the grains of the base layer can erode through the filter layer, but the occurring gradient will stay below the critical value. The geometrically closed filter is chosen, because it is critical that the dam will not erode and this variant will need less surveillance and maintenance.

There are three type of relations that have to be checked to design a filter layer: stability between filter layer and base layer, permeability and internal stability (Schierack, 2001).
Stability: 
\[
\frac{d_{15F}}{d_{85B}} < 5 \quad (4.16)
\]

Internal stability: 
\[
\frac{d_{60F}}{d_{10F}} < 10 \quad (4.17)
\]

Permeability: 
\[
\frac{d_{15F}}{d_{15B}} > 5 \quad (4.18)
\]

<table>
<thead>
<tr>
<th>(d_{15B})</th>
<th>0.15 mm</th>
<th>15% Grain size of base material (medium sand)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(d_{85B})</td>
<td>0.65 mm</td>
<td>85% Grain size of base material (medium sand)</td>
</tr>
</tbody>
</table>

These equations and values for the base material result in the following requirements for filter material:
\[
0.75 < d_{15F} < 3.25 \text{ [mm]} \quad (4.19)
\]
\[
d_{60F} < 10d_{10F} \text{ [mm]} \quad (4.20)
\]

So a possible filter material could have for example these parameters, but a lot more options are possible:

| \(d_{10F}\) [mm] | 1.7 | 1.8 | 2.5 | 2.7 |
| \(d_{15F}\) [mm] | 2.0 | 2.0 | 3.0 | 3.0 |
| \(d_{60F}\) [mm] | 3.1 | 2.7 | 4.7 | 4.0 |

The choice for the filter material will depend on what material is locally available. This material has to be tested to get to know the values of \(d_{10}, d_{15}, d_{60}\). With these parameters and the above requirements for the filter material, a decision on the applicability of the material as filter material can be made.

To prevent the filter layer from washing away, more filter layers have to be added, until the grain size of a filter layer is large enough, so that the material is not washed away through the pores of the underlayer.

With the formulas from prior section the following filter layers could be possible:

<table>
<thead>
<tr>
<th>(d_{10F})</th>
<th>(d_{15F})</th>
<th>(d_{60F})</th>
<th>(t_F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.7</td>
<td>2.0</td>
<td>3.1</td>
<td>500</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>25</td>
<td>500</td>
</tr>
<tr>
<td>100</td>
<td>120</td>
<td>200</td>
<td>500</td>
</tr>
<tr>
<td>750</td>
<td>900</td>
<td>1500</td>
<td>= underlayer</td>
</tr>
</tbody>
</table>

The fourth filter layer has larger dimensions as the underlayer. This means that the underlayer is capable of preventing the third filter layer to flow through the underlayer. The calculated fourth filter layer has therefore not to be constructed.

The thickness of the layers is determined based on the practical minimum layer thickness to overcome placement irregularities.
Crest Protection

The crest of a rock armour breakwater is usually made up of the same rock as the rest of the armour layer (Kamphuis, 2000). A minimum practical width of crest protection is three primary armour stone widths (CIRIA, 2007):

\[ B = 3k_tD_{n50} \]  

(4.21)

<table>
<thead>
<tr>
<th>( D_{n50} )</th>
<th>( k_t )</th>
<th>( B )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.76 m</td>
<td>1</td>
<td>2.3 m</td>
</tr>
</tbody>
</table>

Shape factor coefficient \( k_t \) and characteristic diameter rock \( D_{n50} \) are tabulated.

The stones on the crest should be placed with maximum interlocking or packing density to ensure the greatest stability under wave action (CIRIA, 2007).

The actual crest width also depends on the necessary core crest width \( B_{core} \) (m). Because if the core is built out with dump trucks, the core crest should be large enough for the building equipment (trucks, cranes) to use it. So if the crest is used during the construction, the crest width will be larger than the minimum width necessary to ensure the stability of the crest.

Inner slope protection

The method of (Verhagen et al., 2001) is used to calculate the necessary stone size of the inner slope protection to withstand the velocities of overtopping waves.

Characteristic stone size

(Verhagen et al., 2001) suggests to use \( N_{od} < 0.25 \), which means:

\[ d_{n50} > \frac{u_{char}^2}{3\Delta g} \]  

(4.22)

Where \( u_{char} \) is the characteristic velocity of the water on the inner slope. This velocity can be computed by calculating the velocity of the water on the outer slope first and the layer thickness of the water on the crest. These calculations can be found in appendix N. The outcome of these calculations lead to a characteristic stone size for the armour stones of the inner slope of 0.34 m.

Layer thickness

To calculate the layer thickness of the armour layer on the inner slope, the same calculation as used for the thickness of the armour layer of the outer slope can be used. In general, the layer thickness for any armour stone layer is a minimum of at least two layers of rocks, with a factor \( k_t \):

\[ t_d = 2k_tD_{n50} \]  

(4.23)

<table>
<thead>
<tr>
<th>( k_t )</th>
<th>( D_{n50} )</th>
<th>( t_d )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.34 m</td>
<td>0.68 m</td>
</tr>
</tbody>
</table>

Shape factor coefficient \( k_t \) and characteristic diameter rock \( D_{n50} \) are tabulated.

This calculation leads to a layer thickness of 0.68 m.

---

8 A layer thickness of 2 times the diameter might be considered conservative, for example the Rock Manual will go as far as a \( k_t \) of 0.87 for a double standard layer of irregular rock. For this stage of the design process a layer thickness of 2 times the diameter is a good base to work from.
Underlayer inner slope
The same formulae for the underlayer can be used as for the underlayer of the armour layer of the outer slope:

\[ \frac{D_{n50a}}{D_{n50u}} = 2.2 \text{ to } 2.5 \]  

<table>
<thead>
<tr>
<th>Diameter (m)</th>
<th>Nominal diameter armour layer</th>
<th>Nominal diameter underlayer</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For a greater stability of the armour layer, the high end of the range is chosen: a nominal diameter of the underlayer of 0.15 m.

Using the same formula as used for the thickness of the armour layer, the thickness of the underlayer would be 0.3 m. This thickness is just sufficient to meet the minimum layer thickness of 300 - 500 mm to overcome placement irregularities. To be safe, in the whole report a minimum layer thickness of 0.5 m for loose material is taken into account. The layer thickness for this underlayer should thus be 0.5 m.

Filter layer inner slope
The requirements for the filter layer for the inner slope are taken to be the same as the requirements for the outer slope. As the design for the filter layer starts with the input variables of the base layer, the first few filter layers will be the same as the filter layers of the outer slope:

<table>
<thead>
<tr>
<th>Filter layer 1</th>
<th>Filter layer 2</th>
<th>Filter layer 3</th>
<th>Filter layer 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.7</td>
<td>12</td>
<td>100</td>
<td>750</td>
</tr>
<tr>
<td>2.0</td>
<td>15</td>
<td>120</td>
<td>900</td>
</tr>
<tr>
<td>3.1</td>
<td>25</td>
<td>200</td>
<td>1500</td>
</tr>
</tbody>
</table>

Filter layer 2 will be the last necessary filter layer, because the dimensions of filter layer 4 are larger than the dimensions of the underlayer and the dimensions of the third filter layer are approximately the same or larger than the dimensions of the underlayer \( D_{n50} = 0.15 \text{ m} \). So only the first 2 filter layers are necessary.

Armour layer – Asphalt
A slope with an asphalt armour layer will create an impervious slope. The slope of the dam profile with the asphalt armour layer is designed to have a slope of 1:4.

Asphalt layer failure modes
The most important failure modes of asphalt layers are uplift and shearing (Schiereck, 2001). Shear means that a part of the layer will hang on the upper part of the protection, or is leaning on the lower part. Shear failure should not occur too often to avoid fatigue. So under springtide conditions, with a frequency of occurrence in the order of magnitude of weeks, the shear criterion should be met.

Uplift is worse: sand can move under the protection when it is lifted and the layer can deform. The uplift criterion should be met in design circumstances, like a fast fall of water level after a storm.

Uplift criterion
The protection is stable against uplift when there is equilibrium perpendicular to the slope like in equation (4.27). To find the maximum ground water level in the dam (HGW) during design conditions:
The maximum ground water level in the dam is MSL +1.6 m.

Calculating the maximum excess pressure, for a storm situation:

\[
H = \frac{h_1}{\pi} \arccos \left( 2 \left( \frac{h_1 + d \cos \alpha}{h_1 + h_2} \right)^{\frac{\pi}{\arctan(\cot \alpha) + \pi/2}} - 1 \right)
\]  \hspace{1cm} (4.26)

After a few iterations the result from these calculations is an asphalt thickness of 0.73 m.

**Shear criterion**

The friction between protection and subsoil must be enough to balance the component of the weight parallel to the slope. The friction depends on the effective weight perpendicular to the slope. Stability for a length of slope \( \Delta x \) just below the outside water level is ensured when equation (4.27) holds.

To find the maximum ground water level in the dam (HGW) during spring tide:

\[
HGW = a(HW - MWL)
\]  \hspace{1cm} (4.29)

Calculating maximum excess pressure, for springtide:
\[ H = \frac{h_1}{\pi} \arccos \left[ 2 \left( \frac{h_1 + d \cos \alpha}{h_1 + h_2} \right)^{\frac{\pi}{\arctan(\cot \alpha) + \pi / 2}} - 1 \right] \]  

(4.30)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_1 )</td>
<td>0.5</td>
<td>m</td>
<td>Height distance between HGW and MSL</td>
</tr>
<tr>
<td>( h_2 )</td>
<td>5</td>
<td>m</td>
<td>Height distance between MSL and low end of layer</td>
</tr>
<tr>
<td>( d(1) )</td>
<td>0.5</td>
<td>m</td>
<td>Thickness of layer (estimated)</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>14</td>
<td>°</td>
<td>Slope angle (1:4)</td>
</tr>
<tr>
<td>( H )</td>
<td>0.37</td>
<td>m</td>
<td>Maximum excess pressure</td>
</tr>
</tbody>
</table>

Equilibrium of forces states that:

\[ f[(\rho_m - \rho_w)g \Delta x \cos \alpha - H \rho_w g \Delta x] \geq (\rho_m - \rho_w)g \Delta x \sin \alpha \]  

(4.31)

This results in the equation:

\[ \frac{H}{\Delta d} = \frac{f \cos \alpha - \sin \alpha}{f} \]  

(4.32)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H )</td>
<td>0.37</td>
<td>m</td>
<td>Maximum excess pressure</td>
</tr>
<tr>
<td>( \Delta )</td>
<td>1.4</td>
<td>-</td>
<td>Relative buoyant density</td>
</tr>
<tr>
<td>( f )</td>
<td>0.38</td>
<td>-</td>
<td>Friction, estimate 2/3 tan ( \varphi ) = 0.38</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>14</td>
<td>°</td>
<td>Slope angle (1:4)</td>
</tr>
<tr>
<td>( d(2) )</td>
<td>0.79</td>
<td>m</td>
<td>Thickness of layer (first iteration)</td>
</tr>
</tbody>
</table>

After a few iterations the result from these calculations is an asphalt thickness of 0.76 m.

**Thickness of asphalt armour layer**

Comparing the outcomes for the thickness of the asphalt layer for the uplift and the shear criterion leads to the conclusion that the shear criterion is normative.

This value for the thickness is really conservative, as no cohesion is assumed in the asphalt layer and the thickness for the most unfavourable spot of the revetment is calculated. In (Schiereck, 2001) it is therefore suggested to take the thickness of the asphalt layer as a parabolic with the maximum thickness as outlined above and the minimum thickness determined by construction aspects (0.1 to 0.2 m).

### 4.3.6.2 Toe protection

The toe protection provides a stable footing to the armour layer. In appendix O is described how to calculate the necessary stone size for the toe protection. Because the protection layers are relatively thick in for the rock armour slope, the top of the toe protection will have an elevation high above the sea bottom compared to the water depth. The toe acts like a berm. The stability for this situation should be calculated similarly to the stability of the armour slope. Stones with a \( d_{50} \) of 0.76 m (same as for the armour layer) will be used for the toe protection. If thinner or less filter layers are applied the toe protection will be lower and stones will be smaller. The minimum stone size (for a situation without filter layers) was determined to be 0.44m.
If asphalt is used as a revetment, a drainage filter has to be applied at the toe to prevent high excess pressures under the asphalt layer. The filter should be able to resist the concentrated upflow that can occur due to pressure differences. It has to be closed for the bottom material and permeable for water. In Figure 4.69 and Figure 4.70 two possible options for toe protection are drawn: a toe protection made of rock in the bottom and a toe protection which is located above the bottom. The protection above the bottom is higher and may therefore need courser material. Construction is much easier and therefore this design is chosen for the project.

For both the toe protection for the rock slope and the toe protection for the asphalt slope, one or more underlayers or a geotextile are necessary to prevent bottom material from washing out through the pores of the toe. The usual design approach (Schiereck, 2001) is to establish the grain diameter that is required for stability in waves and currents. Then the filter relations between the filter and base material must be checked. If necessary a geotextile or as many granular layers as required must be added.

In front of the toe a scour protection is necessary to prevent erosion and undermining of the toe.

It is recommended to do scale model tests for the toe of the structure. A smaller grading can lead to cost reductions.

4.3.6.3 Bottom protection

If flow velocities around the structure increase, an erosion hole may occur as a function of time (appendix R). A bottom protection is required to keep the erosion hole away from the dam and prevent the dam from sliding into this erosion gap.

For the largest part of the VT-GC dam, the flow velocities will not increase significantly during construction. At the location of the final gap velocities may reach values up to 4 m/s (section 5.2). A
bottom protection is therefore necessary at this location. In section 5.3.2 this bottom protection will be described further.

Near the ship locks and discharge sluices flow velocities may also cause scour. In the vicinity of these structures a bottom protection is necessary.

## 4.4 Final design

When combining all previous determined heights, slopes and protections, a final dam design can be made. In the foundation as well as in the revetment there are several profiles possible. Due to the availability of local sand a dam with a sand core is designed.

For the foundation, in case of the strong Vung Tau and Go Cong profile the best option is to replace the upper layer by sand. In case of the weak Vung Tau profile drainage and preloading is preferred. For the final design, sand replacement is assumed to be as foundation, because for two out of three profiles this solution is favourable.

For the revetment a rock and asphalt are looked into. Together with the chosen slopes and berms the cross section of the dam design with rock revetment will be as drawn in Figure 4.71.

![Figure 4.71: Cross section dam with rock revetment.](image)

The crest of the rock revetment dam lies at MSL + 6.8 m. On the sea side, a berm of 16 m is present, and the slopes of 1:4 are steep enough to prevent too much overtopping. The berm is located on MSL + 3.2 m, one meter above Still Water Level. On the land side, the slopes are steeper due to the calmer and lower water. On this side a 25 m wide inner berm for a 2x2 road is constructed. The top-protection layer is made out of rock, supported by an underlayer and filter layers, which prevent the sand fill from outflowing.

On the inner side, the stone size and layer thickness of the revetment is determined based on the effects of overtopping water. This results in a smaller protection layer than on the outer side. For stability reasons a toe protection with stones is applied. Bottom protection is placed to prevent scour, which may cause instability.

When an asphalt revetment is used the cross section will look as in Figure 4.72.
Figure 4.72: Cross section dam with asphalt revetment and toe protection above the bottom.

In general, the cross section is the same as for the dam with rock revetment, only the differences will be discussed. The asphalt thickness varies over the cross section: on the sea side near the height of MSL the asphalt is the thickest, 0.76 m. Under this an underlayer of 0.5 m is present to ensure stability of the asphalt layer. The height of the crest has to be higher for the asphalt revetment (the crest level is at MSL + 7.6 m), because of the higher run-up compared to a rock revetment. The toe protection is made above the sea bottom level, consisting of scour protection and rock to prevent the asphalt from sliding.

More detailed technical drawings of the dam can be found in appendix U.
5 DESIGN OF THE CLOSURE GAP

A very important aspect within the construction phase of any closure dam is the final closure. Already during the design phase, much attention needs to be paid to this operation. This chapter will not focus on the cross section of the dam, because this is not relevant for the closure method itself. It will merely focus on the hydraulic behaviour of the system when the closure gap is being narrowed.

First, different closure methods will be discussed. Hereafter a storage area approach will be used to get an idea of the expected velocities in the closure gap. This information will later be used to determine what method and materials can best be used for the closure of the Vung Tau - Go Cong dam.

5.1 Closure methods

For the closure of a dam, roughly 4 different methods are available. These options are schematised in Figure 5.1. The first 3 options are methods for gradual closure. Relatively small sized material is deposited until the flow is blocked. The last option is a type of sudden closure with a prefabricated structures, i.e. concrete caissons.


![Diagram of Different Methods of Closure](image)

Figure 5.1: Different methods of closure (CIRIA, 2007)

Closure methods can also be distinguished according to the topography of the gap. Depending on the topography, shipping routes and expected velocities at the closure gap, the designer can choose to locate the closure gap at for instance a tidal gully or at a shallow part (i.e. tidal flat).

With the design graphs that will be derived in section 5.2.3, the most suitable type of closure method could be determined based on the resulting velocity in the closure gap. Besides the critical velocity in the
In the design of the closure gap, there are a number of aspects which also determine the choice for a certain type of closure. These are, partially after (CIRIA, 2007):

- Construction time: how fast can the gap be closed with a certain method
- Subsoil conditions: what is in the subsoil and what is the erodibility
- Availability of construction materials: sand, rock, clay, gravel
- Locally available labour: skills and numbers
- Possibility of using waterborne equipment
- Possibility of using land-based equipment
- Accessibility of the site

In the following sections, the closing strategies will be discussed in more detail based on the Rock Manual (CIRIA, 2007). In the following sections, different options for the closure of the Vung Tau – Go Cong dam will be discussed, based on the design graphs of section 5.2.3.

### 5.1.1 Horizontal closure

The method of horizontal closure is based on the dumping of stones from the sides of the closure gap. The advantage of end-dumping is that it is a relatively simple system, and that it may be possible to use smaller stone sizes at the start of the operation, since the velocities in the closure gap are low at that point. The dam needs to be wide enough to account for a two-way road on which trucks can pass each other, in case of a long closure (CIRIA, 2007). Because the velocity in the closing gap can become rather high, bed protection should be applied in most cases to prevent erosion. The method has been applied all over the world, see for examples the Saemangeum Sea Wall in Korea (see appendix A).

### 5.1.2 Vertical closure

With vertical closure, the sill is gradually raised over the full width of the closure gap. In the beginning stage, ships can be used to dump stones. The maximum current velocity is limited to that of a broad-crested weir (CIRIA, 2007). The disadvantage of this method is that large stone sizes need to be applied over the full length of the closure gap. During the period of full closure, the porosity of the rock bund is still rather high. At a certain elevation of the bund, ships cannot be used to dump the stones. From that point onwards, other means must be used to transport and dump the stones. There are various options to do this, like:

- Dumping from a bridge built over the closure gap
- Transporting rocks or concrete blocks with a cable way erected over the closure gap
The latter option has been applied in the Delta Works in the Netherlands. For the Haringvliet dam (and also the Brouwersdam), concrete blocks were dumped into the sea from cable ways. See appendix A for this reference project.

With vertical closure, lighter bed protection can be used because the method creates less turbulence downstream of the closure gap. Also, a smaller stone size can be applied in the final stage of closure, compared to horizontal closure. Reason for this is that the velocity in the closure gap is lower compared to horizontal closure near the point of closing, as will be shown in section 5.2.3. The expected velocities reach a maximum (which can still be quite high) but at a certain depth the velocity reduces. Concrete blocks have an advantage over stone blocks because they are easier to load and dump at the right location when using a cable way. However, their production can make it a more expensive option, compared to stone from a quarry. If stones are not locally available, it might be feasible to use concrete.

The stability of the slopes can be affected when a certain grading of rocks leads to a lot of through-flow.

5.1.3 Combined closure

A building strategy could be to combine horizontal with vertical closure. Horizontal closure with sand can be applied until the velocity in the closure gap gets too high. From that point on heavier elements that can withstand the high velocities can be applied. Another option is to switch to vertical closure at some point, as was shown in section 5.2.3. First, a sill is constructed and then the gap is narrowed from the sides.

5.1.4 Sudden closure with caissons

The closure with caissons can be applied to close the final gap suddenly. Caissons can be placed during tidal slack by using boats. There are also caissons which are able to let water through during the period of placement. These caissons have large openings through which water can flow during the construction phase so that the flow is not obstructed too much. This way the velocities in the closure gap remain relatively low. When all caissons are placed, they are all closed at the same time during tidal slack by means of built-in gates. These sluice caissons can be used when the final gap is large, and the tidal motion...
in considerable (Huis in ’t Veld et al., 1987). Closed caissons are applicable in small gaps or in areas with a small tidal motion. Figure 5.3 shows a sluice caisson.

![Figure 5.3: Layout of sluice caisson before gap closure (Huis in ’t Veld et al., 1987)](image)

The disadvantage of the closure with caissons is that the placement procedure is critical. A lot of attention is needed for the planning of the placement operation. The method is highly sensitive to weather conditions and tidal movements. Neap tide is the ideal moment for the gap closure.

The sudden closure with caissons can be divided into three stages (Huis in ’t Veld et al., 1987):

1. A flat, stable sill structure with vertical or oblique abutments is constructed on a stable, scour-resistant sea bed, protected by a bottom revetment.
2. a) Rigid concrete caissons are floated into position, successively, across the gap.
   b) When a caisson has been sunk into position on the sill, wooden boards along the sides of it are opened (in fact totally removed), so that the tide can flow through.
3. When all caissons are in position, their gates are closed simultaneously at slack water, thus affecting the closure.
4. The permanent dam is then built around the caissons, if these have only a permanent function.

Special attention needs to paid to the construction of the sill, since the height of the sill determines the water velocities and hence the amount of scouring. A hydraulic model should be used to determine the expected velocities. In the next section, a storage area approach will be used to make a first evaluation of the expected velocities.

### 5.2 Storage area approach

#### 5.2.1 Model description

The storage area approach is a very simple way of determining the system behaviour of the closing structure. The approach basically consist of a reservoir model which links hydraulic boundary conditions (water levels and discharges) with the geometry of the closure area. The most basic storage area approach schematizes the water movement in the estuary without any friction or inertia and is valid for a short basin with a length smaller than 1/20 of the wavelength (Verhagen et al., 2009). The water level inside the basin is assumed to be horizontal. Weir formulae are applied to the outflow points.
The calculations were carried out in a spread sheet program (Microsoft Excel). The model exists of branches and nodes at which calculations are being made. In the model, an explicit scheme is used for calculating the water level in the reservoir. For numerical reasons, the discharge sluice has been modelled as a branch which has a certain length and roughness. At the weirs, an implicit scheme was used. Figure 5.4 shows the schematisation of the model.

![Simplified reservoir model](image)

The storage area approach links the in- and outflows to the water levels inside and outside the reservoir. For the outflows, weir formulae are used:

\[
Q_{in} - Q_{out} = Q_{river} - Q_{sluice} - Q_{gap} = A_{reservoir} \frac{dh_{reservoir}}{dt}
\]  

\[
Q_{sluice} = \mu A_{sluice} \sqrt{2g\Delta h}
\]  

\[
Q_{gap} = \mu A_{gap} \sqrt{2g\Delta h}
\]  

\[
\Delta h = h_{reservoir} - h_{sea}
\]  

\[
v_{gap} = \frac{Q_{gap}}{A_{gap}}
\]  

When all parameters are known, the water level \( h_{reservoir} \) and discharges \( Q_{sluice} \) and \( Q_{gap} \) can be calculated as a function of time. When the discharge coefficient \( \mu \) is set to 1, the average velocity in the gap can be calculated by using equation (5.5).

In the model, two closing methods were investigated: horizontal and vertical closure. Figure 5.1 shows some methods for gap closure. The construction methods were discussed in section 5.1. In section 5.2.3, design graphs will be given for different discharge sluices so that the closing strategy can be determined for the Vung Tau - Go Cong dam.

### 5.2.2 Model input, boundary conditions and assumptions

The model described in section 5.2.1 needs input about:

- Surface area of the storage basin \( A_{reservoir} \)
- Tidal movement \( h_{sea} \)
- Cross section area of the discharge sluice ($A_{sluice}$)
- River discharge ($Q_{river}$)

The surface area $A_{reservoir}$ has been estimated by drawing the dam in Google Earth. The measured usable area for storage is around 30,000 ha. The exact storage area also depends on the river banks and mangrove forests and needs further study in a later phase of the design. For now, only the enclosed basin downstream of the Soi Rap mouth has been taken into account. In reality, $A_{reservoir}$ is a function of the water level $h_{reservoir}$ but since such a relation is not known, the surface area has been taken equal for every water level. The used numerical calculation time step in the model is 600 seconds.

For the simulation of the water level at sea, the tide has been predicted by using a model which calculates the tide based on the known phase angles and amplitudes of the tide at Vung Tau. Input data was taken from the Admiralty Chart. Table 5.1 shows the used data. In appendix P, the performance of this model is evaluated by comparing the model outcome with measured water levels near Vung Tau. For this phase of the design, the model suffices. For further information about tides and tidal constituents, the reader is referred to textbooks.

<table>
<thead>
<tr>
<th>Tidal constituent</th>
<th>Description</th>
<th>Nature</th>
<th>Amplitude (m)</th>
<th>Phase (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2</td>
<td>Principal lunar</td>
<td>Semi-diurnal</td>
<td>0.79</td>
<td>36</td>
</tr>
<tr>
<td>S2</td>
<td>Principal solar</td>
<td>Semi-diurnal</td>
<td>0.31</td>
<td>81</td>
</tr>
<tr>
<td>K1</td>
<td>Principal declination</td>
<td>Diurnal</td>
<td>0.6</td>
<td>312</td>
</tr>
<tr>
<td>O1</td>
<td>Principal lunar</td>
<td>Diurnal</td>
<td>0.45</td>
<td>263</td>
</tr>
</tbody>
</table>

It is assumed that discharge sluices will be built before gap closure. These sluices will lower the flow velocity at the gap during the gap-closing procedure. After completion of the dam, the discharge sluices will control the discharge in and out of the basin. The needed total cross-sectional area of the discharge sluices has been estimated in the MSc thesis of Ms. Sturm (Sturm, 2011). Three values were evaluated: 10,000 m$^2$, 20,000 m$^2$ and 30,000 m$^2$. These three different sluices were also used in the storage area approach. The depth of the discharge sluice has been estimated at 10 m.

The river discharge coming in from the Soi Rap mouth has been estimated on 5000 m$^3$/s, based on measured discharges from upstream reservoirs. This discharge is rather high, but later it will appear that the river discharge is very small compared to the incoming and outgoing discharges resulting from the tide. In a later stage, a design flood wave discharge could be tested for the closure method but it is assumed for now that the final gap closure will take place in the dry season.

The purpose of these calculations is to determine the expected velocities at the closure gap, so that the building strategy can be determined. When the final design has been made, more detailed hydraulic calculations can be carried out on the required storage area and gap closure velocities.

### 5.2.3 Results

For each simulation, the model has been run for the period of one month. Because of the cyclic character of tides, this is assumed to be sufficient for this rough design. The maximum (absolute) occurring velocity at the closure gap was then obtained from the model. Effects of peak discharges from upstream or storm surges were not taken into account. This can be done in a later stage when more detailed calculations are needed. As stated before, closure is assumed to take place in the dry season.

For the three different sizes of the discharge sluice, the resulting maximum velocities at the closure gap for horizontal closure of the dam are given in the next three graphs.
Figure 5.5: Velocity and discharges with discharge sluice area of 10,000 m$^2$

Figure 5.6: Velocity and discharges with discharge sluice area of 20,000 m$^2$
Figure 5.7: Velocity and discharges with discharge sluice area of 30,000 m$^2$

For the sluices of 20,000 m$^2$ and 30,000 m$^2$, the resulting velocities at the closure gap are nearly equal. Also the discharges at the closure gap and discharge sluice are almost equal. The velocity just before the point of closure is around 4 m/s. With a discharge sluice of 10,000 m$^2$, the velocity increases to around 5 m/s when the gap is almost closed.

It is assumed that sand closure can be applied up to a velocity of around 2.5 m/s, before it washes away (Verhagen et al., 2009). From Figure 5.5, Figure 5.6 and Figure 5.7 it follows that horizontal closure with sand can be applied up to a gap width of 1300 m for the two largest sluices, and up to 1800 m for the smallest sluice. The resulting maximum velocities for vertical closure were investigated for these widths and also for smaller gap widths, for the three different discharge sluices.

The results are given in Figure 5.8, Figure 5.9 and Figure 5.10.
Figure 5.8: Resulting velocity for vertical closure using $A_{\text{sluice}} = 10,000 \text{ m}^2$

Figure 5.9: Resulting velocity for vertical closure using $A_{\text{sluice}} = 20,000 \text{ m}^2$
Figure 5.10: Resulting velocity for vertical closure using $A_{\text{sluice}} = 30,000 \text{ m}^2$

5.2.4 Conclusions

When horizontal closure is applied, the maximum velocity in the closure gap reaches up to almost 4 m/s when a discharge sluice area of 20,000 m$^2$ or 30,000 m$^2$ is opened during construction. With a discharge sluice of 10,000 m$^2$, the maximum velocity reaches almost 5 m/s. For the first two sluices, horizontal closure with sand can easily be applied up to a gap width of 1300 m before the velocity becomes higher than 2.5 m/s. With a latter sluice, horizontal closure with sand can be applied up to a gap width of 1800 m.

With these velocities, the gap closure needs much attention in both the design and in the construction phase. It is in no way an easy operation, and the level of difficulty should not be underestimated.

The discharge from the Soi Rap does not influence the velocity in the gap very much. Most of the discharge in the closure gap and discharge sluice is caused by the tide.

Apparently, there is no significant difference in gap velocity and discharge between the discharge sluices of 20,000 m$^2$ and 30,000 m$^2$ during gap closure. A reason for this can be that the reservoir is able to ‘follow’ the tidal movement quite good with both sluices. Of course the dam is hindering the water from coming in or out of the reservoir, but the limitation the two sluices pose on the system behaviour appears to be equal. When the smallest discharge sluice is used, the gap velocity is significantly higher. This can be explained as follows:

The dam and sluices cause a delay between inside and outside water level changes. When the water level in the reservoir cannot keep up with the water level at sea, a head difference occurs between the reservoir and the sea. This head difference causes flow in the gap/sluice. The effect of a delayed water level is stronger with a smaller flow area of the sluice. The consequence is the head difference over the closure gap become larger and hence the velocities are higher.
It should be emphasized that the calculated velocities in the closure gap are maximum velocities. Much attention should be paid to the planning of the closure procedure, such that the final gap will be closed during a period which has a relatively small tidal range, such that the water velocity does not get too high. When the final design has been made, the design should be tested in more detail. Then for instance a 2-dimensional or a 3-dimensional mathematical model can be used.

5.3 Design

5.3.1 Material

5.3.1.1 Calculation using Shields relation

In appendix Q the required stone class during different phases of the final gap closure are calculated using the Shields relation. When the gap is narrower, the flow velocities are higher and therefore bigger grains are required. Results for the construction phase (stretches of 300 m), gap width, velocity, required median nominal diameter and stone class are displayed in Table 5.2.

<table>
<thead>
<tr>
<th>Phase</th>
<th>( W_{gap} ) (m)</th>
<th>( \bar{u} ) (m/s)</th>
<th>( D_n ) (m)</th>
<th>Stone class</th>
</tr>
</thead>
<tbody>
<tr>
<td>T0</td>
<td>1370 - 1070</td>
<td>2.6</td>
<td>0.11</td>
<td>80/200 mm</td>
</tr>
<tr>
<td>T1</td>
<td>1070 - 770</td>
<td>3.0</td>
<td>0.17</td>
<td>200/350 mm</td>
</tr>
<tr>
<td>T2</td>
<td>770 - 470</td>
<td>3.3</td>
<td>0.23</td>
<td>10-60 kg</td>
</tr>
<tr>
<td>T3</td>
<td>470 - 170</td>
<td>3.5</td>
<td>0.28</td>
<td>60-300 kg</td>
</tr>
<tr>
<td>T4</td>
<td>170 - 0</td>
<td>4.0</td>
<td>0.44</td>
<td>300-1000 kg</td>
</tr>
</tbody>
</table>

5.3.1.2 Reference projects

For the Saemangeum project (see appendix A) three final gaps were closed. In (Konter et al., 1992) calculation results were found for the required stone size in the final gap. These calculations were made using the Shields relation. A relative density of \( \Delta = 1.53 \) was assumed for these calculations. Also the stone sizes that were used in the NEDECO design are given. Results are presented in Table 5.3 until Table 5.4.
To determine the stone sizes $U_0$ was used for these calculations. $U_0 = U/\mu$; where $U$ is the velocity in the final gap and $\mu$ a discharge coefficient.

**Table 5.3: Gap 3; sill at MSL -5.5 m**

<table>
<thead>
<tr>
<th>Phase</th>
<th>Bs (m)</th>
<th>$U_0$ (m/s)</th>
<th>$D_N$ (m)</th>
<th>$D_N$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>In the NEDECO design</td>
</tr>
<tr>
<td>T0</td>
<td>1500</td>
<td>3</td>
<td>0.49</td>
<td>-</td>
</tr>
<tr>
<td>T1</td>
<td>1400</td>
<td>5</td>
<td>0.59</td>
<td>0.59</td>
</tr>
<tr>
<td>T2</td>
<td>800</td>
<td>6.25</td>
<td>0.88</td>
<td>1.05</td>
</tr>
<tr>
<td>T3</td>
<td>200</td>
<td>7.5</td>
<td>1.37</td>
<td>1.24</td>
</tr>
</tbody>
</table>

**Table 5.4: Gap 1; sill at MSL -6 m**

<table>
<thead>
<tr>
<th>Phase</th>
<th>Bs (gap 1 and 2 together) (m)</th>
<th>$U_0$ (m/s)</th>
<th>$D_N$ (m)</th>
<th>$D_N$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>In the NEDECO design</td>
</tr>
<tr>
<td>T0</td>
<td>2600</td>
<td>3.5</td>
<td>&lt; 0.33</td>
<td>-</td>
</tr>
<tr>
<td>T1</td>
<td>2600</td>
<td>6.25</td>
<td>0.85</td>
<td>-</td>
</tr>
<tr>
<td>T3</td>
<td>2600</td>
<td>6.75</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>T4</td>
<td>1700</td>
<td>7.25</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>T5</td>
<td>800</td>
<td>7.25</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>

**Table 5.5: Gap 2; sill at MSL -8 m**

<table>
<thead>
<tr>
<th>Phase</th>
<th>Bs (gap 1 and 2 together) (m)</th>
<th>$U_0$ (m/s)</th>
<th>$D_N$ (m)</th>
<th>$D_N$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>In the NEDECO design</td>
</tr>
<tr>
<td>T0</td>
<td>2600</td>
<td>&lt; 3</td>
<td>&lt; 0.33</td>
<td>-</td>
</tr>
<tr>
<td>T1</td>
<td>2600</td>
<td>6.0</td>
<td>0.49</td>
<td>-</td>
</tr>
<tr>
<td>T3</td>
<td>2600</td>
<td>6.5</td>
<td>0.88</td>
<td>0.59</td>
</tr>
<tr>
<td>T4</td>
<td>1700</td>
<td>7.25</td>
<td>1.3</td>
<td>1.0</td>
</tr>
<tr>
<td>T5</td>
<td>800</td>
<td>7.75</td>
<td>1.4</td>
<td>1.6</td>
</tr>
<tr>
<td>T6</td>
<td>200</td>
<td>7.75</td>
<td>1.4</td>
<td>1.8</td>
</tr>
</tbody>
</table>

It can be seen that the calculations that were made with the Shields relation are in line with the NEDECO design. For flow velocities of 3 to 5 m/s a stone size of 0.49 to 0.59 m was calculated.

5.3.1.3 Design grain sizes

The calculated stone size of 0.44 m for the final closure is smaller than the stone size that was used for the Saemangeum project. Flow velocities in this project were much bigger. For velocities of 3 to 5 m/s a stone size of 0.49 to 0.59 m was found for the Saemangeum project. Hydraulic parameters (e.g. water depth) are different.

These calculations were made for a full horizontal closure. If a partial vertical and partial horizontal closure is applied, flow velocities in the final gap will be higher than 4 m/s. Bigger stone sizes are required. The closure process however goes faster in this case so that material losses are less.

5.3.2 Bottom protection for the closure gap

5.3.2.1 Function

The function of a bottom protection in the vicinity of the final gap is to prevent scour, which occurs due to increased flow velocities near the final gap, to cause instability of the subsoil and the structure. The
bottom protection fixates the soil near the structure. The most important demands for the bottom protection are the following:

- Being sand tight: sand grains (and other bottom material) may not pass through the bottom protection
- Stability: the bottom protection must be able to resist the flow that passes over the protection layer
- Length: the bottom protection must prevent the structure from sliding into the scour hole. The bottom protection must therefore keep the scour hole at a sufficient distance from the structure. In addition erosion will reduce when the bottom protection is longer.

5.3.2.2 Type of bottom protection

The bottom protection should be permeable to drain excess water pressures. A watertight bottom protection is possible, but in that case one should take piping and the occurrence of high overpressures into account. For the VT-GC project, which is constructed on soft soils, the choice was made for a permeable bottom protection.

A permeable bottom protection can be constructed in situ or be prefabricated. If a prefabricated bottom protection is applied, on-site construction goes faster. The construction of the protection is influenced by flow velocities. It is best to construct the bottom protection in an early stage of the project, because the flow velocities are lower when less of the estuary is closed off. Because the final gap and therefore bottom protection will be located in the navigation channel, hindrance for shipping is less when a prefabricated bottom protection is applied.

Examples of prefabricated bottom protections were given in (CIRIA, 2007) and (Huis in ’t Veld et al., 1987):

- Loose ballast
  - Fascine mattresses with quarry stone
  - Fascine mattresses with jute bags filled with clay
- Fixed ballast
  - Block mats (geotextile with concrete blocks)
  - Mattresses with gravel tubes
  - Mattresses with stone asphalt

For all protections a sub layer (e.g. fascine mattress, geotextile) and a top layer (e.g. quarry stone, gravel tubes) is applied. The sub layer prevents the bottom material from being washed away through the pores of the material in the top layer. The properties of the sub layer are determined by the thickness and porosity of the top layer and the material size of the bottom sand and top layer. When the top layer consists of small grains it can be possible to construct the protection without a sub layer.

The use of locally available materials is preferred. Options with loose ballast are therefore favourable. In addition, special equipment is required for options with fixed ballast. Bamboo can be used to create fascine mattresses (see section 6.1.4.2). In a research from Bangladesh into using jute bags filled with clay as ballast material, a maximum flow velocity of $u_{max} = 2 \text{ m/s}$ for 50 kg bags was found. Because flow velocities in the final gap are higher, this is not a good solution for the VT-GC project.

A bottom protection made of loose rock is relatively simple to realize. The rock can be found near Ba Ria (see section 4.3.1). The necessary equipment is available almost everywhere.
5.3.2.3 Dimensions top layer

Calculation using Shields relation

The required length and stone sizes for the bottom protection were calculated in appendix R. For this calculation the construction period was divided into phases, each with a duration of two weeks. A construction speed of 150 m/week was estimated (section 6.3.3).

In Table 5.6 the phase, gap width, maximum velocity in the gap, required stone diameter and required stone class are displayed. It can be seen that when the final gap gets smaller, the required stone size in the bottom protection gets larger.

<table>
<thead>
<tr>
<th>Phase</th>
<th>( W_{\text{gap}} ) (m)</th>
<th>( \bar{u} ) (m/s)</th>
<th>( D_{n} ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T0</td>
<td>1370 - 1070</td>
<td>2.6</td>
<td>0.057</td>
</tr>
<tr>
<td>T1</td>
<td>1070 - 770</td>
<td>3.0</td>
<td>0.086</td>
</tr>
<tr>
<td>T2</td>
<td>770 - 470</td>
<td>3.3</td>
<td>0.11</td>
</tr>
<tr>
<td>T3</td>
<td>470 - 170</td>
<td>3.5</td>
<td>0.14</td>
</tr>
<tr>
<td>T4</td>
<td>170 - 0</td>
<td>4.0</td>
<td>0.21</td>
</tr>
</tbody>
</table>

These stone sizes are about 0.5 times the stone sizes that were calculated for the gap closure (see section 5.3.1.1).

The required length of the bottom protection was also determined in appendix R. It was found that on the landward side 45 m of protection is required. On the seaward side a bottom protection length of 60 m is necessary. This holds for soil that is not vulnerable to settlement flow (e.g. if a soil improvement is applied at the location of the bottom protection).

For soil that is vulnerable to settlement flow a bottom protection length of 110 m at the land side and 180 m at the sea side were determined.

Reference projects

In (Konter et al., 1992) calculation results were found for the stone sizes for the bottom protection of the Saemangeum project, see Table 5.7. These calculations were made using the Shields relation. A relative density of \( \Delta = 1.53 \) was assumed.

<table>
<thead>
<tr>
<th>Phase</th>
<th>Bs (gap 1 and 2 together) (m)</th>
<th>( U_{0} ) (m/s)</th>
<th>( D_{n} ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>2600</td>
<td>6.25</td>
<td>0.47</td>
</tr>
<tr>
<td>T3</td>
<td>2600</td>
<td>6.75</td>
<td>0.56</td>
</tr>
<tr>
<td>T4</td>
<td>1700</td>
<td>7.25</td>
<td>0.71</td>
</tr>
<tr>
<td>T5</td>
<td>800</td>
<td>7.25</td>
<td>0.71</td>
</tr>
</tbody>
</table>

It can be seen that these stone sizes are about 0.6 times the stone sizes that are required for the gap closure (see section 5.3.1.2).

A maximum depth of the scour hole of 22.4 m was calculated, together with a bottom protection length of 180 m. It was assumed that the bottom soil is vulnerable to settlement flow.

Design values

Stones with a \( d_{n,50} \) of 0.21 m were calculated for the final 170 m of the closure. The required length of the protection strongly depends on the subsoil. More research to subsoil conditions is necessary.
The length of the bottom protection is determined for the closure cross section. When the final cross section is made, it may not slide into the scour hole. The scour hole has to be filled with sand after the final gap is closed.

The calculations were made for a full horizontal closure. If a partial vertical and partial horizontal closure is applied, flow velocities in the final gap will be higher, which will increase the required stone sizes. The closure process however goes faster which results in a more shallow erosion hole. The final cap will be wider and therefore a bigger area of bottom protection is necessary.

5.3.2.4 Underlayer

Material from the bottom layer may not be washed away through the pores of the protection layer. In a geometrically open filter, some grains of the bottom can erode through the protection layer, but the critical gradient must remain below a certain value (Schiereck, 2001). A little bit of movement of the grains is allowed. A relatively large difference in grain size between the base and filter layer is now possible compared to a situation where no movement is allowed.

A situation with parallel flow through the base and filter layer applies. At the interface between the bottom and the protection a velocity gradient exists due to difference in porosity. This causes a shear stress on the grains at the top of the bottom layer. A thicker filter means that the sand grains of the bed have to travel a longer distance through the filter layer before they are being washed out. For provisional applications the formula of Wörman (1989) can be used. The required layer thickness depends on the grain size distribution of the base- and filter layer, the relative density of the grains in both layers and the porosity of the top layer. De Graauw (1983) found an empirical relation that can serve as a lower limit for calculations. In this relation the critical parallel gradient is calculated using grain size distribution of the base- and filter layer, the relative density of the grains in both layers, the porosity of the filter layer and the critical shear velocity of the base layer material (Schiereck, 2001).

The minimum layer thickness for construction is typically 0.5 m (CIRIA, 2007). If the required thickness of the protection is much larger or if the flow velocities are too large for a stable situation, an underlayer has to be applied between the bottom and the protection material.

Good options for an underlayer are:

- Geotextile: this can be prefabricated and sunk down on site (see chapter 6.1). When the top layer consists of large grains or if quarry stones are not largely available this option is favourable. An advantage of prefabrication is that construction on-site is faster (only one layer of rocks has to be dumped on the bottom protection).
- Full granular protection: the underlayer consists of layers of quarry stones with smaller stone sizes. In this case only natural material (rocks) are used. Several layers of stones have to be dumped. This may cause hindrance for shipping.
- No underlayer: this might be possible for the bottom protection with the smallest rock sizes. More research is required.

5.3.2.5 Recommendations

Slacks are due to their wide grading a better top layer material than quarry stones (Schiereck, 2001). The availability of slacks should be checked.

Many estimates were made for the calculation of the bottom protection length and stone size. It is recommended to determine some of the parameters more accurately in later design stages of this project. For example it is recommended to check the soil parameters that were used for this calculation.
in a laboratory or in a model test. When the dimensions of the discharge sluice have more accurate values, the velocities in the final gap can be determined more precisely. Also the exact geometry of the sea bottom and navigation channel can be incorporated in future calculations.
6 Building Method

The building method is one of the most important aspects in the construction of a dam. A dam can be theoretically and technically be well designed, but one must be able to construct it in reality as well. Therefore in this section the building method, building sequence, transportation and equipment will be discussed.

6.1 Building method of the dam with rock revetment

6.1.1 Offshore working conditions

The dam will be constructed offshore. Therefore the influence of winds, waves and tides have to be taken into account. According to the Rock Manual (CIRIA, 2007) for common types of equipment tidal current should not exceed maximum values of 1.5 – 2 m/s without special precautions. Because this velocity will not be obtained during the major construction process (only in the final gap this velocity will be reached) floating equipment can be used.

Also given in the Rock Manual are the maximum wave weights for rock dumping or rock placing vessels, for several types of vessels, see Table 6.1. From this information it can be concluded that dumping and placing of rocks is possible up to significant wave heights of respectively 1.5 and 1.0 m, roughly corresponding with wind speeds corresponding with 6 on the Beaufort scale (CIRIA, 2007). Because these conditions will not be always met the use of floating equipment will be possible most of the time.

For construction in the shallow parts (near Go Cong), land based equipment must be used, as the water depth is not sufficient for the use of barges.

Table 6.1: Maximum wave height for several types of rock placing and rock dumping vessels (CIRIA, 2007)

<table>
<thead>
<tr>
<th>Type of vessel</th>
<th>Size</th>
<th>$H_s$ limit for dumping</th>
<th>$H_s$ limit for placing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crane barge</td>
<td>60 x 20 + 150 t crane</td>
<td>0.80 m</td>
<td>0.60 m</td>
</tr>
<tr>
<td></td>
<td>40 x 15 + 75 t crane</td>
<td>0.65 m</td>
<td>0.50 m</td>
</tr>
<tr>
<td>Large excavator on barge</td>
<td>35 x 12 m + 70 t excavator</td>
<td>0.65 m</td>
<td>0.50 m</td>
</tr>
<tr>
<td>Side stone dumper</td>
<td>650 t</td>
<td>1.25 m</td>
<td>1.00 m</td>
</tr>
<tr>
<td></td>
<td>1400 t</td>
<td>1.50 m</td>
<td>1.25 m</td>
</tr>
<tr>
<td>Split hopper</td>
<td>800 t</td>
<td>1.50 m</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>2000 t</td>
<td>2.00 m</td>
<td>N/A</td>
</tr>
<tr>
<td>Fall-pipe barge</td>
<td>50 x 17.5 m</td>
<td>N/A</td>
<td>0.65 m</td>
</tr>
<tr>
<td>Fall pipe vessel</td>
<td>10000 t</td>
<td>N/A</td>
<td>3.50 m</td>
</tr>
</tbody>
</table>

6.1.2 Material transportation

A lot of material has to be transported to the construction site. The most important are the required sand (for the fill) and the rock (for the protection).

6.1.2.1 Sand

The sand needed for the core can be obtained near Vung Tau or to the south-east of Vung Tau (offshore), see section 4.3.1, by means of dredging. The transportation method of the dredged material depends on the distance from the source. Dredging and transportation options are:

- Use of floating pipelines and a fixed dredger
- Use of a dredger which can transport the material to the site itself
- Use of barges and a fixed dredger
In Figure 6.1 for several pipeline diameters the capacity as function of the pipeline length is given. One might see that even with a pipeline fairly large diameter of 0.8 m the maximum length lies in the order of several kilometres. It can be concluded that pipelines are only a viable option when the sand is closer than a few kilometres from the dam. Because the sand deposits are located further away the use of pipelines is not a viable option.

![Figure 6.1: Capacity versus pipeline lengths for several pipeline diameters (Matousek, 2004)](image)

Movable dredgers

Some dredgers, like a trailing suction hopper dredger, dredge the material and transport it to the site themselves. The capacity, draught and dredging depth of several suction hopper dredgers used by Boskalis (Royal Boskalis Westminster NV, 2011) are given in Table 6.2. The offshore dredging depth is approximately 10 – 25 m, so all ship sizes can dredge the offshore sand. However, it can be concluded that this type of transportation is not suitable because most of the dam is located in shallow water (above MSL -5 m). Big dredgers are required for the capacity, while only small dredgers can reach the site.

<table>
<thead>
<tr>
<th>Ship name</th>
<th>Capacity [m$^3$]</th>
<th>Maximum draught (loaded) [m]</th>
<th>Dredging depth [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Queen of the Netherlands</td>
<td>35500</td>
<td>13.7</td>
<td>67</td>
</tr>
<tr>
<td>Seaway</td>
<td>13255</td>
<td>10.6</td>
<td>57</td>
</tr>
<tr>
<td>Willem van Oranje</td>
<td>12000</td>
<td>10.0</td>
<td>62</td>
</tr>
<tr>
<td>Coastway</td>
<td>4900</td>
<td>5.7</td>
<td>28</td>
</tr>
<tr>
<td>Flevo</td>
<td>2130</td>
<td>4.6</td>
<td>27</td>
</tr>
</tbody>
</table>

Fixed dredger and barges

Therefore a stationary dredger, like a suction dredger, is preferable, and a lot of barges are required to transport the material. Depending on the sea conditions, a lot of small barges with a small draught can be used, or more seaworthy vessels which load the sand into smaller barges or pontoons close to the shore.

Some characteristics of barges and hoppers used by Boskalis are given in Table 6.3. The draught is just sufficient to navigate to most parts of the dam during high water levels (water level above MSL).

<table>
<thead>
<tr>
<th>Ship name</th>
<th>Type</th>
<th>Dimensions (h x w x d) [m]</th>
<th>Capacity [m$^3$]</th>
<th>Max. draught (loaded) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terrafere 501/502</td>
<td>Split hopper</td>
<td>90 x 16.6 x 7.2</td>
<td>3823</td>
<td>5.2</td>
</tr>
<tr>
<td>Terrafere 301/302</td>
<td>Split hopper</td>
<td>71.9 x 14.6 x 5.8</td>
<td>2302</td>
<td>5.2</td>
</tr>
</tbody>
</table>
The dam will have an estimated volume of 30 km (length) x 800 m² (approximate area of the cross section with bottom depth MSL -5 m) = 24·10⁶ m³. When we assume to have no material loss and using ships with a capacity of 1000 m³ (because of the draught) 24000 ship loads are required!

6.1.2.2 Rock

For the different filter layers, the underlayer and the armour layer of the slope protection, as well as the toe protection, quarried rock is used. Rock can be obtained near the project area, north and north-east of Vung Tau, see section 4.3.1. These sites can only be accessed by trucks and lorries.

The transport of rock from the quarry to the building site can be done in different ways (road, rail, water). For this case a combination of road and water transport will probably be the most efficient. This means transportation of the rock from the quarry to the Vung Tau harbour, and then transferring the rock from trucks to ships and transporting the rock to the building site. A lot of transportation equipment will be required.

6.1.3 Placement techniques

When the rocks and sand material are transported to the site it should be placed on the exact location. This can be done with different methods, see Figure 6.2.

1. Bottom dumping from a vessel (sand and rock)
   When the sea bottom has sufficient depth sand and rock can be dumped from a transportation vessel. A depth of 3.5 – 5 meters is required, because of the draught of a loaded dumping ship, see Table 6.3. This method is preferred because of the low complexity and the relatively low material loss. However, after a number of dumping steps the ships cannot navigate above the dam anymore and a different method should be applied. Material could be dumped from a pontoon with a low draught. Sand slopes attained under water are between 1:10 and 1:15 and an efficiency between 30 and 60% will be reached (Huis in ‘t Veld et al., 1987).

2. Rainbowing from a vessel (sand)
   Rainbowing is the ‘spraying’ of sand over some distance. The advantage is that the ship can stay next to the dam in the deeper water. Horizontal distances of 50 m could be reached. However, the material loss will be quite high. This method is especially useful to construct larges plains of sand, like working islands.

3. Placement with fall pipe (rock) or suction pipe (sand) vessels
   When more precise placement of the material is needed fall pipe vessels can be used. Although it takes more time to place the stones the loss of material is minimal. Sand slopes below the water table are between 1:4 and 1:6 and an efficiency of 60 to 80% could be reached (Huis in 't Veld et al., 1987).

<table>
<thead>
<tr>
<th>Ship name</th>
<th>Type</th>
<th>Dimensions (h x w x d) [m]</th>
<th>Capacity [m³]</th>
<th>Max. draught (loaded) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Avon &amp; Test &amp; Itchen</td>
<td>Split hopper / rainbow / pump shore</td>
<td>72.9 x 13.2 x 5.7</td>
<td>1650</td>
<td>5.1</td>
</tr>
<tr>
<td>Cork Sand &amp; Long Sand</td>
<td>Split hopper / rainbow / pump shore</td>
<td>65 x 11.8 x 4.3</td>
<td>1040</td>
<td>3.7</td>
</tr>
<tr>
<td>Frigg &amp; Rind</td>
<td>Split hopper / rainbow / pump shore</td>
<td>60 x 11.4 x 4.5</td>
<td>750</td>
<td>3.8</td>
</tr>
</tbody>
</table>
4. Side stone dumping from a vessel
Stones are transported on deck of the vessel and pushed overboard by moving beams. The loading capacity can be up to 1500 tons, while the dumping capacity is up to 60 or 70 tons per minute (Schiereck, 2001). The dumping is reasonably well controlled. Dumping can be done for water levels at MSL -5m or higher and takes about 11 minutes for 1300 tons (Tideway Offshore Contractors, n.d.).

5. Use of rolling equipment
Rolling equipment (like trucks and excavators) can be used to dump material from newly constructed dam parts. The dam should be extended from supply points. Disadvantages are the complicated logistics, the need of storage and the fact that construction is only possible at dam ends. A minimal required width is needed for trucks to drive. Two-way traffic is needed, because the working front of the dam will be a dead end. A minimum width of 7 meters is needed for two-way traffic (Verhagen et al., 2009). Alternatively a width of 4 meters (one-way traffic) can be used, when sufficient passing places are constructed. With bulldozers and other rolling equipment, a maximum slope of 1:5 can be constructed (Huis in 't Veld et al., 1987).

The use of waterborne equipment is preferable over the use of rolling equipment, because of the easier logistics and larger capacities.

Figure 6.2: Placement techniques
6.1.4 Dam construction sequence and method

Two different construction methods for the construction of the dam are distinguished: the ‘traditional method’ and the ‘alternative method’ (Verhagen et al., 2009), see Figure 4.48. In the first method first the complete core is constructed before placing the protection, whereas in the latter one the dam is constructed layer by layer. Although the alternative method gives a better protection of the fill during construction, more material for the first underlayer is required than strictly necessary, and the construction method is a little more complicated.

![Figure 6.3: Different construction sequences (Verhagen et al., 2009)](image)

<table>
<thead>
<tr>
<th>Traditional method</th>
<th>Alternative method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Filters</td>
<td>a. Filters</td>
</tr>
<tr>
<td>2. Core</td>
<td>b. First under-layer (part)</td>
</tr>
<tr>
<td>3. First under-layer</td>
<td>c. Core</td>
</tr>
<tr>
<td>4. Armour layer</td>
<td>d. First under-layer (part)</td>
</tr>
<tr>
<td>5. Crest (working back)</td>
<td>e. Core</td>
</tr>
<tr>
<td></td>
<td>f. Armour layer (part)</td>
</tr>
<tr>
<td></td>
<td>g. First under-layer (part)</td>
</tr>
<tr>
<td></td>
<td>h. Armour layer (part)</td>
</tr>
<tr>
<td></td>
<td>i. Crest (working back)</td>
</tr>
</tbody>
</table>

Because steep slopes of 1:3 to 1:4 are designed the alternative method will be preferable. For the traditional method a lot of extra material is necessary due to the gentle slopes and the lack of lateral containment. It should be noted that the designed underwater slope is quite steep compared to the values from literature, and that the construction of this slope will need much attention during the construction phase to be viable.

A schematic building process of the dam profile is given in Table 6.4. These phases are explained in more detail in the following sections.
Table 6.4: Construction sequence dam

<table>
<thead>
<tr>
<th>Stage</th>
<th>Element</th>
<th>Short description of the method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Foundation</td>
<td>Soil replacement by dredging, or accelerated consolidation with a preload and vertical drains</td>
</tr>
<tr>
<td>2</td>
<td>Scour protection</td>
<td>Dumping of stones with a side stone dumper</td>
</tr>
<tr>
<td>3</td>
<td>Subsequent layers of core material, in combination with filter layer, underlayer, toe and armour layer</td>
<td>Layer by layer constructing of the core and the several layers with equipment depending on the water depth and the reached height of the dam during construction</td>
</tr>
<tr>
<td>4</td>
<td>Crest</td>
<td>Cranes will place the underlayer and armour layer of the crest</td>
</tr>
<tr>
<td>5</td>
<td>Finishing</td>
<td>The dam will be finished by constructing the road and other additional structures</td>
</tr>
</tbody>
</table>

6.1.4.1 Stage 1: Foundation

The most suitable foundation depends on the thickness of the weak compressible layers. In the Vung Tau Weak-profile (weak clay until a depth of MSL ≈ -25 m) accelerated consolidation is the best method, whereas in the Vung Tau Strong-profile (weak clay until a depth of MSL ≈ -8 m) and the Go Cong-profile (weak mud until a depth of MSL ≈ -12 m) soil replacement is favourable. The construction method of both solutions will be treated in general hereafter. A schematisation of both methods is given in Figure 6.4.

![Figure 6.4: Foundation soil improvement techniques](image)

Accelerated consolidation

Increasing the settlements, and thus increasing the strength and bearing capacity of the soil, can be obtained by using vertical drains, either made of granular permeable material (sand, gravel) or PVC/geotextile drains. They can be installed from a floating pontoon with the use of equipment made for this purpose (Figure 6.5).

After installation loading can be applied by bottom dumping of dredged sand, see section 6.1.3. First a layer with a maximum thickness of a few meters is required (because the undrained bearing capacity of the soil is too low). When the weak layers have settled enough (a few per cent, see section 4.2.3.1) the undrained strength is large enough to apply more preloading. This pre-load must be placed for a duration of several years and must be protected to prevent erosion of the load.

After almost full consolidation has been reached the settlement gap can be filled up and the final dam profile can be built.
Soil replacement

In the case of soil replacement a lot of dredging capacity is required to excavate the weak top layer. The dredging material has to be transported and stored somewhere. Because of the high water content, slow consolidation and accompanying weak strength it is not likely to be of use elsewhere in the project, so large storage basins have to be constructed. Quickly after dredging the hole has to be filled with sand, to minimise silting up of the gap. Large amounts of sand are required.

To estimate the required time for this process an excavation volume with a length of 30 km, a depth of 5 meters and width of 100 m is assumed (15 million m$^3$). With a dredging ship capacity of 3000 m$^3$/hour, see Figure 6.1, it takes 5000 hours for one ship to dredge all material, assuming no siltation takes place. So when two dredgers are assumed, a 10 hour working day and 5000 hours extra needed to fill the hole again the whole process could take almost 1.5 years.

6.1.4.2 Stage 2: Scour protection

A scour protection is necessary to prevent erosion and undermining of the toe of the structure. Grains will be dumped on the bottom with a side stone dumper or a crawler crane where the water is to shallow to operate with a side stone dumper.

6.1.4.3 Stage 3: Subsequent layers of filter layer, toe protection, underlayer, core and armour layer

The construction sequence of the first layers and the toe can be seen in Figure 6.8. When the toes at the inner and outer side are placed the volume in between can be filled with sand. The best method to build
this part of the dam depends mainly on the available water depth, because waterborne construction is preferred (see section 6.1.3) and the water depth is the limiting factor.

**Waterborne construction**

The construction of a granular filter is complicated since the filter has to be built up layer after layer. To ensure the sand-tightness, it is unacceptable that layers are interrupted locally. In view of the tolerances on one hand and the measuring accuracy on the other hand, a layer thickness of at least 0.5 m or $2d_{50}$ is used for the filter layers so they can be barge-dumped (Verhagen et al., 2009).

The dumping of the filter layers and underlayer will be done by side stone dumping barges that create a sort of curtain of filling material with which they can make relatively thin rock layers (Verhagen et al., 2009).

The dumping of the core material will be done using split barges that can transport large quantities of material and dump this quickly on a concentrated spot.

Layer by layer the first filter layer, core material, other filter layers, underlayer and armour layer can be constructed. The armour layer will be placed using a crane, for the first layers operating from a pontoon. Using a side dumping barge to dump the armour layer is not a good option, because the dumping of the stones cannot be controlled precisely. Constructing a two-layer revetment with a barge is not possible.

**Land-based construction**

Further in the process, the area where the dam is constructed will be too shallow for a barge to sail over. Working from pontoons near the dam structure or rainbowing from ships near the dam will take over from the barges. The same procedure has to be followed: constructing new rock protection bunds at the inner and outer side of the dam, dumping the sand core till the ridge of the bunds and placing the filter layers. This sequence should be repeated until the dam surfaces.

Rainbowing can only be used for the sand fill material and more research is needed to the amount of material that goes to waste using this method. Working from pontoons with cranes will be a good method to construct the toes and the filter layers, because the construction can be done more precisely.

After surfacing the steeper sand slopes can be built with rolling equipment, see section 6.1.3. Now the core material is constructed first and the protection layers will be made afterward. (Huis in ’t Veld et al., 1987) advise a slope of 1:5 for sand constructions above water. It could be beneficial to choose a gentler slope for the final design of the dam. More core material will be used in that case, but construction is easier and dimensions of protection material can be smaller. More research into material use and
construction is advised. Required material can be either transported over the newly constructed dam, or over the water to a nearby work island and transferred to trucks there.

A schematisation of the several phases is presented in Figure 6.11.

![Figure 6.11: Schematised view of the construction method of the dam core and layers.](image)

6.1.4.4 **Stage 4: Crest**

The crest will be constructed from the construction zone of the crest itself. Therefore it is necessary to work in the ‘opposite’ direction, from the middle of the closed dam backward to the land. Crawler cranes will position the underlayer and armour layer. The necessary material will be delivered by trucks.

The crest can also be constructed from the road on the inner berm using cranes. To make sure that the road is not damaged by the construction equipment only the sublayer should been constructed. The asphalt can be placed when the crest is finished.

6.1.4.5 **Stage 5: Finishing**

After the dam profile and protection have been built the permanent road on the inner berm of the dam will be bituminised. The dam can be finished and opened for use.

6.2 **Building method of the dam with asphalt layer**

This section will describe in short the building method of an asphalt armour layer on a dam. The major part of the construction of this alternative is the same as for the dam with the rock armour layer. Only the differences will be described in this section.

Distinction is made between applying asphalt products in hydraulic works under water and above water, in the dry or in the wet. In general, greater control and accuracy can be achieved with placement above the water. Some techniques can only be used above water whereas others are more suitable for under water use.

Extra care must be taken when placing asphalt in the tidal zone. In this respect asphalt should not be placed on a slope as long as the water-level in the dam body is causing a flow of water out of the slope. This would cause construction problems because of the deformation of the subsoil, damage to the asphalt and, possibly, uplift pressures.
6.2.1 Type of asphalt material

From the different asphalt types and construction techniques described in (Van de Velde, 1984), dense stone asphalt and mastic seem to be best applicable. These types of asphalt can be constructed above and below the water line.

6.2.1.1 Dense stone asphalt

**Mixture**

Dense stone asphalt is a gap-graded mixture of stone, sand, filler and bitumen (Van de Velde, 1984). The amount of bitumen slightly overfills the mixture. The material is therefore water impermeable.

**Transport**

Preparation of the asphalt mixture will be done in a mix-plant. Transport from this mix-plant to the construction site will be done by (covered) asphalt containers. Transport forms a large part of the total placing costs and on large projects it may be profitable to locate the asphalt mix-plant close to the construction site (Van de Velde, 1984). This is an option for the VT-GC dam.

Temporary storage of the mix on site for a long duration and of large quantities, due to for example interruptions in operations, must be avoided. Dense stone asphalt should be kept in asphalt containers if it has to be stored temporarily. Remixing, for example using the bucket of a crane, is then essential and should be carried out carefully.

**Application**

Dense stone asphalt can be placed with buckets, a crane or directly from the means of transport. The application temperature, above water, is in the range of 100 to 190 °C, below water the maximum temperature is 130 °C.

6.2.1.2 Asphalt mastic

**Mixture**

Mastic is a mixture of sand, filler and bitumen. There is more bitumen available than necessary for filling the voids in the sand filler mixture. The mix, therefore, is naturally dense and need not be compacted.

**Transport**

The asphalt mastic must be transported in a stirring-kettle, to be able to keep the mastic at a homogenous composition and temperature. The mastic should be stored on site in a stirring-kettle as well.

**Application**

Mastic can be poured at working temperatures and is used, for asphalt slabs above and under water for lining or as bed and toe protection. Equipment to use: a chute or pipe, crane bucket (above water level) or bottom-opening bucket suspended from a crane, or a dosing bucket running on tracks.

The maximum slope on which mastic can be applied is largely dependent on the mix composition, especially the proportion of bitumen. Maximum slopes for normal viscosity mixes are 1:7 (under water) to 1:10 (above water).

When cold, mastic forms a viscous quasi-static mass.
6.2.2 Dam construction sequence and method

Before the asphalt layer can be applied, the sand fill has to be placed and maintained in the correct place. There are two options for the pre-asphalt building method of the dam. The first is almost the same as the building method described in section 6.1.4 and results in the same cross section. The second is different and will lead to a cross section with a gentler slope under the water line.

6.2.2.1 Construction method using dense stone asphalt

The construction method of the asphalt dam is similar to the rock revetment dam (as described in section 6.1.4). First a scour protection will be applied. Sand will be dumped between the scour protections. The sublayer of the asphalt will be used to make construction bunds. Layer by layer sublayers, core and asphalt will be constructed. A crane will be used to place the asphalt, partly from pontoons and partly from the dam body.

6.2.2.2 Construction method using mastic

The sand fill will be dumped on the sea bottom, unconstrained by toes and not protected by a protection layer. This sand body will be very gentle sloped, in the order of a 1:15 slope under water. Above the water line a steeper slope of about 1:5 will be possible, using trucks and bulldozers to move the material.

On top of the sand layer first an underlayer for the asphalt mastic has to be constructed. This can be done in a similar way as the underlayer for the armour revetment is made: using side dumping barges for the deeper part, and then switching to pontoons with cranes for the shallower area.

On this filter layer the mastic layer can be constructed using for example a bottom-opening bucket suspended from a crane on a ship for the deep part. For the shallow part and above the water level, a crane bucket from a pontoon or the dam body can be used.

The advantage of this construction method is that it is much easier than constructing layer by layer. A disadvantage is that more core material has to be used. A more gentle foreshore and slope result in a lower run-up and therefore possibly a lower crest. More research is advised for the correlation between slope, run-up, required material and building method.

6.3 Building method closure gap

For the closure of the final gap, the following strategies can be applied:

- Horizontal closure: narrowing the gap from the sides
- Vertical closure: heightening of the sill from the bottom up
- Combined closure: a combination of the latter two
- Sudden closure with concrete caissons

The closure methods have already been discussed in detail in section 5.2. The next section focuses on the building method of a sand/stone closure.

6.3.1 Sand closure

The building method for the final gap is different from the construction of the rest of the dam. The main reason for this is that the velocities go up when the closure gap is being narrowed. The higher velocities cause more scouring, which leads to loss of material. The loss of material can be divided into gross loss and net loss. Gross loss is the sand that deposits outside the closure dam profile and if of importance for the time involved in the closure operation. The net loss is the sand taken beyond the profile of the final
dam and determines the actual loss (CIRIA, 2007). The ebb flow will be normative, because then the velocity in the gap is the largest. By shifting the axis of the closure dam in the direction of the largest velocity, the net loss can be minimized, see Figure 6.12.

![Image: Figure 6.12: Sand loss of closure dam (CIRIA, 2007)]

The height of the crest of the closure dam depends on the tide, wind setup and run up. Overtopping during construction will erode the dam and should therefore be kept to a minimum. The closure operation should ideally take place in a period when the tidal range is at its minimum, and the weather is expected to be calm. The width of the crest during construction is important for sand closure. A minimum crest width is favourable with respect to (after (CIRIA, 2007)):

- Reducing net loss. The larger the difference in cross section between the final dam and the closure dam, the smaller the net loss will be.
- Reducing the gross loss. A closure dam with a small crest needs a smaller sand volume during the closure operation. For this reason the time involved during the critical stage of the operation will be shorter and so the gross loss of sand will be reduced.

With a wider crest, more working space is available for equipment. For instance for horizontal closure with trucks, some space is needed for trucks to pass each other. This can be achieved by making the crest wide enough (7 m), or by creating some passing spaces (then 4 m is sufficient).

Application of sand by means of pipes or trucks has been discussed in section 6.1.3. It was concluded that sand cannot be applied by pipes, since the source of sand is too far away which makes it impossible to pump the sand to the building site.

A crest width of 7 m is assumed, so that trucks can pass each other.

### 6.3.2 Construction sequence and method

The construction sequence depends on the chosen construction method (horizontal / vertical / combined closure or sudden closure with caissons). Which construction method is best depends on:

- The expected velocities in the closure gap, which are dependent of the bathymetry, tide, upstream discharges etc.
- The amount of expected sand losses, resulting from the velocity in the closure gap
- The possibilities to apply building materials by either truck/barge/dredger/pipe. See section 6.1.2
- The time duration of gap closure

The mentioned construction methods have already been discussed in section 6.1.4. The following building sequence is recommended for the closure gap:

- Constructing the bottom protection at the location of the final gap (described in section 6.3.2.1)
- Heightening of the sill with waterborne equipment (e.g. barges) until the water gets too shallow (this will be around MSL -5 m) and/or the velocity gets too high for the barges to manoeuvre (see section 6.1.1.
- Horizontal closure with trucks. Building material will be transported to the sides of the closure gap by trucks, see Figure 6.13. The last section of the dam will be built during a period with a small tidal range and in the dry season (low river discharge, avoid monsoons). The final closure will take place during neap tide.
- Finishing the dam construction with sand core, toe protection, filter layers and armour layer, similar to the construction of the dam.

![Combined vertical and horizontal closure](image)

*Figure 6.13: Horizontal and vertical closure of the final gap*

Based on the calculated velocities in the closure gap from section 5.2 the diameter of the rock for different gap widths is calculated in appendix Q. The following sequence of stone classes is recommended for the final closure gap, see Table 6.5. It is concluded that closure with sand is possible up to a gap width of ±1400 m. The final gap will be closed with increasing stone sizes, in stretches of 300 m. This calculation is made for a full horizontal closure. If the sill is heightened to a level of MSL -5 m first, the final gap where construction with sand is not possible will be wider and construction stretches will be longer.

<table>
<thead>
<tr>
<th>Phase</th>
<th>(W_{gap}) (\text{m})</th>
<th>(\bar{u}) (\text{m/s})</th>
<th>(D_{50}) (\text{m})</th>
<th>Stone class</th>
</tr>
</thead>
<tbody>
<tr>
<td>T0</td>
<td>1370 - 1070</td>
<td>2.6</td>
<td>0.11</td>
<td>80 / 200 mm</td>
</tr>
<tr>
<td>T1</td>
<td>1070 - 770</td>
<td>3.0</td>
<td>0.17</td>
<td>200 / 350 mm</td>
</tr>
<tr>
<td>T2</td>
<td>770 - 470</td>
<td>3.3</td>
<td>0.23</td>
<td>10 - 60 kg</td>
</tr>
<tr>
<td>T3</td>
<td>470 - 170</td>
<td>3.5</td>
<td>0.28</td>
<td>60 - 300 kg</td>
</tr>
<tr>
<td>T4</td>
<td>170 - 0</td>
<td>4.0</td>
<td>0.44</td>
<td>300 - 1000 kg</td>
</tr>
</tbody>
</table>

Because there is no flow after the finishing of the final dam construction, the slope of the sand under water without protection can be steeper. This is favourable for the use of the traditional construction method (section 6.1.4). A slope of 1:4 is however probably still too steep for underwater construction with the traditional method.

6.3.2.1 Construction of the bottom protection

In (CIRIA, 2007) construction methods are described for several types of bottom protections. In this section the construction of a full granular protection and a protection with geotextile will be described.

Full granular protection

If only granular material is used for the bottom protection, a side stone dumping vessel can be used. Discontinuities in the bed protection can be prevented by: moving the vessel over the section to be covered, while uniformly dumping stones, constructing each layer from multiple thin layers to correct discontinuities from previous dumps, dumping from both sides of the vessel and by creating an overlap
between the sections to be covered. The minimum layer thickness is 0.5 m for construction reasons. With more advanced equipment (e.g. trailing suction hopper dredgers and bed levellers) thinner layers can be constructed.

Strong currents can segregate the falling stones, depending on gradation and water depth. When current velocities are higher than 0.5 m/s, dumping must be done carefully or the construction has to be stopped until calmer conditions apply.

Wave action corrects discontinuities during stone dumping but affects the positioning accuracy of the vessel. The vessel operability is the limiting criterion for wave action, see section 6.1.1.

The first layers in a full granular bed protection will be fine and should be placed in limited wave and current action. The bed protection in the final gap of the VT-GC project can be constructed right after the bed protection for the discharge sluices and ship locks is finished. The same equipment, experts and experienced workers can be used for construction. The dam has not yet been constructed and therefore the current velocities in the final gap are as low as possible. Shipping delays may occur as the bottom protection works take place in the navigation channel. Not the entire channel has to be closed off at the same time.

Geotextile covered by armour stones
Before placing the geotextile everything that may cause damage (stones, tree roots et cetera) must be removed from the sea bottom. The geotextile can be placed on the sea bottom by different methods. The method that is considered most suitable for the VG-GC project for equipment and material reasons is described hereafter.

A filter mattress is created by connecting the geotextile to a grid of bamboo stiffeners. The bamboo prevents the geotextile from folding and increases the buoyancy. The mattresses are created above water, preferably near the construction site. On the work islands that are created for the construction of the ship locks and discharge sluices, space should be made available to create the mattresses. The geotextile is unwound and the stiffeners are attached. After this, the geotextile is carefully pulled to the position where it will be sunk down.

A sinking beam and a tail beam are connected to the mattress to position it and to sink it down properly. The sinking beam is lowered such that a side stone dumper can dump stones on the mattress. Light stones are dumped onto the filter mattress to sink it. 1 – 10 kg or 5 - 40 kg per m² is considered suitable. The stone sizes that were calculated can be dumped onto the geotextile directly. The bamboo stiffeners prevent the stones that are dumped onto the mattress from rolling down to the sea bottom when ballasting.
6.3.3 Duration of gap closure

For the Saemangeum dam, estimates were done for the duration of the final gap closure. Narrowing the closure gap with 600 m would take approximately two weeks, for a sill at MSL -5.5 m. Two gaps with a sill at MSL -6 m and MSL -8 m were closed at the same time with 900 m in two weeks. Averaged, that means a sill at MSL -7 m with a closure speed of 450 m per two weeks. With this information it was estimated that an additional depth of 1.5 m means a reduction in construction speed of approximately 150 m per two weeks.

For the VT-GC dam the sill is located at MSL -9.5 m. Following the trend of the Saemangeum project this would lead to a construction speed of 250 m in two weeks. However, the maximum flow velocity in the final gap of the VT-GC dam is lower than for the Saemangeum project. (For that project flow velocities over 7 m/s in the closure gap were calculated). Therefore the construction speed is estimated somewhat higher: 300 m in two weeks. The total closure of the final gap then takes 9 weeks.

6.3.4 Planning of gap closure

The final stage of the closure of the VT-GC dam calls for careful planning. In section 5.2 it was calculated that the expected maximum velocities will be high. By planning the closure procedure in the dry season (with low river discharge, good working conditions and smaller change of occurrence for monsoons) and the final closure during neap tide, the head difference over the closure gap will be the smallest and the velocities the lowest. Figure 6.15 shows an example of a good point during the tidal cycle for the final gap closure. The tidal range is smallest in the few days that are highlighted with a red rectangle. A detailed 2-D hydraulic model (like SOBEK or MIKE11) of the hydraulic system, together with an exact tidal prediction can be very helpful for planning the exact time/timing of the closure. This needs to be done in a later stage of the design.
6.4 Building method navigation locks and discharge sluices

The proposed global sequence to construct both the navigation locks and the discharge sluices is the following:

<table>
<thead>
<tr>
<th>Stage</th>
<th>Short description</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bottom protection</td>
<td>Bottom protection is needed to prevent scour when sluices and locks are in function. For the construction method, see the construction of the bottom protection for the final gap, section 6.3.2.1.</td>
</tr>
<tr>
<td>2</td>
<td>Building pit</td>
<td>To be able to construct the concrete structures a building pit is needed to construct them in situ. This makes for an easier (and probably cheaper) construction than ‘wet’ construction with prefab elements. (See for an example of this method the construction of the construction of the sluices in the Afsluitdijk (The Netherlands, appendix A) Dredgers and barges will be needed to transport sand to the side to build soil ring dikes and an artificial ‘polder’. Continuous pumping makes for a dry pit.</td>
</tr>
<tr>
<td>3</td>
<td>Foundation</td>
<td>Pile hammering is used to install the required pile foundation for the structures. Prefab piles and equipment can be supplied by ship, because both pits are next to deeper waters.</td>
</tr>
<tr>
<td>4</td>
<td>Structure</td>
<td>The locks and discharge sluices can be built in the dry pit with in situ casted concrete. Required equipment and material can be supplied by ship, because both pits are next to deeper waters.</td>
</tr>
<tr>
<td>5</td>
<td>Removal building pit and opening of the gates</td>
<td>When the construction of the structure has finished the building pit must be removed and the gates must be opened to simplify the final closure.</td>
</tr>
</tbody>
</table>

6.5 Construction sequence

6.5.1 Points of attention

The building of such a large closure is a complex process. A short overview of the required points of attention, according to the Rock Manual, is given in Figure 6.16.
6.5.2 Schematisation

The construction sequence of the construction of the dam is schematized in Figure 6.17. Four major building phases are distinguished. These construction phases are worked out below.

- **Stage 0:** Initial situation
- **Stage 1:** Preparation and construction of the bridge
  - Construction of the bridge: the bridge is built first to be able to use it in later stages for transportation of materials by road. This is necessary because most areas are too shallow for transport by ship
  - Construction of a work islands, by means of rainbowing. These islands are located in shallow water or next to the building pits or the bridge
  - Construction of building pits for the discharge sluices and navigation locks, as described in section 6.4
- **Stage 2:** Construction
  - Construction of discharge sluices and navigation locks in the dry building pits
  - Construction of dam parts, as described in section 6.1
- **Stage 3:** Construction
  - Removal of the building pits for navigation locks and discharge sluices, and opening of the discharge sluice gates
  - Completion of dam parts
  - Dredging of a new navigation channel to the navigation locks, to enable shipping during the final closure
- **Stage 4:** Final closure
  - Final closure in the Soi Rap navigation channel, as described in section 6.3
Stage 0
- Work island
- Building pits
- Bridge
- Vung Tau
- Can Gio
- Go Cong
- Navigation channel
- Deeper water, bottom depth < MSL -7.5 m

Stage 1
- Bridge
- Building site
- Deeper water

Stage 2
- Discharge sluices
- Locks

Stage 3
- Dredging

Stage 4
- Closure

Figure 6.17: Schematic representation of the building sequence
In this chapter an estimation of the costs of the dam will be made. This analysis will not be performed in much detail, because the project is still in the prefeasibility phase. A general estimation of the cost will be made, based on costs in reference projects.

In general the total costs can be divided into:

- Planning and engineering costs
- Material costs
- Labour and equipment costs
- Costs for implementation in the environment
- Management and maintenance costs

Ms. Olga Sturm performed a study for the costs of several barrier alternatives downstream of HCMC (Sturm, 2011). She looked at dam construction costs, discharge sluice construction costs and operational and management costs. In addition, data from the Feni closure in Bangladesh is obtained, see Table 7.1.

<table>
<thead>
<tr>
<th>Dam</th>
<th>Source</th>
<th>Length [m]</th>
<th>Height [m]</th>
<th>Depth [m]</th>
<th>Costs [M€]</th>
<th>Costs [M€/km, 2009]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saemangeum closure</td>
<td>(Sturm, 2011)</td>
<td>4694</td>
<td>20</td>
<td>10.2</td>
<td>52700 M KRW</td>
<td>7.44</td>
</tr>
<tr>
<td>Saemangeum closure</td>
<td>(Sturm, 2011)</td>
<td>9936</td>
<td>35</td>
<td>9.6</td>
<td>754400 M KRW</td>
<td>50.28</td>
</tr>
<tr>
<td>Saemangeum closure</td>
<td>(Sturm, 2011)</td>
<td>2693</td>
<td>16</td>
<td>8.5</td>
<td>37800 M KRW</td>
<td>9.31</td>
</tr>
<tr>
<td>Saemangeum closure</td>
<td>(Sturm, 2011)</td>
<td>11436</td>
<td>20</td>
<td>1</td>
<td>534800 M KRW</td>
<td>30.97</td>
</tr>
<tr>
<td>Saemangeum total</td>
<td></td>
<td>28759</td>
<td></td>
<td></td>
<td>913.74</td>
<td>31.77</td>
</tr>
<tr>
<td>Feni Closure</td>
<td>(Linham &amp; Nicholls, 2010)</td>
<td>1200</td>
<td></td>
<td></td>
<td>38 M$</td>
<td>23.7</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>29959</td>
<td></td>
<td></td>
<td>942.14</td>
<td>31.44</td>
</tr>
</tbody>
</table>

The estimation matches with a IPCC CZMS study in the Netherlands, which estimated a dam cost value of 24.3 – 40.6 M€/km (2009 price level). All values are total building cost values, including planning and engineering, material, labour and equipment and implementation costs.

For the costs of the discharge sluices an estimation of 23900 €/m³ is made, based on (Sturm, 2011). The reference projects she used are listed in Table 7.2. To find the costs for the discharge sluices this value has to be multiplied by the length, the depth and the head difference.
Operational and maintenance costs are estimated as 0.5% of the investment costs per year. This percentage is based on the costs of the Thames barrier (UK), the Eastern Scheldt barrier (The Netherlands) and the Maeslant barrier (Sturm, 2011).

The costs of the bridge are estimated with a typical cost of 175$ per square feet (for a high long span bridge) derived by the Florida Department of Transportation (Florida Department of Transportation, 2011). Assuming a 5 km long and 18 m wide bridge gives a value of 583 M$ (435 M€).

When the costs of the ship locks are neglected the total costs for the designed dam can be calculated. The result is presented in Table 7.3. The total costs of the dam are estimated 3.1 billion US dollar (2.3 billion €).

---

<table>
<thead>
<tr>
<th>Name barrier</th>
<th>Type</th>
<th>Year</th>
<th>Width [m]</th>
<th>Height [m]</th>
<th>Head [m]</th>
<th>Construction costs [M€]</th>
<th>Construction costs 2009 price level [M€]</th>
<th>Unit cost prices per cubic meter [1000 €/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hartel barrier (Hartel channel)</td>
<td>Vertical lifting gates</td>
<td>1991</td>
<td>170</td>
<td>9.3</td>
<td>5.5</td>
<td>98⁴</td>
<td>143</td>
<td>16.45</td>
</tr>
<tr>
<td>Eastern Scheldt Barrier</td>
<td>Floating sector gate</td>
<td>1991</td>
<td>360</td>
<td>22</td>
<td>5</td>
<td>450¹</td>
<td>656</td>
<td>16.57</td>
</tr>
<tr>
<td></td>
<td>Vertical lifting gates</td>
<td>1986</td>
<td>2400</td>
<td>14</td>
<td>5</td>
<td>2500³³</td>
<td>4021</td>
<td>23.93</td>
</tr>
<tr>
<td></td>
<td>Vertical lifting gates</td>
<td>1996</td>
<td>240</td>
<td>8.2</td>
<td>4.4</td>
<td>100</td>
<td>132</td>
<td>15.24</td>
</tr>
<tr>
<td>The Netherlands Maeslant barrier (New Waterway, Rotterdam)</td>
<td>Flap gates</td>
<td>2010</td>
<td>3200</td>
<td>15</td>
<td>3</td>
<td>4678</td>
<td>4678</td>
<td>32.49</td>
</tr>
<tr>
<td></td>
<td>Sector gates</td>
<td>1998</td>
<td>360</td>
<td>8.5</td>
<td>3.8</td>
<td>290</td>
<td>368</td>
<td>31.65</td>
</tr>
<tr>
<td></td>
<td>Sector gates</td>
<td>1980</td>
<td>530</td>
<td>17</td>
<td>7.2</td>
<td>800</td>
<td>1449</td>
<td>22.34</td>
</tr>
<tr>
<td></td>
<td>Vertical lifting gates/sector gates</td>
<td>2010</td>
<td>130</td>
<td>8</td>
<td>4</td>
<td>114.7⁵</td>
<td>115</td>
<td>27.64</td>
</tr>
<tr>
<td></td>
<td>Sector gates</td>
<td>2010</td>
<td>250</td>
<td>12</td>
<td>6</td>
<td>518³</td>
<td>518</td>
<td>28.78</td>
</tr>
</tbody>
</table>

Remarks:
1. Maeslant barrier has a relatively low cost price due to heavy competition for the contract.
2. The Hartel barrier has one very large horizontal span which increased the cost price.
3. The Eastern Scheldt barrier is relatively inexpensive due to its repetitive character.
4. The Seabrook barrier (New Orleans) has two different types of gate in a small span.
5. From the IHNC/St. Bernard storm surge barrier only the parts containing the gates have been taken into account, the floodwall was excluded.
### Table 7.3: Estimation of total costs based on reference projects

<table>
<thead>
<tr>
<th>Type</th>
<th>Price per unit</th>
<th>Units</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction dam</td>
<td>31.44 M€/km</td>
<td>30 km</td>
<td>943 M€</td>
</tr>
<tr>
<td>Construction discharge sluices</td>
<td>23900 €/m³</td>
<td>10000 m² (discharge area) x 3 m (estimated head difference) = 30000 m³</td>
<td>717 M€</td>
</tr>
<tr>
<td>Operation and maintenance dam</td>
<td>5% of construction costs / year</td>
<td>50 year</td>
<td>236 M€</td>
</tr>
<tr>
<td>Construction of bridge</td>
<td>435 M€</td>
<td></td>
<td>435 M€</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>2331 M€</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>≈ 3100 M$</td>
</tr>
</tbody>
</table>
8 SIDE PROJECTS

In the project not only design of the cross section and closure gap are treated, but also some side projects. These include a study into the salt intrusion in the Van Co River, as well as looking into a SOBEK-model and measurement. The results will be discussed in the following sections.

8.1 Salt intrusion in the Vam Co River

8.1.1 Introduction

An estuary can be described as the area where a river meets the sea. Estuaries are an unique environment, with characteristics of both a sea and a river. Estuaries have always been important to mankind, because they are a source of food, and a transport link between the river and sea (Savenije, 2005). Some of the most densely populated areas of the world are situated in estuaries. Salt intrusion can cause damage to society due to its need for fresh water for agriculture, industry and individual households. This is also the case in HCMC.

The canals and rivers near HCMC are directly connected to the East Sea and are therefore affected by both the tide and river discharges. Besides regular flooding, salt intrusion also causes serious problems around HCMC. During the dry season, salt intrusion can cause problems at the intake of HCMC’s drinking water treatment plant, upstream of HCMC. Therefore, two upstream reservoirs regularly release water during the dry season (February - April) to push the salt back. The Dau Tieng reservoir, built for irrigation purposes, releases about 20 - 22 m$^3$/s into the Saigon River. The Tri An reservoir, built for electricity generation, releases about 200 m$^3$/s into the Dong Nai River (IGES, 2006).

The two branches of the Vam Co River, the Vam Co Dong and the Vam Co Tay, are however still susceptible for salt intrusion. These branches flow through the area known as the Dong Thap Muoi area (plain of reed) which is intensively used for agriculture. In past dry seasons, saline water intruded up to 120 - 140 km upstream (MONRE, 2010), posing serious threats to farmers. The impact of climate change on the river discharges is likely to be complex, and the response may vary across the river basin (Vietnam-Netherlands Cooperation, 2011). (UNDP, 2007) reported that “in the whole Mekong Delta a 1-metre rise in sea level would affect approximately 5 per cent of Viet Nam’s land area, affect 11 per cent of the population, impact on 7 per cent of agriculture, and reduce GDP by 10 per cent.” The combination of a higher sea level and lower fresh water discharges (partly due to freshwater extractions) will only make the salt intrusion problem worse. Without further measures the salt intrusion will continue causing economic and ecological damage.

8.1.2 Description of the study area

The Dong Thap Muoi area is part of the Eastern Mekong Delta and is very important for the agriculture. Rice is the main crop, although fruit trees and aquaculture are also present. In recent years, farmers moved to aquaculture farming (shrimps, fish, oysters, scallops) during parts of the year when rice production was not optimal due to the occurrence of brackish water. The increase in shrimp farming contributed to the food supply and employment, but also led to fresh water shortages for both rice and shrimp production (MONRE, 2011) since the production process requires large amounts of fresh water. It has also been observed that aquaculture is contributing to the deterioration of the water quality.

Project Flood Defence HCMC | Side projects
An extensive irrigation system brings water from the Vam Co Tay and Vam Co Dong towards the fields. Both branches originate in Cambodia and flow east through the Mekong Delta. The size of the catchment areas of both rivers are given in Table 8.1.

Table 8.1: Catchment areas of Vam Co River (Spaans & Thanh Lam, ND)

<table>
<thead>
<tr>
<th>River</th>
<th>Catchment area (km²)</th>
<th>Whole basin</th>
<th>Inside Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vam Co Dong</td>
<td>8546</td>
<td>5005 (59%)</td>
<td></td>
</tr>
<tr>
<td>Vam Co Tay</td>
<td>921</td>
<td>921 (100%)</td>
<td></td>
</tr>
</tbody>
</table>

Discharges are generally low (see Figure 8.1) except for the rainy season, when the Vam Co receives substantial amounts of water from the Mekong River through an interconnected system of river systems and canals. A lot of these canals are manmade and were constructed in the past century with the purpose of developing agriculture and transportation. The irrigation system made it possible to reclaim large areas of alkaline soil, enabling two or three crops per year (Vietnam-Netherlands Cooperation, 2011). The highest salinities occur in April or in early May, which is the end of the dry season. At the start of the flood season, fresh water pushes the salinity back downstream.

Figure 8.1: Monthly average discharges based on measured data from 1988-2007
Saline water intrudes the Dong Thap Muoi via the Tien River and the Vam Co Tay. The latter has no upstream resources, so the salt intrusion can reach very far upstream. The Vam Co Dong is under the influence of the Dau Tieng reservoir.

The two branches of the Vam Co join near the Soi Rap mouth, which flows into the East Sea, see Figure 8.8. The tidal range in the East Sea is between 2.5 and 3.5 m, depending on the location. The tides in the East Sea have a mixed diurnal and semi-diurnal character. Generally there are two peaks a day, but some days the tide is mainly diurnal. See section 2.1.1.2 for more details about the tide.

The river is meandering, and has a moderate sinuosity index (Spaans & Thanh Lam, ND). The canals have a very gentle slope, and are therefore under heavy influence of the tide. The average slope of the Vam Co River is around 1/4000.
8.1.3 Characteristics of alluvial estuaries

8.1.3.1 Topography
Alluvial estuaries are in a dynamic equilibrium between deposition and erosion, in which some points in time, erosion is dominant, and at other points in time, deposition (Savenije, 2005). Both the river and the sea supply water, substances and sediments. The water movement depends on the topography, but it also shapes the topography. The natural shape of an alluvial estuary is such that the width reduces in upstream direction, following an exponential function. In coastal plain estuaries, the depth is constant. This shape of an ‘ideal estuary’ has been observed in coastal plain estuaries all over the world.

8.1.3.2 Mechanisms for salt intrusion
The salinity within an estuary is influenced by both the river and the sea: the river that discharges fresh water into the estuary, and the tidal movement of the sea that fills the estuary with saline water. Both fluxes depend on topography, and they also shape the topography by means of erosion and deposition.

In an estuary, the tidal propagation has the character of both a progressive wave and a standing wave. There are two main drivers for salt intrusion in estuaries: the density difference and the tide. With these two main drivers, four different mixing processes can be distinguished:

1) Turbulent mixing
This type of mixing results from the difference in water velocity over the vertical due to friction. The friction balances the tidal driving force. The difference in water velocity is caused by the shear stress, which is the largest near the bottom. This type of mixing is not so important in estuaries (Savenije, 2005).

2) Gravitational mixing
This type of mixing is caused by the imbalance in the equilibrium between the hydrostatic pressure on the seaside and on the riverside caused by density differences. Because of this density differences the pressure over the vertical is not equal. Near the surface, the resulting pressure is pointing towards the sea, while at the bottom the resulting pressure points upstream. This results in residual circulation which carries salt water upstream over the bottom. This type of mixing is important in estuaries where the salinity gradient is large.

3) Tidal trapping
Tidal trapping is caused by the phase difference between the filling and emptying main estuary branch and a dead-end tidal branch (Nguyen, 2008). At ebb, the delayed emptying of these ‘pockets’ of water, causing density differences. This mechanism is mainly important in irregular estuaries.

4) Tidal pumping
Tidal pumping is the mixing due to residual currents (Savenije, 2005). It is the dominant mechanism near the mouth of a wide estuary, but not much is known about the mechanism. The mixing is proportional to the estuary width instead of the salinity gradient.

8.1.4 Steady state salt intrusion model (Savenije)

8.1.4.1 Preface
Many formulae derived to determine the salt intrusion length were derived for prismatic channels. However, alluvial estuaries seldom have this shape. The occurrence of similar characteristics of alluvial estuaries was observed during many boat surveys which Savenije carried out in Mozambican and Asian
It appeared that these estuaries, although quite different in hydrology and geometry, had certain geometric characteristics in common (Savenije, 2005). It was observed that:

- The width reduces in upstream direction as an exponential function.
- In coastal plain estuaries there is no significant bottom slope, but in estuaries with strong relief, the depth may decrease exponentially. As a result, the cross-sectional area varies exponentially.

The analytical model of Savenije calculates the longitudinal distribution of the salinity for given topography, tidal condition and river discharge, starting from the mouth of the estuary. The model was successfully tested in over 18 estuaries in the world. The semi-empirical model was developed for single-channel estuaries with moderate tidal damping, but has also been tested for one multi-channel estuary, the Mekong estuary in Vietnam (Nguyen, 2008). The model uses the following ‘laws’, after (Savenije, 2005):

- The tidal excursion $E$ (the distance a water particle travels during a tidal cycle) is constant along a coastal plain estuary. This is related to the morphological equilibrium.
- There are simple analytical relations for estuary topography ($h/b$), wave celerity, and phase lag that can be derived from the equation for conservation of mass.
- We also see that tidal amplification obeys a simple linear function, whereas tidal damping is partly linear and partly exponential, based on the equation for conservation of momentum. Although this equation is more complex than ‘Green’s Law,’ it is still surprisingly simple.
- The propagation of the tidal wave is influenced by tidal damping (and vice versa). This interaction can also be described by a simple analytical equation.
- The phase lag between the moment of high water and the subsequent moment of slack, when the current changes direction, is a key parameter in tidal hydraulics, often disregarded.
- We observe that salt intrusion is well mixed or partially mixed at the time when it matters. In tidal estuaries, the salt wedge, which most people think is the dominant salt intrusion mechanism, either does not occur at all, or only occurs during high river floods, when nobody is worried about the salt intrusion but rather about flood protection.
- Mixing of salt and fresh water, although a complex process that results from many different mixing mechanisms, can be described by a surprisingly simple formula, originally coined by Van den Burg (1972).
- Salt intrusion can be described by an analytical equation that can be applied to new situations with a minimum of calibration. In fact the equation is predictable in that it can be applied outside the range of calibration, e.g. to analyse the effect of river discharge, interventions in the estuary by dredging, sea-level rise, etc.
The method developed by Savenije is different from numerical modelling in the sense that it is easy and predictive. The method assumes that a steady-state situation occurs, where the tidal average salinity stays constant over time in the estuary, assuming the estuary is well mixed. Steady state occurs when there is equilibrium between the advective transport of salt by the river in downstream direction, and the dispersive transport of salt in upstream direction.

Not all of the above mentioned aspects will be treated in this report. The reader is referred to (Savenije, 2005) for more in-depth theory. The method to predict the salt intrusion length will be treated in section 8.1.4.2. A section about unsteady-state salt intrusion models is given in appendix S.

8.1.4.2 Summary of the method

The method of Savenije assumes that the topography of an alluvial estuary can be described by exponential functions for the cross-sectional area, width and depth:

\[ A = A_0 \exp\left(-\frac{x}{a_i}\right) \quad (i = 1) \]  
(8.1)

\[ A = A_{i-1} \exp\left(-\frac{x - x_{i-1}}{a_i}\right) \quad (i > 1) \]  
(8.2)

\[ B = B_0 \exp\left(-\frac{x}{b_i}\right) \quad (i = 1) \]  
(8.3)

\[ B = B_{i-1} \exp\left(-\frac{x - x_{i-1}}{b_i}\right) \quad (i > 1) \]  
(8.4)

\[ h = h_0 \exp\left(-\frac{x(a_i - b_i)}{a_i b_i}\right) \quad (i = 1) \]  
(8.5)

\[ h = h_{i-1} \exp\left(-\frac{(x - x_{i-1})(a_i - b_i)}{a_i b_i}\right) \quad (i = 1) \]  
(8.6)

Savenije (2005) demonstrated that the steady-state salt balance equations for High Water Slack (HWS), Low Water Slack (LWS) and Tidal Average (TA) can be written as:

\[ S_i - S_f = c_i \frac{dS_i}{dx} \]  
(8.7)
Where \( i = 1, 2, 3 \) indicates three different states: HWS, LWS and TA. \( S_i \) [ML\(^{-3}\)] is the steady-state salinity and \( S_f \) is the fresh water salinity. The coefficient \( c_i \) is an \( x \)-dependent coefficient defined as:

\[
c_i = \frac{A}{Q_f} D_i
\]  

(8.8)

In which \( Q_f \) [L\(^3\)T\(^{-1}\)] is the river discharge which is negative since the positive \( x \)-axis points upstream, and \( A \) [L\(^2\)] is the tidal averaged cross-sectional area. \( D_i \) [m\(^2\)/s] is the dispersion coefficient for each state \( i \), which can be calculated by using the Van der Burgh equation:

\[
\frac{\partial d_i}{\partial x} = K \frac{Q_f}{A}
\]  

(8.9)

Where \( K \) [-] is the van den Burgh’s coefficient, which has a value between 0 and 1. \( K \) is a shape factor which is related to the general characteristics of an estuary such as its shape and average hydraulic conditions (Zhang et al., 2011). Equation (8.9) can be integrated for an estuary with an exponentially varying cross section by using equation (8.1). This yields an expression for the dispersion along the estuary:

\[
\frac{D_i}{D_{oi}} = 1 - \beta_i (\exp \left( \frac{x}{\alpha} \right) - 1)
\]  

(8.10)

with:

\[
\beta_i = -\frac{K a Q_f}{D_{oi} A_0} = \frac{K a}{\alpha_{oi} A_0}
\]  

(8.11)

and:

\[
\alpha_0 = -\frac{D_{oi}}{Q_f}
\]  

(8.12)

In which \( D_{oi} \) [L\(^2\)T\(^{-1}\)] is the boundary condition at the river mouth (\( x = 0 \)) for HWS, LWS and TA conditions, \( A_0 \) [L\(^2\)] is the tidal average cross-sectional area at the estuary mouth and \( \alpha \) [L] is the convergence length of the cross-sectional area. \( \alpha_{oi} \) [L\(^{-1}\)] is the mixing coefficient at the estuary mouth. The values of \( K \) and \( \alpha_{oi} \) can be obtained by calibrating the computed salt intrusion curve against measured longitudinal salinity distributions at HWA, LWA and TA. These measurements can be carried out by using the moving boat method (Savenije, 1989), measuring the salinity at HWS and LWS.

Combining equations (8.7) - (8.9) yields an expression for the longitudinal salinity distribution along the estuary:

\[
\frac{S_i - S_f}{S_{oi} - S_f} = \left( \frac{D_i}{D_{oi}} \right)^{1/K}
\]  

(8.13)

Where \( S_{oi} \) is the salinity at the estuary mouth (for HWA, LWS and TA) and \( S_f \) is the fresh water salinity. The salinity curve derived for the TA situation can be used to derive the salinity distribution at HWS or LWA by shifting the curve upstream of downstream over half the tidal excursion \( E \). The tidal excursion is the length a water particle travels between HWS and LWS. When \( S_i = S_f, D_i = 0 \) according to equation (8.13) \( D_i \) equals zero. Using this in equation (8.10) and re-arranging yields an expression for the total intrusion length:

\[
L_i = \alpha \ln \left( \frac{1}{\beta_i} - 1 \right)
\]  

(8.14)
Where $L_i$ is the salt intrusion length at HWS, LWS or TA.

8.1.4.3  Data for the Vam Co

Unfortunately it is not possible to perform moving boat measurements in the Vam Co River basin. Instead, salinity measurements from fixed measuring stations along the Vam Co River were used to fit a salinity distribution curve according to the method of Savenije. The data consists of measurements over the period of Feb - June 2005 and is supplied by the Water Resources University (WRU). The maximum and minimum salinity during a half tidal cycle were taken as the HWS and respectively LWS salinity. These maximum and minimum values are close to the HWS and LWS salinity.

The depth, cross-sectional area and width were obtained from measured cross sections which were supplied by the WRU. In addition, Google Earth is used to calculate the distance from the mouth to the measuring stations. The estuary characteristics are summarized in Table 8.2 and Figure 8.5 - Figure 8.7.

Table 8.2: Topographical parameters of the Vam Co River

<table>
<thead>
<tr>
<th>River</th>
<th>$A_o$ [m$^2$]</th>
<th>$B_o$ [m]</th>
<th>$h$ [m]</th>
<th>$a$ [km]</th>
<th>$b$ [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vam Co Chung</td>
<td>16200</td>
<td>1820</td>
<td>13.5</td>
<td>33.3</td>
<td>33.3</td>
</tr>
<tr>
<td>Vam Co Dong</td>
<td>2300</td>
<td>270</td>
<td>11.9</td>
<td>50</td>
<td>250</td>
</tr>
<tr>
<td>Vam Co Tay</td>
<td>2400</td>
<td>190</td>
<td>14.6</td>
<td>143</td>
<td>250</td>
</tr>
</tbody>
</table>

Figure 8.5: Topography of the Vam Co Chung
Figure 8.6: Topography of the Vam Co Dong

It can be observed that the cross-sectional area and the width obey an exponential function reasonably well for the Vam Co Dong and Vam Co Tay. The depth is relatively constant along the x-axis for the latter two. For the Vam Co Chung, the exponential relation is less obvious. There appears to be a sill at the confluence of the Vam Co Chung and the Soi Rap. The depth is about 8 m, while the rest of the river is between 13 – 15 m. The value of $A_0$ does not have its largest value at the estuary mouth as with an ‘ideal estuary’.

Figure 8.7: Topography of the Vam Co Tay
Figure 8.8 shows the Vam Co river system. Upstream the river splits into two branches. The location of the measurement stations is also given in Figure 8.8 and in Table 8.3.

![Vam Co river system](image)

**Figure 8.8: Vam Co river system (obtained from Google Maps)**

<table>
<thead>
<tr>
<th>Table 8.3: Measurement stations</th>
</tr>
</thead>
<tbody>
<tr>
<td>River</td>
</tr>
<tr>
<td>Vam Co Chung</td>
</tr>
<tr>
<td>Vam Co Dong</td>
</tr>
<tr>
<td>Vam Co Tay</td>
</tr>
</tbody>
</table>

Figure 8.11 till Figure 8.13 shows the measured data from the fixed stations which is used in the calculations. The delay between the high water and HWS (the peak of the salinity concentration) can clearly be observed. The highest and lowest salinity occurs at HWS and LWS respectively.
Figure 8.9: Measured salinity at Cau Noi station on April 16, 2005

Figure 8.10: Measured salinity and water level at Ben Luc station on April 16, 2005
8.1.4.4 Results

With the measured data from the fixed stations and the data from section 8.1.4.3 a salinity curve is fitted for April 16, 2005. Two curves were fitted: one for the Vam Co Chung + Vam Co dong and one for the Vam Co Chung + Vam Co Tay. The vertical line in the figures below indicates the confluence of the Vam Co Dong and Vam Co Tay. This point lies at x = 32.7 km from the mouth of the estuary.

The salinity at the mouth of the Soi Rap estuary (x = 0) is not known. The salinity should be between the sea salinity (±30 g/l) and the measured salinity at the Cau Noi station. Because the salinity of the Soi Rap mouth is also influenced by the Saigon River, the tidal average salinity will not be equal to the sea salinity of 30 g/l. Therefore the salinity is estimated at 25 g/l. If the Vam Co would have discharged directly on the sea, the boundary of 30 g/l would have been used.

Figure 8.11: Measured salinity and water level at Tan An station on April 16, 2005

Figure 8.12: Computed salinity distribution of the Vam Co Dong
The following values for $i$, $E$ and $\alpha$ were obtained from fitting the curves, see Table 8.4.

<table>
<thead>
<tr>
<th>River sections</th>
<th>Van den Burghs coefficient $K$ [-]</th>
<th>Tidal excursion $E$ [m]</th>
<th>Mixing coefficient $\alpha_0$ [1/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vam Co Chung-Vam Co Dong</td>
<td>0.64</td>
<td>17000</td>
<td>$\alpha_{01}$ 13.5 $\alpha_{02}$ 42</td>
</tr>
<tr>
<td>Vam Co Chung-Vam Co Tay</td>
<td>0.64</td>
<td>17000</td>
<td>$\alpha_{01}$ 13.5 $\alpha_{02}$ 56</td>
</tr>
</tbody>
</table>

With these parameters, the total salt intrusion length can be calculated using equation (8.14). This yield a total intrusion length of 102 km for the Vam Co Dong and 162 km for the Vam Co Tay.

### 8.1.5 Conclusions

The calculated values for the total salt intrusion length seem to make sense. The Vam Co Tay is most influenced by salt intrusion, which is also observed in reality. However the salt intrusion calculations made in this chapter are in no way accurate enough to make predictions about the salt intrusion under different circumstances (e.g. when the Vung Tau – Go Cong Dam has been built). The fitting of the salinity distribution curve is based on only two measured points, and some very rough estimations.

This estimation of the salinity $S_0$ at the mouth of the estuary is very weak. This boundary has a large influence on the salinity distribution curve and hence on the fit which is made. The fitting of the salinity distribution is already not very accurate because of the low amount of measuring stations. With more measurements, the tidal excursion $E$ can be estimated more accurately. The cross-sectional area of the Vam Co Chung (where the seawater enters) does not obey an exponential function perfectly. Since the method of Savenije assumes an exponential varying cross-sectional area, it remains the question whether it can be applied for the Vam Co.

There is also some uncertainty about the data supplied by the WRU. The amount of cross sections per river (VCC, VCD or VCT) is not very large, and the exact location of the cross section is not known (only relative distance, assuming it starts at the Soi Rap mouth).
8.1.6 Recommendations
The method of Savenije has proven itself in many estuaries worldwide, but due to lack of data it is not possible to draw good conclusions for the Vam Co estuary. There is still a large range of error caused by all the assumptions and lack of data. It is therefore advised to carry out moving boat measurements when one wants to use this method. That way, the salinity distribution can be fit with more data points, which gives more confidence for the computed values of $E$, $\alpha_0$ and $K$. The influence of the Go Cong – Vung Tau dam on the salt intrusion can be studied better in that case.

8.2 Model and Measurements

8.2.1 Hydraulic Model: SOBEK
SOBEK is the name of a sophisticated hydraulic software package, which in technical terms is a one-dimensional open-channel dynamic numerical modelling system, which is capable of solving the equations that describe unsteady water flow, salt intrusion, sediment transport, morphology and water quality. In less technical terms SOBEK can be described as a flexible, powerful and reliable tool to simulate and solve problems in river management, flood protection, design of canals, irrigation systems, water quality, navigation and dredging (Sturm, 2011).

Olga Sturm made a SOBEK model of the Saigon-Dong Nai delta for her Master thesis project at TU Delft. This model is used for this project. For this project it is not necessary to model the entire delta system in detail, therefore simplifications are made. The schematisation is based on information received from the Institute for Water & Environmental Research (IWER). The includes cross sections of the rivers, land elevation, measured water levels and measured river discharge.

This model is not adjusted by this project group, but it is used in its current form. For details about this hydraulic model, see the report of Olga Sturm (Sturm, 2011).

8.2.2 Measurements

8.2.2.1 Velocity measurements on the Saigon river
On Friday 1st of September a delegation of the project group did velocity measurements in the Saigon river near the Phu An measurement station. The goal of these measurements was to get an idea of the velocities in the Saigon river and to see if they were in the same range as the expected velocity from the hydraulic SOBEK model.

To be able to compare the model with the situation when modelling, the boundary conditions at the time of modelling must be known or assumed.

Upstream boundary conditions
The upstream boundary conditions depend on the rainfall over a larger period of time and the operation of the dams. The discharges during our measurements will be comparable with the discharges of Septembers in earlier years for the part concerning the rain fall. The discharge due to the opening of the dam is a block function and it is not possible to see a pattern in it, and to make an estimate of at what day the flood wave will come downstream the river.

The measurement data of the discharges of the last 10 available years are used to calculate the expected discharges for this month September. (data in appendix T) These discharges are used in the SOBEK model as the upstream boundary conditions.
**Downstream boundary conditions**

The downstream boundary conditions depend on the tide. The tide can be predicted accurately. For this reason, a combination of predicted data for the first weeks of September and measurement data of the last 10 measured years will be used to create a representative boundary condition of the tidal elevation in September 2011.

The predicted water level at Vung Tau for the first weeks of September is showed in Figure 8.14. This prediction from the UK Hydrographic Office (UKHO, n.d.) only predicts for one week ahead. For the other data of the month September, data of water level measurements is used. The predicted values of the UKHO are compared with the measured water levels to choose the right period for the water levels, this is illustrated in Figure 8.15. The red square is part of tidal cycle that matches the prediction of UKHO. The water level data from the start of this red square running for 30 days will be used as the downstream boundary condition.

*Figure 8.14: Predicted water level in Vung Tau (31 August - 6 September) (UKHO, n.d.)*
8.2.2.2 Prediction of velocities

The velocities at the Phu An reach at the 1st of September are predicted with the SOBEK model. The outcome of the run that predicts the velocities can be found in appendix T. The velocity at the Phu An reach will rise from 0 m/s to 1.25 m/s between 10h30 and 14h00.

8.2.2.3 Measurements – Set up

With the SOBEK model, the best time is determined (regarding to Low Water and High Water) to do the velocity measurements. The highest velocities are the most interesting for our study. To know for sure we have measured the top velocity, we continued our measurements until the peak velocity has occurred.

From the study prior to the measurements it was estimated that the highest velocities occur around 14h00. The highest velocities have to be measured, so this is the best time to do the measurements. To be sure that the way of measurements works and to get a feel for doing the measurements, the measurements are started at 13h00.

Two persons are involved in doing the measurements to obtain the velocity, both standing on a line with reference points on both sides of the river (A and B, see Figure 8.16). The time it takes a floating object to get from one line to another is measured with a stopwatch. The distance between the two lines is measured with Google Earth (see Figure 8.16). With this information the velocity can be calculated.

![Figure 8.15: Water level in Vung Tau - September 2007](image)
8.2.2.4 Measurements – Outcome and conclusion

The measurements resulted in less measurement points than planned, because of the dependence of usable floating objects in the water. Before 14h00 a clear decrease in velocity was measured and the peak velocity appeared to be approximately 1 m/s.

![Velocity - Measurements](image)

*Figure 8.17: Measured velocity - Phu An*

Some remarks have to be made about this measurement method.

1. Theoretical remarks
   The measured velocity is not the same as the mean velocity. The measured velocity is the velocity of the water surface, and the surface velocity is not the same as the mean velocity of the depth profile. The velocity of floating objects in the middle area and surroundings were
measured. The velocity in the middle of the river is larger than on the sides, so the measured velocity is different than the mean velocity of the river.

2. Remarks regarding external factors influencing the reliability of the measurements.
Passing ships and wind will influence the velocity of the water (surface). Wind can also speed up or slow down the floating object if the float has a large volume above the water surface.

3. Remarks about possible measurement errors.
Due to human errors in misreading of signs or delay in handling measurement errors can occur.

4. Remarks about the uncertainties in the prediction of the velocity
The exact tidal elevation is difficult to get exactly right in the model and the average discharge will be different than the real discharge. Therefore this method will and should only be used to get an indication of the velocities at the location.

8.2.3 Velocity measurements on the Nhà Bè and Vam Cố Tây
On Sunday 25th of September our group was invited to see some field work: velocity measurements on the Nhà Bè River (near Nhà Bè) and Vam Cố Tây River (near Tân An). On both locations velocity measurements are performed by the Southern Institute of Water Resources Research (SIWRR) to calibrate their hydraulic model of the Saigon - Dong Nai delta.

At the Nhà Bè River velocity measurements are done an hourly interval for three days, on 6 different depths in the river profile: near the bottom, on 0.2d(depth), 0.4d, 0.6d, 0.8d and ±20 cm under the water level. In one river cross section there are 2 (Vam Cố Tây, at both sides) or 3 (Nhà Bè, at the two sides and in the middle) different measurement locations, with all at least 3 days of measuring. At the Vam Co Tay they measured for 15 days to estimate the tidal cycle. The corresponding water level is measured by a datum point along the river. The measurements are done every hour, also during the night. Therefore, the measurement crew consists of three persons and sleeps on the boat.

The measurements are done with a lead fish. On this fish there is a calibrated propeller (see Figure 8.20). A small machine is counting the turns of this propeller and gives a beep when 5 turns are made. By counting the number of beeps and measuring the time, the water velocity can be determined. One series of measurements takes about 10 minutes. The results of the measurements are written down to be processed with a computer after finishing the measurement campaign. An impression of some measurement data is given in Figure 8.19.

By plotting the velocities against the corresponding depth a relation will be found for the water discharge, to use in the hydraulic model and to make future predictions.

Figure 8.18: Measurement location in the Vam Cố Tây River
Figure 8.19: Registration of measurements results
Figure 8.20: Measurement lead fish and propeller

Figure 8.21: Measurement equipment


9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

The construction of a dam between Vung Tau and Go Cong is a solution for the flooding problems in the area around Ho Chi Minh City. A preliminary design for this dam was made. Also the building method of this dam was investigated, with special attention to the closure of the final gap.

Dam design

The use of locally available material can reduce the building costs for the dam significantly. Rock, bamboo and sand are available in the area. The core of the dam will be made of sand, which can be obtained near Vung Tau.

No soil data was available of the project location. Therefore soil data from the surroundings was used to draw up three possible soil profiles. To create a good foundation and to reduce settlements during the lifetime of the dam soil improvement is necessary. Different solutions are investigated for the different soil profiles. In case a relatively thin weak top layer is present, the layer will be replaced by sand. For a soil with a thick weak top layer, drainage and pre-loading will be applied.

The outer slope can be protected with either rock or asphalt. In Figure 9.1 and Figure 9.2 the cross sections of the designs with these protection materials are presented.

The crest of the dam with a rock revetment will have an elevation of 6.8 m above Mean Sea Level. To calculate the required crest height, the elevated water level, the freeboard for overtopping and the settlements of the dam body and subsoil were taken into account. The elevated water level consists of tidal elevation, storm surge, sea level rise and wave surge. An overtopping discharge of 50 l/m/s is allowed during design conditions. This decreases the crest height significantly compared to a situation where no or only a little overtopping is allowed.

On the sea side a berm of 16 m is designed, to reduce wave overtopping and provide stability. The berm will be constructed at MSL +4.2 m, one meter above the design Still Water Level. Settlements and sea level rise during the lifetime of the dam were considered in the design process. Slopes reduce the amount of material that is required for the core of the dam. The outer slopes will have an inclination of 1:4.

On the land side, where calmer water and a lower water level are present, the slope above water will have an inclination of 1:3. Under the water level, the slope will be 1:3.5 for stability reasons. The berm on the inner slope with a width of 25 m will be used for the road connection between Vung Tau and Go Cong. This berm also provides additional stability for the dam.

---

Figure 9.1: Cross section dam with rock revetment.
The armour layers on the slopes and crest prevent erosion of the fill material. The rock dimensions of the slope protection on the outer slope are mainly determined by wave attack. The slope protection on the inner slope is calculated based on the overtopping conditions that can occur. The armour layers rest upon an underlayer to ensure stability and on several filter layers to prevent the sand fill from washing out through the pores in the protection layers. A toe protection supports the armour layer and underlayer on both sides of the dam. A scour protection is applied to prevent erosion and undermining of the toe.

![Figure 9.2: Cross section dam with asphalt revetment](image)

The main differences between the cross section with rock revetment and the cross section with asphalt revetment are the thickness of the protection layers and the crest elevation. The asphalt layer requires only one under layer with a thickness of 0.5 m to ensure stability. The thickness of the asphalt layer varies over the cross section. On the sea side the asphalt layer is the thickest (0.76 m) around MSL to prevent uplift and shear. The crest level for the dam with asphalt revetment is located higher than for the rock revetment (MSL +7.6 m compared to MSL +6.8 m), because a larger freeboard height is required to allow for the same amount of overtopping as on the rock slope.

**Building method**

Construction of the dam core and protection material will be done layer by layer. The sand fill material will be enclosed between bunds of protection material during construction. With this method it is possible to construct the steep slopes that were designed. Another advantage is that material losses are minimised. Where possible, waterborne equipment will be used, because of the larger capacity and easier logistics. If the water depth is too shallow for barges, pontoons with cranes and rolling equipment will be used. This alternative construction method is more complex than the traditional building method, where core material is dumped first, followed by the construction of the protection. This traditional method will be used for the part of the dam above the Mean Higher High Water level.

Flow velocities that occur during closure of the final gap were calculated using a storage area approach. Velocities up to 4 or 5 m/s can occur for the last part of the closure, depending on the cross sectional area of the discharge sluices, which are open during the closing procedure. For a gap width smaller than 1370 m, flow velocities will be too high to construct the dam with sand. Rock will therefore be used instead to close the final gap.

First, a bottom protection will be made, which prevents the dam from falling into the scour hole that will originate due to increased flow velocities. Next, the gap will be closed vertically using barges until navigation is impossible due to high velocities or shallow water depths. Then, the gap will be closed horizontally by dumping big rocks, preferably during neap tide when flow velocities are the lowest. Finally, the dam construction will be finished by constructing the sand core, toe protection and slope protection around the rock core. The ultimate dam profile in the final gap will be similar to the rest of the dam.
9.2 Recommendations

The designed dam and the building method of the dam are based on the available data and some assumptions for unknown parameters. The use of more accurate data, obtained by measurements or research, will lead to a better design for the situation. This design of the dam is a preliminary design, partly based on simplified calculations and rules of thumb. More detailed calculations will be necessary in the next phase of the design. Also the design process should be repeated several times, to finally result in the best design.

Soil
No soil data was available of the location of the dam. It is highly recommended to do soil investigation on the location of the dam to obtain information about soil layering, strength parameters, consolidation parameters, et cetera.

The calculations rules used for the bearing capacity of the subsoil are meant for smaller constructions. Rules for large dams such as the VT-GC-dam could not be found in the Eurocode 7. The use of an advanced calculation method is recommended.

Materials
Assumptions were made for the material parameters (weight, strength parameters, grading etc.) of the sand and rock that can be found in the area. The exact parameters should be determined for future design. Further, it was assumed that there is enough material available. This also needs to be confirmed.

At the location of the dam, plenty of mud is available. This might be appropriate to use as fill material, but too many parameters are unknown to confirm this. Investigation of the possibilities of mud could be interesting, because it can be obtained from the project site and therefore it could be a very cheap construction material.

Hydraulic parameters
The storm surge elevation of the still water level was determined using wind speed data for a 1/200 year design storm, an estimated fetch length and the geometry of the sea bottom. Research into the estimated input parameters will be necessary. Also a more detailed calculation of the storm surge level is advised in further stages of the design.

The design wave height was determined using parameters from the Vietnamese design code and a computer program. Several input parameters (bottom geometry, wave heights etc.) were estimated. It is advised to use measurement data to improve the input parameters for the wave height calculation.

Closure gap
The velocities in the closure gap are important for the building method and the closure gap-design. A rough estimate for the basin area was used in the calculation for this preliminary design. To make a good prediction of the occurring velocities, the surface area of the basin (including the mangrove and river system) has to be measured.

When more information is available about the water depth at the location of the dam (depending on the trajectory of the dam) and the sizes of the discharge sluices, better predictions for the velocities in the final gap can be made. A 2D hydraulic model (like MIKE11 or SOBEK) can be used for this purpose.

Salt intrusion
To make the predictive steady-state salt intrusion model from Savenije more accurate, it is recommended to perform moving boat measurements at High Water Slack (HWS) and Low Water Slack (LWS). With the
salinity concentrations obtained with these measurements, a better calibration can be made which makes it possible to draw sound conclusions for the Vam Co rivers in the future situation.

**Total design**

The design of a dam is a cyclic process. In this preliminary design some iteration steps were made during the design of the cross section. However, the building method was not taken into account during the design process. This should be lead to a better design, as the design will be better executable.

The relatively steep slopes that were designed to save on material are difficult to construct, even with the building method that was chosen. More gentle slopes are easier to build, but result in the use of more material. It is advised to research a design with more gentle slopes.

Model testing is recommended to determine the stone sizes for the outer slope protection, the final closure and the bottom protection.
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