Design of a pilot dike on Cat Hai Island

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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.

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Report

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Main Report

Preface

This report presents the results of designing two test dikes on Cat Hai Island in Vietnam. In the period of half August to half October 2010 we have done a multidisciplinary project for the Ministry of Agriculture and Rural Development - Center of Water Resources Consultant and Technology transfer (MARD-CWRCT). The CWRCT is a non-state sub division of the MARD, they operate like an independent consultancy company. We also got support and information from the Water Resource University (WRU) in Hanoi.

We are very thankful for the hospitality of the MARD-CWRCT and WRU. We want to thank all people that have helped us with this project. Especially ME. Nguyen Viet Tien, the director of MARD-CWRCT for giving us the opportunity to do our Master project in Vietnam and organizing all the small side activities around Ha Noi. Besides that we like to thank the two departments CWRCT and DDMFC for arranging a visit to see the dikes during the field trips to the Hai Phong area and the Nam Dinh province. Further, Miss Thu for translating documents from Vietnamese to English for us and being our interpreter. From the WRU we like to thank Dr. Thieu Quang Tuan, for his keen eye on or report and for answering all our questions. Next to that ir. Gerrit Jan Schriereck, one of the senior staff members of TU-Delft for discussing the possibilities of our project in Vietnam. Last but not least, our instructor Henk-Jan Verhagen for providing this wonderful project at the MARD-CWRCT in Hanoi.

To do a project in a different country than the Netherlands was a great opportunity. It gave us an insight in the civil engineering world, where we as young professionals could enter after our graduation. Looking back on the time in Vietnam, we can only say that it was an unforgettable and fantastic experience.

On behave of the Mastergroup ‘Lets go Vietnam 2010’ Thanks!

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Hanoi, October 2010

Abstract

On behalf of, and in cooperation with the TU Delft and the MARD-CWRCT two dike designs are made for a pilot project on Cat Hai Island. Two different types of revetments are used, namely the Dutch block type revetment Basalton® and the classic rip-rap revetment. The designs are based on the available boundary conditions, the new Vietnamese guidelines and the Dutch knowledge. The focus points of the technical design are the geometry, the revetment, toe construction and the settlement. Compared to the current Vietnamese dikes, the design is revolutionary due to the presence of an outer berm. Besides the technical part of the design, the logistics and finance for the pilot project is explored. The different possibilities of producing, placing and transporting together with a time and cost estimations are given. In addition a proposal is given for the maintenance schedule and how to monitor the dike.

Eventually two optimal dikes are further investigated, this height is based on zero overtopping and the most economical feasible. The crest heights of the old Vietnamese dikes on Cat Hai Island are around 4,80 m and it has a dike footprint width of 40,20 m. The crest height of the Basalton® dike will be one meter higher than that of the rip-rap dike and the rip-rap will be one meter higher than then old dike. Because Baslaton® has a larger wave run-up than a rip-rap dike. Therefor the footprints of both the new Basalton® and rip-rap dike are almost twice as large as the old footprint. The reason for this is because of the use of a berm in the outerslope for the Basalton® dike. An estimation of the cost of this project is for a rip-rap dike, roughly 24 billion VND and for the Basalton® dike more or less 21 billion VND.

Table of Contents

List of tables

List of figures

1 Introduction / project description

The MARD-CWRCT wants to build two pilot test dikes with different revetments on Cat Hai Island. In this report the steps which have been taken to design the dikes are explained. With this report, the CWRCT can assess the results of the two test dikes and give a good perception whether the dikes are suitable for larger areas in Vietnam. Also the logistic part of building the dikes together with a cost estimating will be explored.

1.1 Vietnam and typhoons

Vietnam is located in a typhoon area. There was a lot of damage at the coastal area after typhoon 'Damrey' in 2005. Due to that typhoon, the government designed a new guideline with new safety standards. This guideline was published in March 2010. New dike designs will be based on this new guideline. Some parts of the new guideline are specific for the Netherlands and not useful for Vietnam.

1.2 Vietnam in general

The Socialist Republic of Vietnam is located in the Southeast of Asia and has a population of 86 million. Vietnam is known for the Vietnam war between 1955 and 1975, its rice export, interesting cities, temples, pagodas and a beautiful mixed scenery of mountainous, beaches and unique historic leftovers.

In the north, Vietnam is bordered to China, in the northwest to Laos and in the southwest to Cambodia. The countries are split by the Truon Son Mountains. The maximum distance from north to south is 1650 kilometers, and the width of the country ranges from 50 to 600 kilometers.

The total coastline of Vietnam is 3200 kilometers long. 75% of Vietnam consists of Mountains and hills. Besides the main land, thousands of islands also belong to Vietnam. The two major cities of Vietnam are Ho Chi Minh City and Hanoi.

In Table 1-1 an overview is given about some statistics of Vietnam compared to the Netherlands.

Figure 1-1 Map of the Netherlands and Vietnam

Statistics of Vietnam	Netherlands	Vietnam
Area of country [sq. km]	33.760	310.070
Population in 1960 [million]	11,5	34,7
Population in 2008 [million]	16,4	86,2
GDP growth in 2008 [annual %]	2,10%	6,20%
GDP per capita in 2008 [current $]	$52.963	$1.051
Length of coastline [km]	353	3200
Length of seadikes [km]	*34*	*2000*
Economy near seaside [%]	*70%*	*< 50%*
Export [% of GDP, 2008]	77%	78%
Import [% of GDP, 2008]	69%	95%

Table 1-1 Overview of statistics of the Netherlands vs Vietnam

1.3 Problem definition

Designing dikes in Vietnam is done by a relatively old principle. The dikes are made for a storm surge with typhoon force 9 or 10. This has resulted in a lot of damage on some of the dikes. Especially the typhoon of 2005 caused severe damage to different (pilot) projects on Cat Hai Island near Hai Phong. The revetments have been made according to the Vietnamese standards at that time. At that time the trend was to use interlocking blocks such as TAC. Interlocking blocks are good to spread the load on the revetment to a bigger area. However, one of the disadvantages is that when one block needs to be replaced it isn't easy to do so. Because of the interlocking system a whole dike section needs to be lifted up to restore just a few damaged blocks. This replacement can be necessary due to bad quality of the concrete, storm surge damage or a combination of these two factors.

A good alternative for interlocking blocks is the use of blocks that have friction between the blocks. In this way the function of spreading the load over a bigger area remains. This however creates a whole new design and a new design involves a new standard. In March 2010 the new guidelines for dikes & revetments was approved by the policy makers.

On Cat Hai Island a new pilot project is set up by the Ministry of Agriculture and Rural Development (MARD) to test the new guidelines. This testing will be the task of the Center for Water Resources Consultant and Technology Transfer (CWRCT).

The CWRCT wants to make a 400 m long pilot project on Cat Hai Island spited up into two sections of 200m. The CWRCT asked for two different kinds of revetments; one for each section.

1.3.1 Research objectives

The following main objectives can be formulated:

(1) Design two pilot dikes on Cat Hai Island, one pilot dike with a Basalton® revetment and one alternative dike with a rip-rap revetment;
(2) Determine the best logistics for the two pilot dikes;
(3) Creating a monitoring manual for surveying the two pilot dikes;
(4) Setting up a financial overview for the two pilots dikes.

1.3.2 Goal objectives

For each objective several goals can be formulated. These goals are the basis for the planning and the task that has to done.

Objective (1):

Design two pilot dikes on Cat Hai Island, one pilot dike with a Basalton® revetment and one with a rip-rap revetment.
This objective can be divided into four goals:

1. Design the geometry of the two pilot dikes;
2. Design the buildup of the two pilot dikes;
3. Design the first dike with a Basalton® revetment;
4. Design the second dike with a rip-rap revetment;

Objective (2):

Determine the best logistics for the two pilot dikes.
This objective can be divided into two goals:

1. Finding out efficient constructions methods for the two pilot dikes;
2. Finding out the most efficient logistics for the two revetments.

Objective (3):

Creating a monitoring manual for surveying the two pilot dikes.
This objective can be divided into two goals:

1. Determine sort of survey;
2. Creating time schedule for surveying during and after construction.

Objective (4):

Setting up a financial overview for the two pilots dikes.
This objective can be divided into two goals:

1. Determine the total costs for the Basalton® dike;
2. Determine the total costs for the rip-rap dike.

1.4 Reading guide

Chapter 2 gives an overview of the current situation including the problems with Vietnamese dikes. Chapter 3 gives a description of the boundary conditions, which are needed in order to design a dike. Chapter 4 will discuss the choice for the second revetment. This is done by making a Multi Criteria Analyses.

The geometry will give as result all dimension of the dike, this is described in chapter 5. The soil build up with settlement and stability is given in chapter 6. Chapter 7 gives a calculation of dimension of the height of the revetment and thickness of filter layer. An overview with calculation of a toe design is given in chapter 8.

The logistics and financial part is described in chapter 9 and an overview of the monitoring is given in chapter 10. The conclusion and recommendations of the project are given in Chapter 11. The list of used literature is mentioned is chapter 12.

2 Current situation

To get a better understanding of the current state of the dikes and infrastructure of north Vietnam and Cat Hai some dikes on Cat Hai Island and in the area of Hai Phong were inspected.

This chapter will discuss the different kind of dikes and revetments that are used on the current test dike sections and the technical aspects of the dikes will be explained. There is also an impression of the state of the infrastructure in Vietnam. And finally there is some information about Basalton® from the Netherlands.

In the region of Hai Phong there are several dikes with different kind of revetments. Figure 2-2 shows the location of Hai Phong. The following dikes will be discussed:

- Test dike #1 on Cat Hai Island
- Test dike #2 on Cat Hai Island
- Rip-rap revetment dike on Cat Hai Island
- Sea dike #1
- Sea dike #2
- Sea dike of Tinh Quang Ninh

Figure 2-1 Map of Vietnam, Hai Phong located in the north

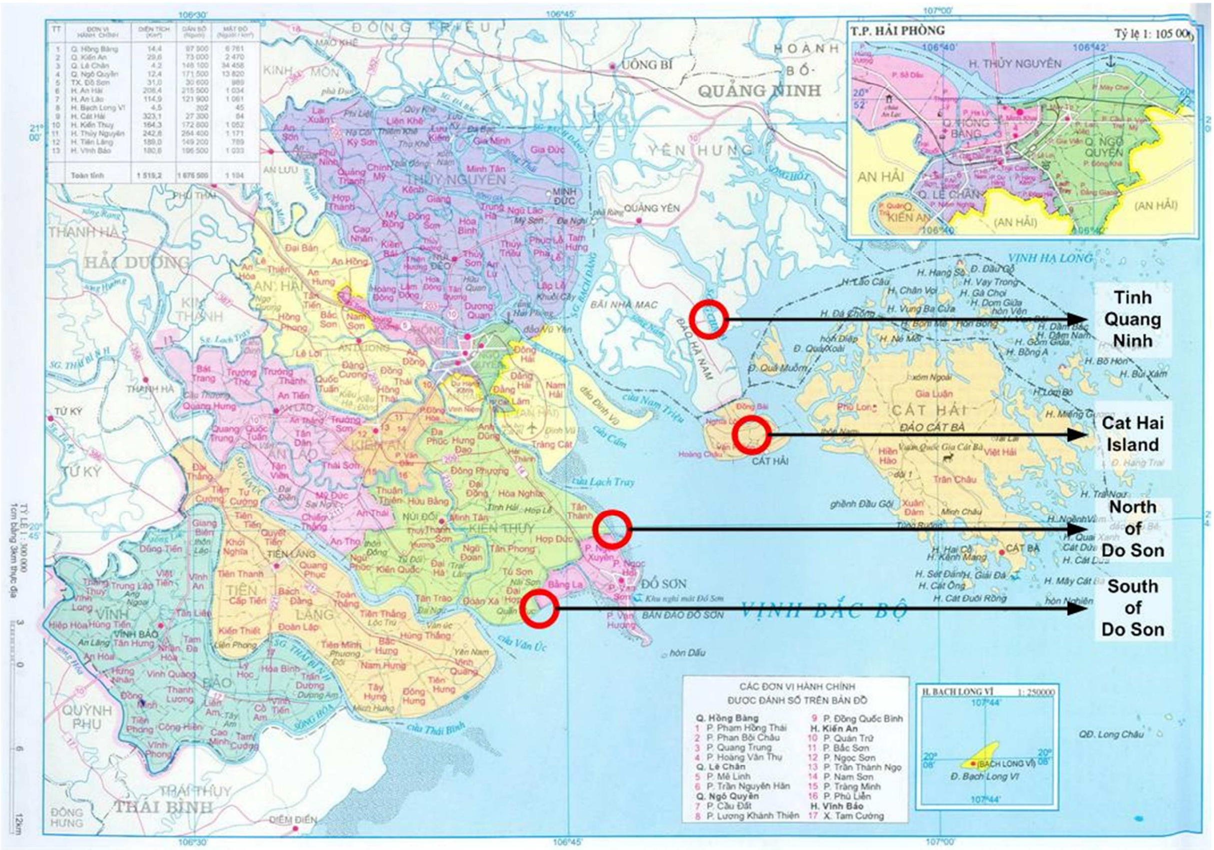

Figure 2-2 Hai Phong Province

2.1 Cat Hai Island

Cat Hai Island is located in north-east Vietnam, the geographical coordinates are 20 ° 48' 0" north and 106° 55' 0" east. Cat Hai is a small island with a total area of 30 km². There are 28.113 people living on the island. The island is a district of Hai Phong, bordering the Ha

Nam Island (Quang Ninh province) in the North, Gulf of Tonkin in the South, Lach Huyen river mouth in the East and the Nam Trieu river mouth in the West.

The southern coastline of Cat Hai Island has 10 km of sea dikes to protect the hinterland. Some parts of the sea dikes have been damaged during the typhoon in 2005. Figure 2-3 gives an overview of the Island and shows the land use. The main economic value of these areas is considered low. The northern part of the island are wetlands, the southern part exists of salt and shrimp fields. The sea dikes protect 2000 ha of land, 700 ha of wetlands outside the dikes is used for shrimp cultivation. The main economic activities are fishing, shrimp cultivation and fabrication of fish sauce. The pilot test dike (shown in green) is located in the south-east of Cat Hai and consists of two times 200 meter. Cat Hai has erosion at the southern dike, and a shipping channel on the north which is silting up.

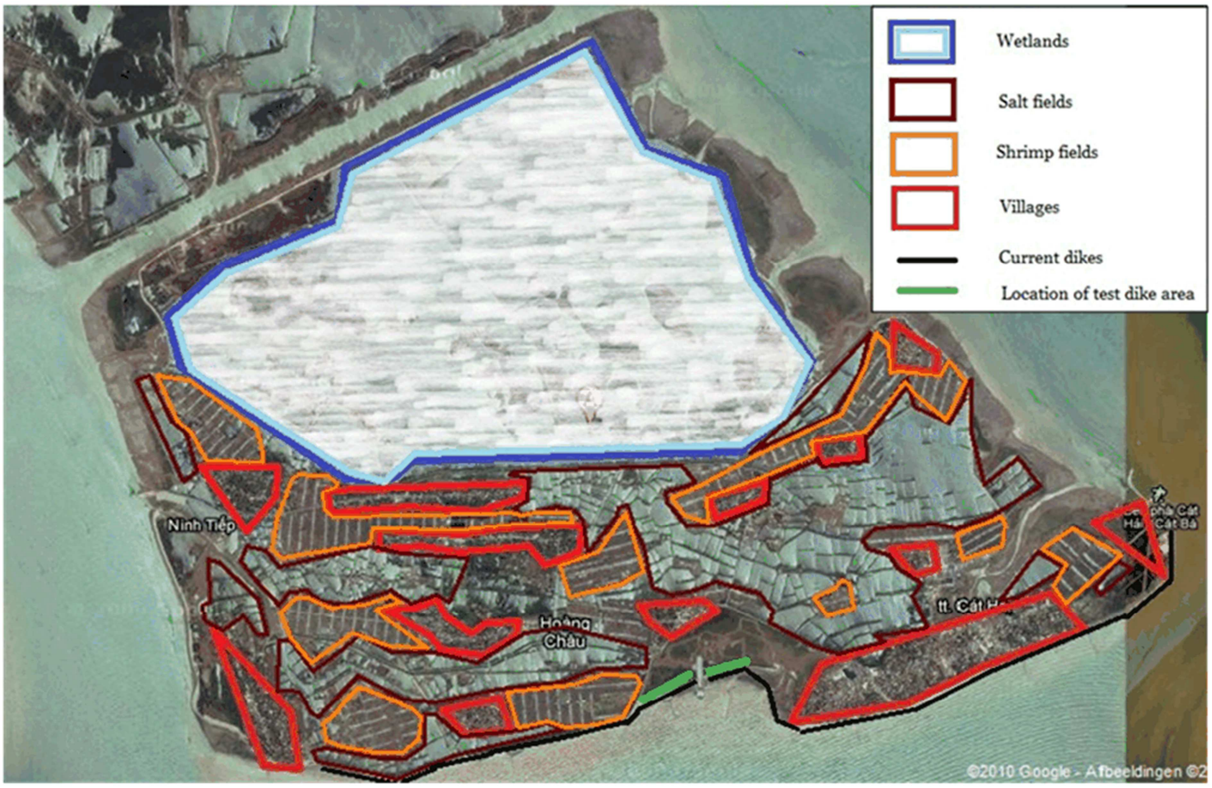

Figure 2-3 Overview of Cat Hai Island

2.1.1 Technical data of test dike #1

The existing dike consists of two types of different concrete block revetments. Figure 2-6 shows a block with the size of 60x60x28 cm Figure 2-4 shows a schematic cross section of the dike. The block revetment uses an interlock system and some concrete blocks are tired out from wave attack, run up and currents. On some places, the revetment clearly shows the damage caused by typhoon Damrey in 2005, see Figure 2-5.

Figure 2-4 Schematic cross section of test dike #1 on Cat Hai Island

Figure 2-5 Test dike #1 on Cat Hai Island with concrete interlocking blocks 60x60x28 cm

2.1.2 Technical data of test dike #2

Test dike #2 is located next to test dike #1. On this test dike a different kind of revetment was applied. Figure 2-6 shows this revetment: a flat concrete interlocking block measuring 60x60x28 cm. A cross section of the dike is shown in Figure 2-7. The dike shows no damage because this revetment has been built after the typhoon of 2005.

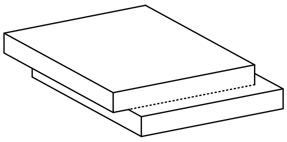

Figure 2-6 Flat concrete interlocking blocks 60x60x28 cm

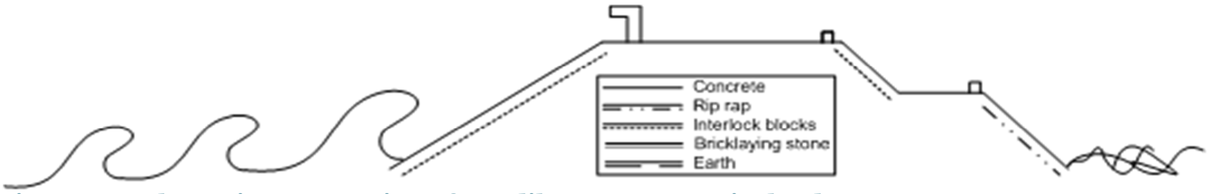

Figure 2-7 Schematic cross section of test dike #2 on Cat Hai Island

Remarkable is the large inner slope with a berm. There are a couple of reasons for this large inner slope and inner berm:

- It is easy to drive over the berm and do some maintenance;
- The knowledge in Vietnam on how to design a sea dike is limited. Designs used for a river dike has just been copied to act as a sea dike;
- Often farmers damage parts of the inner slope. The inner slope protects the dike for failure of instability.

Figure 2-8 Test dike #2 on Cat Hai Island with flat concrete blocks

2.1.3 Technical data of rip-rap revetment dike

A few kilometers from the test dikes on Cat Hai Island there was a dike with rip-rap revetment, shown in Figure 2-9 & Figure 2-10. Some parts of this revetment also showed signs of damage.

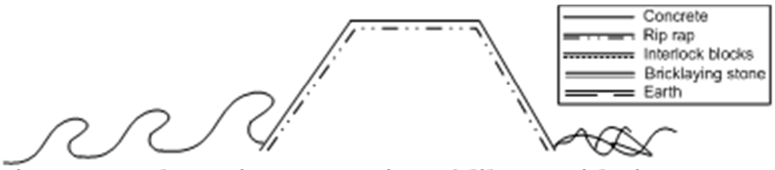

Figure 2-9 Schematic cross section of dike #3 with rip-rap revetment on Cat Hai Island


Figure 2-10 Rip-rap revetment dike on Cat Hai Island

2.2 Sea dikes of Do Son

2.2.1 Technical data of sea dike #1

This dike has two different revetments: a TAC-block revetment and a rip-rap revetment. The size of the TAC-blocks is 40x40x24 cm. In front of the dike there is a mangrove forest which serves as an extra protection for the dike against wave attack. At some places the mangrove forest has a width of 250 meters, at other places the forest is just new and has to develop into a more spacious forest in the future.

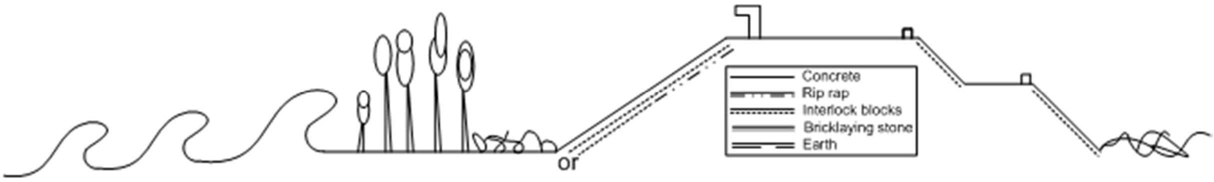

Figure 2-11 Schematic cross section of sea dike #1


Figure 2-12 Sea dike #1 in Do Son

2.2.2 Technical data of sea dike #2

Sea dike number two exists of two different revetments: one with concrete blocks with a size of 40x40x24 cm and another revetment with grass. A lot of mangrove forest, rice-fields and the grass revetment create a natural environment. It is interesting to note that, as shown in Figure 2-14 that at the moment of the inspection there was no water on the seaside of the dike.

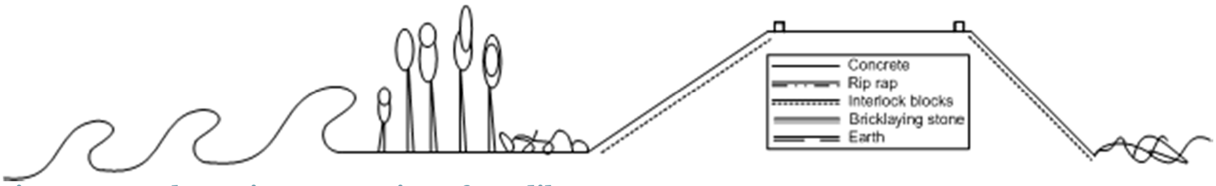

Figure 2-13 Schematic cross section of sea dike #2

Figure 2-14 Sea dike #2 in Do Son with concrete block 40x40x24 cm

2.3 Sea dike of Tinh Quang Ninh

This dike (Figure 2-15) is located in the orifice of the river Song Chanh and the sea. The dike is 6 km long and was built in 2007/2008.

2.3.1 Technical data

In front of the dike some mangroves are growing. The outer slope exists of interlocking blocks with an outside dimension of 40x40x24 cm. These blocks give no protection against wave attacks. The dike has a wave deflecting wall with a typical shape. The shape is used to reduce the speed and direction of the wave. After the crown wall there is a strait plateau. This plateau has no direct function for transport. On this plateau plants grow through the stones. This plateau is probably only used for the mass and stability of the dike. After this plateau there is a road on the inner berm. The road is wide enough for two cars to pass each other. On the inner slope the revetment is made of rip-rap.

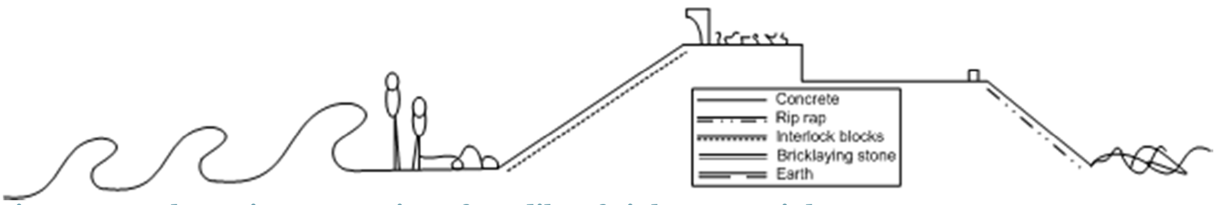

Figure 2-15 Schematic cross section of sea dike of Tinh Quang Ninh

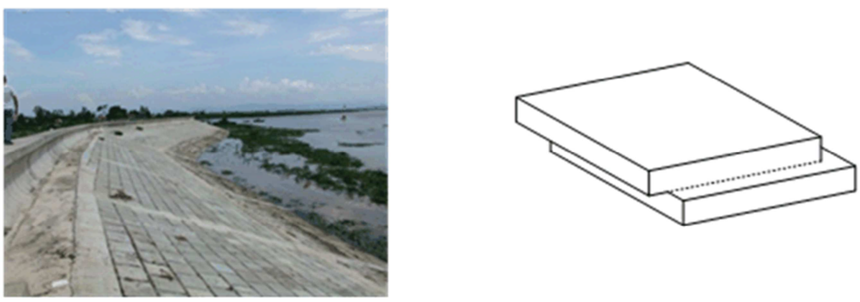

Figure 2-16 – Sea dike Tinh Quang Ninh with flat concrete block 40x40x24 cm

2.4 Rip-rap revetment dike

The dike with rip-rap revetment is located more land inwards at the orifice. The dike is 28 km long and in front of the dike grows some very small mangrove plants.

2.4.1 Technical data

The outer slope exist of rip-rap and the slope is very steep in comparison to Dutch rip-rap slopes. On the rip-rap slope a lot of plants grow which give the dike a natural look. This dike shows no damage from a typhoon. It is unknown when it was built exactly, but is has withstand the typhoon of 2005. Likely because the dike is located more land inwards and as a result of this the waves have reduced before they hit the dike. The dike has a big crown wall build from bricklaying stone; it looks the same as the rip-rap stones. Between the crown wall and the road on the dike there is a lot of vegetation. It is an unpaved road without a guardrail. The inner slope has no placed revetment: it solely exists of soil with vegetation on top of it.

Figure 2-17 – Schematic cross section of Tinh Quang Ninh dike with rip-rap revetment

Figure 2-18 Sea dike of Tinh Quang Ninh with rip-rap revetment

Noticed that the blocks are in bad condition. Some blocks are broken, other blocks were blasted away during the typhoon and others are eroded by the currents. An overview of the different damage is given in the next paragraphs.

2.4.1.1 Uplift

Recent storms have caused blocks to be uplifted by the water inside the dike, shown in Figure 2-19.

Figure 2-19 Uplift

2.4.1.2 Rust

To blocks are put on the dike by hand. In order to transport the blocks, a steel ring is attached. Because of the wet environment, the steel rusts. This is not good for the concrete quality.

Figure 2-20 Rust of the transport ring

2.4.1.3 Irregular placement of blocks

In order for the dike to behave like it is designed, it is important that the blocks are placed precisely. As can be seen in Figure 2-21 this is not the case.

Figure 2-21 Irregular placement of blocks

2.4.1.4 Placing instead of dumping of rip-rap

Rip-rap works as designed when it is dumped and makes a natural slope. In Vietnam they place the rocks in order and by hands, causing the rip-rap to lose strength.

Figure 2-22 Rip rap dike

2.4.1.5 Cracks in concrete

The concrete that is placed in-situ for dilatation shows cracks. Concrete is placed in between sections. If a section is damage just that specific section need replacement and not the whole dike. These transitions are made from reinforced concrete.

Figure 2-23 Cracks in concrete

2.5 Toe

Toes are very important for dikes. Without toe protection erosion can occur. This will result in scour holes, which can lead to instability of the outer slope. Placing a toe, doesn't mean erosion will not occur, it will resituate the erosion to the beginning of the toe. There are different options to protect the toe for a scour hole, geo-textile with rip-rap, a long sheet pile or a very long toe of concrete. In Figure 2-24 there is placed an example of a regular designed tube.

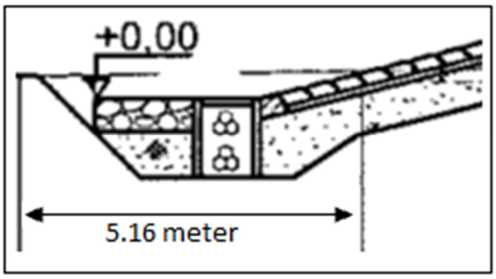

Figure 2-24 A designed toe with a concrete tube

"*The main function of a bottom protection, is not to minimize scour, but to keep the scour hole far away from the structure that needs protection*"[1]. There are many options to do this. In Vietnam a concrete tube is used. The toe is placed in a different way compared to Europe.

In Vietnam, when the water level is low, a concrete tube is placed in the correct position. Then a laborer will go inside of the hole and dig sand from the bottom out of the tube. When the tub is deep enough the hole is filled with rip-rap. From the placing until the filling there is not a lot of time available because the water level will rise. The difficulty with this system is not only the short amount of placing time, but it's also difficult to place a second concrete tube exactly next to the other. This can be seen in Figure 2-25 below. There is now a difference between the design and the placed toe structures.

[1] (Schiereck J. , 2001)

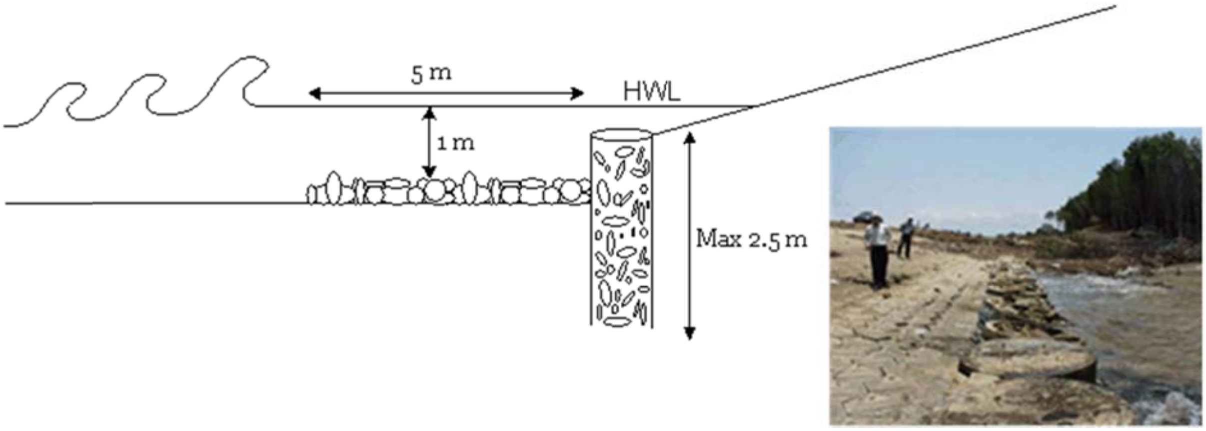

Figure 2-25 A situation sketch of the concrete tube during observations

2.6 Transportation in Vietnam

During fieldtrips a better understanding was created of the infrastructure in Vietnam. Below follows a sum up with the interesting aspects of the infrastructure. This is divided in general remarks on transportation modes and remarks on the location of the test dike: Cat Hai Island.

2.6.1 General remarks transportation by road

The traffic on the road is a mix of a lot of motorbikes and some cars. There are some traffic rules but nobody sticks to them:

- Passing is done both on the left as on the right;
- There are no ‘right-of-way rules’;
- Mirrors are rarely used by both car and motorbike drivers;
- Motorbikes can legally only be used by 2 people, in practice 4 people on one bike is not uncommon;
- Sometimes horses, cows or ox’ can be spotted near or on the roads [see Figure 2-26];
- On some parts, the roads are in very bad condition [Figure 2-26].

Figure 2-26 Remarks on transport by road

Some of the dikes are hard to reach by road, some small towns with very tiny streets have to be crossed, see Figure 2-27.

Figure 2-27 Small roads to access the dike

The maximum speed reached in urban areas is about 30 to 40 km/hour. The maximum speed reached on a 'highway' is about 60 to 70 km/hour.

2.6.2 General remarks transportation by water

The current ferry to Cat Hai Island, seen in
Figure 2-28, takes about one hour to cross the water.

Figure 2-28 Ferry to Cat Hai Island & boats near the coast of Cat Hai Island

The mangrove in front of some of the dikes can make it difficult to reach the dikes by boat as shown in Figure 2-29. At some dikes the water level can be so low, there is only mud instead of water at the bottom of the dike [see Figure 2-30].

Figure 2-29 Mangrove in front of dike, not possible for boats to go through.

Figure 2-30 Dikes during low tide

2.6.3 General remarks transportation by train

At the moment there is no railway heading directly towards the dike, making it difficult to reach. However, there is a railway to Hai Phong. An overview of the rail routes in Vietnam can be seen in Figure 2-31.

Figure 2-31 Overview of the rail road in the northern of Vietnam

2.7 Construction of revetment

Constructing and producing of a revetment in Vietnam is done primarily on site and everything is done by local laborers. The reason for this is that making the blocks on site is a lot cheaper compared to a factory because labor is very cheap in Vietnam. The decrease of quality as a result of this is not a problem for the Vietnamese.

The process of revetment construction can be split in three parts:

- The making of blocks;
- Curing and storing of blocks;
- Placement of blocks.

2.7.1 Making of blocks

The blocks in Vietnam are made on site. This is done by using a concrete mixer, wheelbarrows and formworks.

Figure 2-32 Mixing concrete and putting it into the formworks

Figure 2-33 Micing machine

Figure 2-34 Filling the formworks and the supply of raw materials

In total 11 people are producing blocks:

- 6 people working by concrete mixer: 3 for stones, 2 for sand, 1 for mixing;
- 2 people for unloading cement from truck;
- 3 people running around with a wheelbarrow with mixed concrete;
- 2 people at the formworks, one for using the vibrator, one for getting all the concrete out of the wheelbarrow.

The making of the blocks can be done by unskilled, local laborers. The cement used is PCB30. Once finished, the quality of the blocks is around C15-C20. The blocks have a density of 2200-2300 kg/m^3. A mixing machine of 250 liter can produce 10,5 m^3 of concrete per day (8 hours) if it is used non-stop (no breaks), this equals 235 Am Duong blocks per day.

2.7.2 Curing and storing of blocks

Once blocks the blocks are hard enough to be taken out of the fomework, they still have to cure for 3 weeks.

Figure 2-35 Storing and curing blocks on site

A lack of space can be a problem for the storage and curing of the blocks before they are placed at the dike. Sometimes the blocks are put on a big stock in order to be able to store more blocks. Because the blocks are cured outside, this is not a controlled process. This means the quality of the blocks decrease.

2.7.3 Placement of blocks

Once the blocks have hardened enough, they can be placed on the dike. Below are pictures from dike construction going on in the Nam Dinh Province, September 2010. The blocks are hoisted by a crane to the place of construction. This saves the workers who place the blocks an extra walk. The blocks which are placed here are Am Duong blocks with a size of 40x40x38 cm. The blocks weight 140 kg per piece. Two workers will pick up a block with a bamboo stick; a third person guides the lifters to make sure the blocks are placed correctly.

Figure 2-36 First part placing by crane

Figure 2-37 Second part placing by hand

In total 10 people are placing the blocks. They can place around 450 Am Duong blocks a dayby using three cranes. Two cranes for excavating sand, one crane for hoisting the blocks to the workers. In contrary to the making of the blocks, the placement of blocks is done by skilled laborers.

2.7.4 Dike build up

Before the blocks are placed on the dike, the dike body has to be constructed. The dike is build up from sand, with on top a geotextile layer and stones. On this layer the blocks are placed.

Figure 2-38 Placement of stone layer and geotextile

The maximum capacity for the construction of a dike/revetment is 60 workers: 20 skilled and 40 unskilled. Skilled workers are used for placement of blocks, geotextile, etc. The unskilled, local workers are used for making of blocks. Max storage area and placement of blocks is governing in the speed of constructing the dike.

2.8 Basalton® in the Netherlands

2.8.1 History of Basalton®

When basalt became scarce and transport cost increased, it got too expensive to use as dike revetment. Because basalt was running out, the quality also decreased. It became an expensive natural product.

As a replacement Basalton® was introduced in the 80's: an irregular 8 sided tapered block of strong concrete. Basalton® is the precast concrete variant of basalt.

2.8.2 Production of blocks

The materials needed to make Basalton® are sand, various kinds of gravel and cement. In Holland these materials are delivered (by company) ships and put into an automated concrete machine.

The blocks are made in factory in a controlled environment. The blocks are made by machine and under pressure. This way, no water has to be added; all the moisture comes from the sand. Because of this, high quality concrete can be achieved.

The height of the blocks ranges from 15 to 50 cm, with intervals of 5 cm.

After the blocks are made, they are put in a curing shed for 28 days. Once cured, the blocks are ready to be put on trucks or ships and transported to their destination

The strength of the blocks after 28 days of curing is around 60 N/mm^2. The standard asks for 50 N/mm^2 per stone and 45 N/mm^2 average for the set. This translates to a strength of about C60-C75.

During maximum production, 110 tons of material can be produced per hour. For blocks with a height of 25 cm, 180 m^2 can be produced per hour. For blocks with a height of 45 cm, this is around 100 m^2.

2.8.3 Placing of blocks

Often the blocks cannot be delivered at the work area, but because the blocks are placed on pallets they can easily be delivered to the work area by a forklift.

Although placing the blocks by hand is possible, laying the blocks with a machine a higher production level and constant product are achieved.

Before a set of blocks is placed, first a filter layer has to be played over the embankment, see Figure 2-39.

The first row of blocks is the so called base set and is laid along the toe of the dike, it can be seen in Figure 2-40. Because the first row of blocks needs to make a nice fit with the toe, the base set has a straight edge. The other sets of blocks don't have this.

The griper of the crane can easily be adjusted to move the blocks, and change between the shape of the first row and the other rows. The griper of crane is shown in Figure 2-41. A spotter will look on to see if the blocks are correctly placed.

Figure 2-39 Filter layer

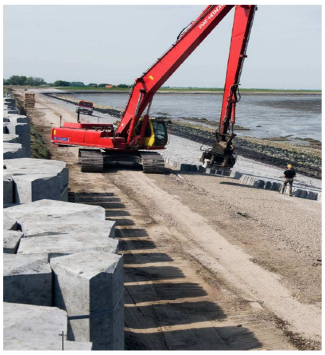

Figure 2-40 Crane placing blocks

Figure 2-41 Griper of crane

When all blocks are placed, small wash material is put in to fill the cracks. The grading of wash material is 8 -32 mm. Preferred material is basalt because of the high density (2900 – 3000 kg/m^3) of the basalt.

An experienced team can place around 500 m^2 per day. In total 1 crane and 3 people are necessary: 1 crane operator, 1 person at the pallets and 1 person at the placement of the blocks.

An experienced stone setter can place around 30 to 40 m^2 per day, depending on the height and thus weight of the blocks.

In the Netherlands, a Basalton® griper to mount on a hydraulic crane costs 35.000 euro.

2.8.4 Characteristics of blocks

Because of its design, Basalton® is difficult to move over the dike slope.

Height (cm)	15	20	25	30	35	40	45	50
Weight (kg)	310	433	554	654	736	880	990	1038

Table 2-1 Weight of blocks

Together the blocks make a polygonal connection which is easy to place and enables a quick and cheap production.

The density of the blocks is 2,3 kg/m^2. The weight of the sets depends on the height of the sets, which can be seen in Table 2-1. The blocks get their specific density from the adding of heavy admixtures to the mix.

The Basalton® which is characterized by the its polygonal link , 8 sides irregular shape, joint filling and machine placing method, modern production techniques and quality control make Basalton® a product of optimal strength and sustainability.

2.8.5 Kind of blocks

There are 4 kinds of Basalton® sets: the base sets, continuation sets, corner sets and end sets.

2.8.5.1 Base sets

Consists of 18 blocks with an area of about 1,2 m². This set is used for straight pieces of dike. The base set is used at the toe of the dike.

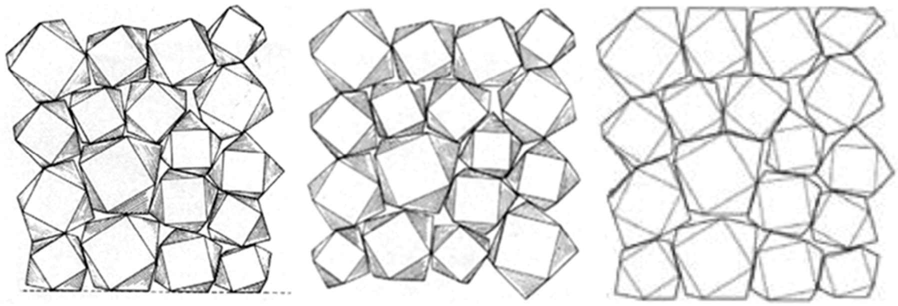

Figure 2-42 From left to right: Base set – Continuation set – Corner set

2.8.5.2 Continuation sets

Consists of 17 blocks with an area of about 1,2 m². This set is used for straight dikes.

2.8.5.3 Corner sets

Consists of 18 blocks with an area of about 1,3 m². These sets are used for making continuous bends.

2.8.5.4 End sets

Consist of 4 different blocks which can be placed on top of the last corner set to create a nice finish to the dike.

The blocks can also be used for the whole revetment. A less comprehensive griper than for the other sets can be used to place the blocks.

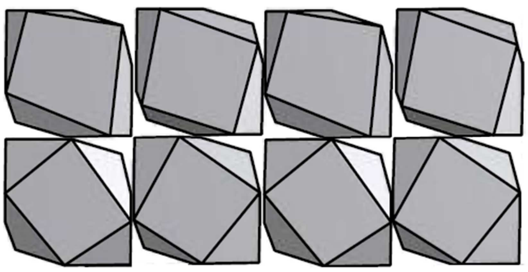

Figure 2-43 End sets

3 Boundary conditions

The scope of the boundary conditions is made for the project location on Cat Hai Island. For each boundary condition the data based on is given as well as substantiation. In the final paragraph on short overview of the boundary conditions is shown.

To design an optimal dike in Vietnam, it is necessary to know how the weather behaves in a typhoon season. The extreme conditions are very heavy in comparisons with the Netherlands. It is important to know the extreme conditions and how often they occur. Interpretation of data must be researched carefully. Because provided data can be misleading if one should just assume that the given data is correct. The reason for this is that the provided data is often gained with different methods than is common in the Netherlands and not always accurate.

As a reference level the Vietnamese use the national vertical datum 'VN-2000', established in 2000. This is a fixed reference (land) datum. The mean sea level (designated at Hon Dau tidal station) is at 0.0 m absolute level according to VN-2000. The national nautical "0" datum (at Hon Dau) is about 186 cm lower than the national land "0" datum (VN-2000). For civil engineering work including sea dikes the national land datum (VN-2000) is used.

3.1 Functional requirements

The main function of a dike is to protect the hinterland. People that life close by the dike, whom income mainly depends on fishing and agriculture wants a guaranty that there income will not be washed away after an each typhoon. Therefore the dike have to protect not only the humans but also the flora and fauna. Beside this primary function the dike has also secondary functions; one of those secondary functions is to provide a road connection between the several agricultural land. With a good road on the dike it is easier to evacuate the local people if necessary and completes and supplement the local infrastructure.

The chance of failure is the chance that a system or component under specified circumstance between a certain time period fails. When the impact on the society is relatively low it will not be necessary to protect that area just as good as a very overpopulated area. The Vietnamese guide lines use five different dike categories. Depending on the population and protected area the dike will be category I for the maximum protection up to category V for the minimum protection. The protected island Cat Hai has an area of 34,5 thousand ha and a population of more than 28.000[2]. With this data the safety standard can be taken out of the new Vietnamese guideline as shown in Table 3-1. The Safety standard for Cat Hai will be a category IV, which means that the island will be protected for a storm that occurs once in 30 years.

Note for using Table 3-1: Firstly the protected areas must be classified using the given criteria. Then the two criteria are considered in order to determine the safety standard. In case the protected area meet only one criterion, the level is lowered by one.

[2] (Wikipedia.org)

Characteristics of protected area	Safety standards (return period: years)	Dike Category
Developed industrial urban area: • **Protected area > 100.000 ha** • **Population > 200.000 people**	150	I
Rural areas having developed industry and agriculture: • **Protected area : 50.000 - 100.000 ha** • **Population: 100.000 - 200.000 people**	100	II
Developed rural and agricultural area • **Protected area :10.000 - 50.000 ha** • **Population: 50.000 – 100.000 people**	50	III
Medium-developed rural and agricultural area • **Protected area : 5.000 – 10.000 ha** • **Population: 10.000 – 50.000 people**	30	IV
Under-developed rural and agricultural area • **Protected area : < 5.000 ha** • **Population : < 10.000 people**	10 ~ 30	V

Table 3-1 Safety Standards[3]

3.2 Technical Conditions

Besides the functional requirements there are also technical conditions to take into account. For determination of the design parameter conditions are needed about the storm that occurs once in 30 years.

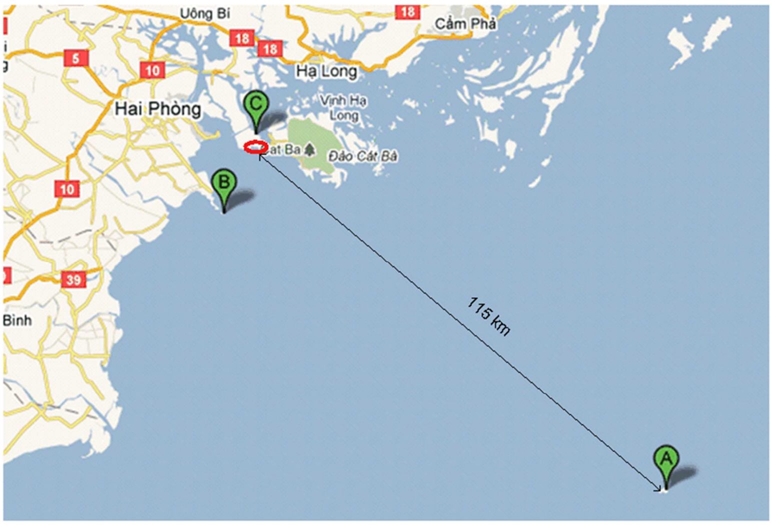

Figure 3-1: Measurement stations

There is information from three different measuring stations shown in Figure 3-1, these are the places which depicted. Point **A** is island Bach Long and **B** is island Hon Dau, at these places measuring about the wave heights over more than thirty years. Each day at 07:00, 13:00 and 19:00 hour wave heights and direction are measured also the wind speed and direction are measured. The wave dataset from Bach Long contains data from 1970 up to 2008 excluding years 1988 & 1998- 2002. The wind dataset from Bach Long contains data

[3] (CWRCT, 2010 march)

from 1970-1975 & 1996-2007. The distance between Bach Long to Cat Hai is 115 km. **C** is Dong Bai, this point is at Cat Hai Island this is the province Hai Phong. At this point the frequency of the total water level is measures.

3.2.1 Water depth

The data from the waves are measured at Bach Long Island, the bathymetry is needed to calculate the wave conditions at the toe of the dike.

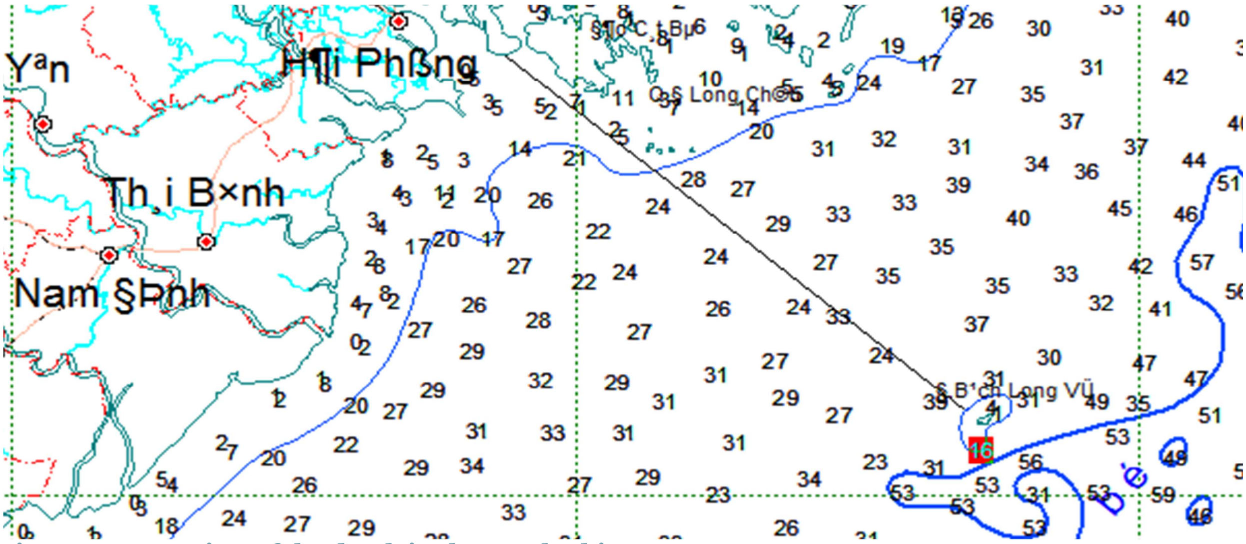

Figure 3-2 Overview of the depth in the South China Sea

Figure 3-2 shows the depths of a part of the South China Sea. The depths given are in meters and use the reference level VN-2000.

3.2.2 Wave conditions

Extreme waves are very variable in time and intensity. So for a Long-term statistical analysis extremes are estimated based on a relatively short record. For long term statistics several different methods can be used. The maximum annual wave height is a possibility but also the maximum wave height per storm is possible. Most of the time the more data is used the better and more reliable the estimations are. But this data set is not one of them. When looked at the maximum wave height for each storm result in an unreliable graph. That's way in this case the maximum annual wave height was chosen. For the maximum annual wave height a Weibull or Lognormal distribution is generally accepted.[4]

Using the annual maximum wave height the next data will be the input data:

Year	Hs[m]	Year	Hs [m]	Year	Hs [m]	Year	Hs [m]
1970	4,5	**1978**	3,6	**1986**	5,0	**1995**	5,0
1971	3,5	**1979**	4,0	**1987**	6,0	**1996**	8,0
1972	3,8	**1980**	7,0	**1989**	7,0	**1997**	6,0
1973	3,8	**1981**	4,0	**1990**	8,0	**2003**	5,0
1974	3,8	**1982**	4,0	**1991**	7,0	**2004**	5,0
1975	6,0	**1983**	6,0	**1992**	7,0	**2005**	6,0
1976	3,3	**1984**	3,0	**1993**	5,0	**2006**	1,5
1977	3,7	**1985**	5,0	**1994**	7,0	**2007**	2,0

Table 3-2 Overview of the annual maximum wave height

4 (Holthuijsen, 2008)

The maximum annual wave height is shown in Table 3-2, in the top left the biggest wave height and in the right bottom the lowest annual wave height. When the data is in order for each distribution the plotting position formula can be calculated by the method given in Table 3-3.

Distribution	Cumulative Density Function	Plotting position Method
Weibull:	$F(x) = 1 - e^{-(x/\alpha)^k}$	$F(x) = \dfrac{m - 0{,}20 - \dfrac{0{,}27}{\sqrt{k}}}{N + 0{,}20 + \dfrac{0{,}23}{\sqrt{k}}}$
Log-Normal:	$F(x) = \Phi\left(\dfrac{\ln(x) - \mu}{\sigma}\right)$ $\Phi(X) = \dfrac{1}{\sqrt{2\pi}} e^{-\frac{1}{2}x^2}$	$F(x) = \dfrac{m - 0{,}375}{N + 0{,}25}$

Table 3-3 Mathematic expressions for Wei bull & Lognormal distribution[5]

Using these plotting position formulas and the density functions the data and distributions can be plotted in one graph to see what distribution is the best fitting.

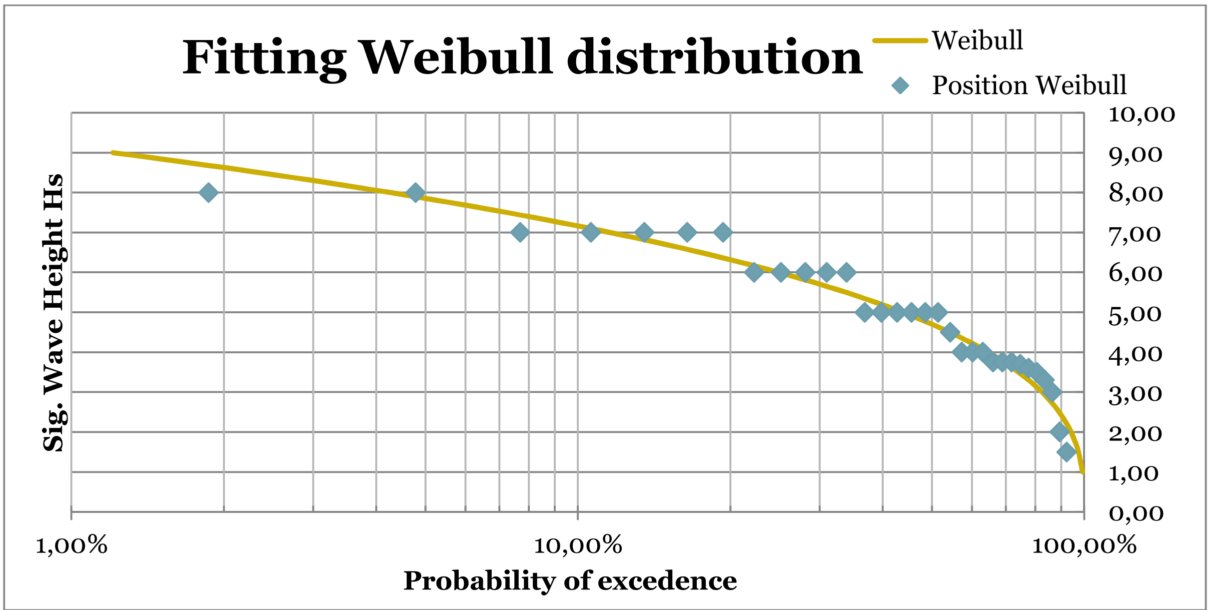

Figure 3-3 Graph with best fot for the Weibull distribution

As shown in Figure 3-3 & Figure 3-4, the best fit for the data is the Weibull distribution. Using this method the significant wave height at Bach Long Vi with exceeding change of one in 30 years will be: $Hs = 8{,}3\ m$.

Due to a lack of wave period data a period has to be estimated based on the known data. For each location of the world a specific relation can be found between wave heights and wave periods. For estimate the peak period at Cat Hai Island a formula is used from the United States. This formula is valid for typhoons in America and it is known for high outcome when used for the Vietnamese waters. By using this significant wave height at Bach Long Vi island

[5] (U.S. Army Corps of Engineers, 2008)

the peak period is calculated by the next formula[6]: $Tp = 12{,}1 * \sqrt{(Hs/g)}$. Using this formula the peak period become: $Tp = 12{,}1 * \sqrt{(8{,}3/9{,}71)} = 11{,}16\ s$.

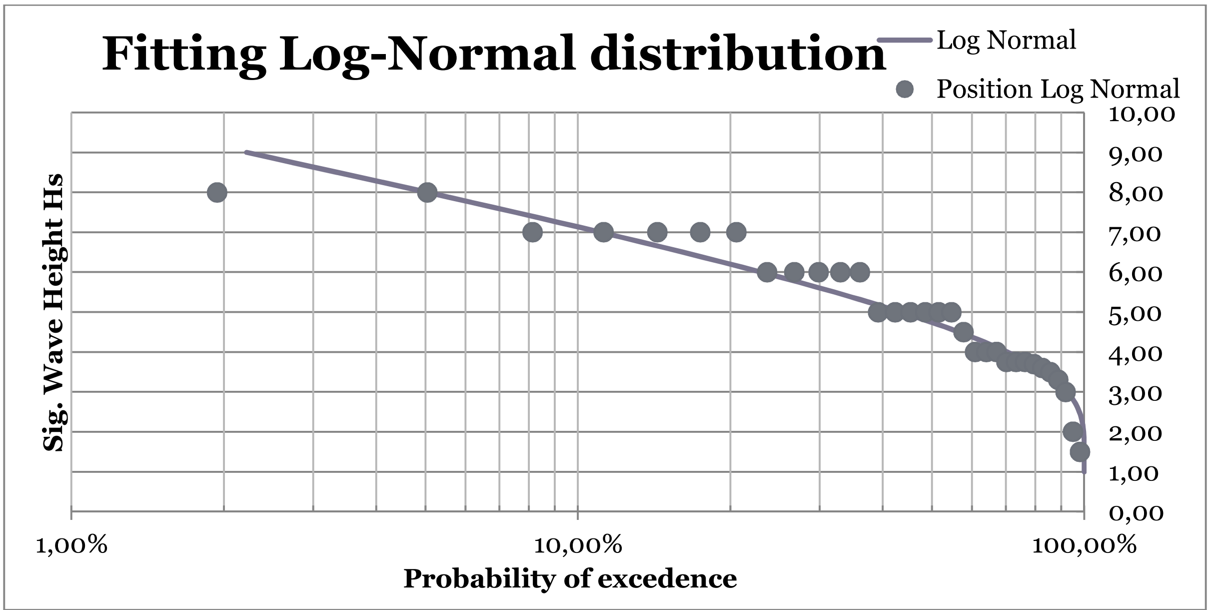

Figure 3-4 Graph with best fit for the Lognormal distribution

The design wave is calculated for the Bach Long Vi but the needed design wave at the toe is required. This toe is situated one fourth of the deep water wave length (50 m) for the dike toe. For this translation SWAN1D is used. First the translation to shallow water is made in SWAN and a second translation is done for the shallow water until the design point. The results from SWAN are seen in Figure 3-5 so the design water wave is: $Hs = 1{,}7\ m\ \&\ Tp = 11{,}16\ s$.

The direction of the waves is neglected because the direction is unpredictable and too divers to make a good conclusion. In addition, waves can be redirected by refraction due to the different water depths.

Some notes while using Swan1D. Firstly; the bathymetry is not very detailed but doesn't has a lot of influence on the wave height. As long as the bottom profile near the coast is realistic. Secondly; wind is neglected due to no visual correlation between wave height and wind speed. And when wind is taken into account the effect is negligible.

3.2.3 Tide

Tide is a regular up and down motion of the water level of the ocean. The vertical tidal movement is generated by the interaction forces between the oceans on the one hand and

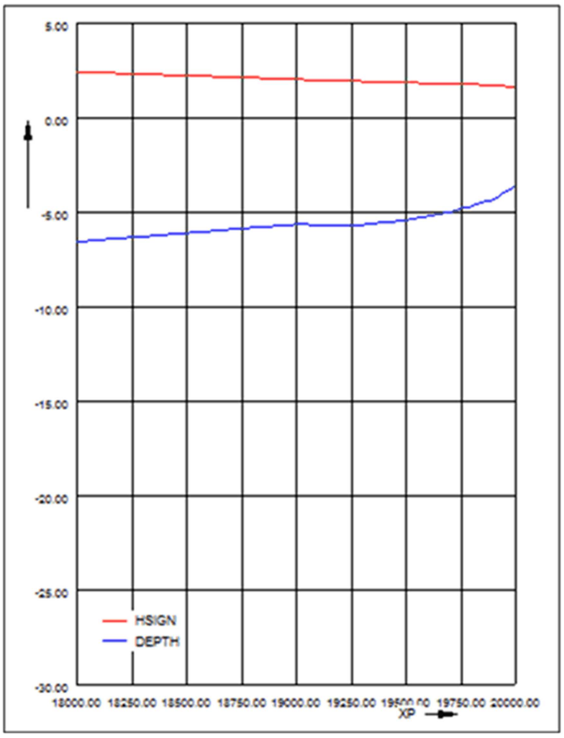

Figure 3-5 Wave height near the coast

[6] (U.S. Army Corps of Engineers, 1984)

the Moon and the Sun on the other. The tide in the Pacific Oceans is a semidiurnal tide however in Vietnam the tide is diurnal, this means that there is roughly one high and one low tide each day. For the year 2010 the tide can be seen in Figure 3-6, this is the calculated tide for Hon Dau station. The average high water level is 1,04 m + VN-2000 and the average low water level is -0,96 m + VNB-2000. The highest astronomic high tide for Hon Dau station is 2,14 m + VN-2000.

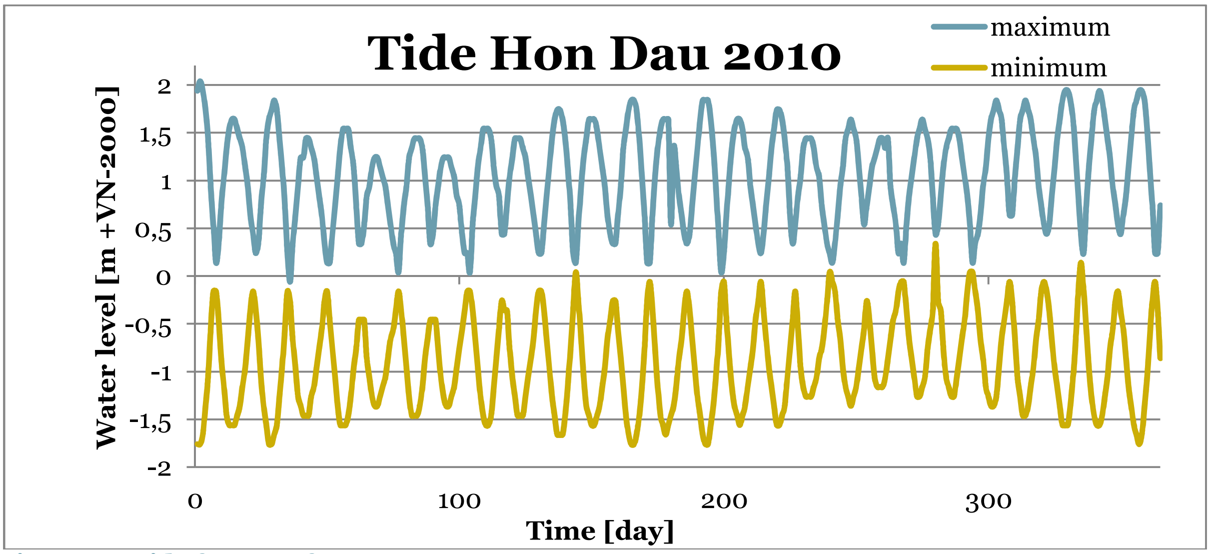

Figure 3-6 Tide for 2010 for Hon Dau[7]

Beside 2010 also the tide tables of 2008 and 2009 was inspected and year 2010 was representative for all three years.

3.2.4 Water level conditions

The water level is measured included tide and storm surge at the island Cat Hai. There are measurements from 1951 to 2007 which includes 349 storms, the parameters from these storms origin from storms that hits land from the East Sea. All the measurements are shown in the Figure 3-7. For the safety standards of 30 years there is a normative water level of 3,17 meter including surge and tide. In 2005 there was a large typhoon, the water level was during this storm 4,26 meter at Hon Dau Island (see Figure 3-1 for the exact location). The storm during the typhoon of 2005, was a storm with a frequency of once in 100 year.

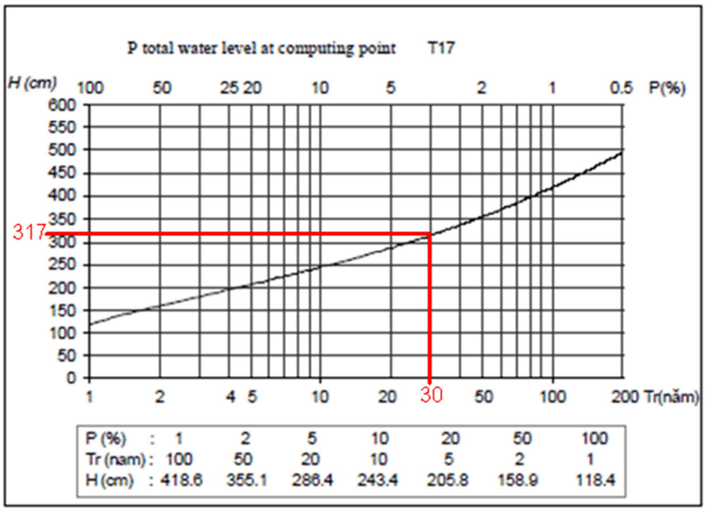

Figure 3-7 Frequency curves of high water level, Cat Hai

[7] (Marine Hydrological Center, 2009)

3.2.5 Soil conditions

In November 2000, 85 ground measurements were made to gain more information about the geological situation at the shoreline of Cat Hai. At the moment of the measurements there was already a dike. The measurements were taken at, or between, the crest and the inner berm of the dike. The soil build up is given in Table 3-4 . More detailed technical information about the different layers can been found in appendix 4. The thickness of the layers differs from measurement point to measurement point. Layer 7, 8 and 11 are local disturbances that not occur everywhere. The thickness of layer 8 and 11 differs a lot from place to place and it is not possible to give a good range of the thickness. From the last measured layer (layer 12) it is not known till how far it reaches.

Number of layer	Kind of layer	Layer thickness
Layer 1	Revetment (incl. filterlayer)	1 - 2.5 meters
Layer 2	Medium to heavy loam	1.5 – 3.0 meters
Layer 3	Clay and heavy loam mixed with organic matter	1.2 – 2.0 meters
Layer 4	Light loam, clayed sand and sand	1.0 – 1.5 meters
Layer 5	Medium and fine grained sand	≥ 4.0 meters
Layer 6	Light and moderate clay loam, mixed with organic matter	≥ 3.0 meters
Layer 7	Fine sand and clayed sand	low thickness
Layer 8	Light to medium loam	?
Layer 9	Clayed sand	≥2,2 meters
Layer 10	Light to medium loam	≥2,0 meters
Layer 11	Clayed and fine sand	?
Layer 12	Heavy loam to clay	?

Table 3-4 Soil build up (combination of measurements)

For the geotechnical calculations a single soil build up will be used, given in Table 3-5. Within this soil build up the local disturbances, layer 7 and 8, are absent. Because the revetment will be removed before upgrading the current dike to the new design, the revetment layer will be ignored for the soil build up.

Because of a lack of information and the fact the lower layers are already settled true the years, the deepest two layers will not be taken into account for the calculations.

Number of layer	Kind of layer	From [m +VN2000]	Till [m +VN2000]
Layer 2	Medium to heavy loam	+0.50	+2.75
Layer 3	Clay and heavy loam mixed with organic matter	-1.10	+0.50
Layer 4	Light loam, clayed sand and sand	-2.35	-1.10
Layer 5	Medium and fine grained sand	-6.35	-2.35
Layer 6	Light and moderate clay loam, mixed with organic matter	-9.35	-6.35
Layer 9	Clayed sand	-9.35	-11.55
Layer 10	Light to medium loam	-11.55	-13.55

Table 3-5 Soil build up (for calculations)

3.3 Summary

$Dike\ catogery\ IV\ = Withstand\ a\ storm\ that\ occurs\ once\ in\ 30\ years$
$Hs = 1{,}7\ m$
$Wave\ direction = 0°$
$Peak\ Period = Tp = 11{,}16\ s$
$Average\ high\ water\ level = 1{,}04\ m + VN2000$
$Average\ low\ water\ level =\ -0{,}96\ m + VN2000$
$Higest\ atronomical\ tide = 2{,}14 + VN2000$
$Design\ water\ level = 3{,}17\ m + VN2000$
$Wind\ set\ up = 0\ m$

4 Multiple Criteria Analyses

For the project two dikes with two different revetments will be designed. On forehand, it is determined that one of the dikes will be made of Basalton®

The goal of the Multicriteria Analyses (MCA) is to objectively determine what type of revetment is best to use for the second test dike.

4.1 Work approach

Two MCA's have been carried out. The first MCA is used to determine what kind of revetment will be used for the second test dike by comparing asphalt, concrete blocks and rip-rap. The second MCA is done to compare the Basalton® blocks with the Dutch Hydroblock and the Vietnamese TSC and Am Duong block on a number of criteria.

4.2 Assumptions

Only realistic options for sea dikes revetment are looked at. This means that grass (doesn't grow in salt environment) and mangroves (only grows in specific conditions, with brackish water. Can only be found in estuaries) will not be taken into account for the MCA.

Also the dike build up is considered to be properly designed, meaning that the choice for the best revetment alternative is not bound by the dike build up.

Cost are not looked at, because they are very dependent on the final design of the dike.

Further assumptions made can be found per MCA.

4.3 The first MCA between several alternatives

4.3.1 What to compare

The first MCA will look at different kinds of revetments. Asphalt, concrete blocks and rip-rap will be compared.

Below is a short summary per alternative.

Asphalt: Is a sticky, black liquid which is highly viscous. It's usually made for roads but it can also be used as dike revetment. In order to use this kind of revetment on a dike, heavy machinery is necessary.

Blocks: This type of revetment has several kinds of shapes and possibilities. There are however 2 main types of blocks that can be distinguished. The first one is the interlocking system, which means that the block revetment interlocks on several points with each other. The second one is a friction system, where the blocks consist of columns instead of blocks shaped like tiles.
Friction blocks are mainly placed by machines, but can also be placed by hand. In the Netherlands the blocks are placed by machine because labor costs are too expensive. In Vietnam, the interlocking blocks are mainly placed by hand because machine costs are expensive.

Rip-rap: This type of revetment consists of loose rocky material. This is dumped on site by trucks. During the dumping of the stones, they take a natural position by themselves from which they derive their resistance against wave attacks.

4.3.2 Step 1: List of criteria

A set of criteria has been made in order to compare the different type of revetments. The criteria's are listed below:

- *Maintenance:* Amount of inspection and maintenance of dike during normal conditions;
- *Constructability:* When comparing the different revetments, how easy each revetment can be placed correctly;
- *Building time:* How quickly the revetment can be placed, compared by using the same amount of workers, in m²/day;
- *Ecology:* Damage to nature and habitat;
- *Repairment:* How easy, compared the alternatives to each other, the revetment be repaired when it is partly broken;
- *Durability:* Is the ability to endure. In this case, the lifespan of the revetment in terms of material;
- *Costs: The total cost of the production, transport, placing and maintenance.*

It is hard to make a realistic analyze of the total cost for the different revetments. Because there can't be made a realistic approximation of those costs, it is not possible to make a good distinction between the different types of revetments. Because of those reasons costs are not taken into count within this multiple Criteria Analyses.

4.3.3 Step 2: Determining weight factors

The importance of the different criteria is determined by comparing each criterion to the other criteria's. When comparing, one of the criteria will get the advantage above the other. It is important to note that this process has been done as objective as possible. It might well be that someone else favor other different criteria than the one below. Table 4-1 gives the result of the weight factor.

Weight factors	Maintenance	Constructability	Building time	Ecology	Repairment	Durability	Weight factor
Maintenance		1	1	1	0	0	**4**
Constructability	0		1	1	0	0	**3**
Building time	0	0		1	0	0	**2**
Ecology	0	0	0		0	0	**1**
Repairment	1	1	1	1		1	**6**
Durability	1	1	1	1	0		**5**

Table 4-1 Determination of the weight factor for the criteria

The figure can be read as follows:

- The *maintenance* is more important than constructability, resulting in a (1) in the row of *maintenance*. Consequently, in the row *constructability* with column *maintenance*, a (0) will be placed. Comparing all the alternatives will result in a matrix with (0)'s and (1)'s.

Counting the (1)'s per row will result in the weight factor, which will later be used to determine the weighted MCA's.

All the criteria are taken into account for the weighted MCA, that's why all criteria are awarded 1 point to start with.

4.3.4 Step 3: Making unweighted MCA

In this step the three different revetments will be compared. The alternative which scores the best on a criterion will be rewarded with a (3), the second best alternative will get a (2) and the worst alternative will get a (1).

This results in the following unweighted MCA:

Unweighted	Asphalt	Blocks	Rip-rap
Maintenance	2	3	1
Constuctability	1	2	3
Building time	2	1	3
Ecology	1	2	3
Repairment	1	2	3
Durability	1	2	3
Total points	**8**	**12**	**16**

Table 4-2 Unweighted MCA

4.3.4.1 The unweighted criteria explained:

Maintenance: Rip-rap has to be checked every once in a while, because it loses its structure. Asphalt suffers from fatigue due to the high temperatures throughout the year in Vietnam. In general, the blocks have the most predictable wear down over the years, which means that blocks need less inspection and maintenance.

Constructability: Rip-rap doesn't have to be placed in a specific order, making it easy to construct. Blocks, when only looking at the placement, are relatively easy to place in order. Asphalt always needs large equipment in order to be laid properly, making it the worst alternative when looking at constructability.

Building time: Rip-rap can be dumped by trucks, making it a fast process. The placement of asphalt is a complicated continues process, once the machine is running big slabs can be made in a relatively short amount of time. Not looking at the placement, blocks are either placed by hand or machine which can be placed relatively easy (depending on the type of block). However, compared to the two other alternatives, blocks have the longest building time.

Ecology: Rip-rap is a natural product and flora and fauna can find a habitat between the open areas of the rocks. Blocks are ranked second because this alternative is less harmful to nature and environment when compared to asphalt. Asphalt is potentially harmful to nature during the process of placement and decay.

Repairment: Rip-rap is easy to repair because new stones can just be dumped on top of the old rip-rap. Damaged segments of blocks can be replaced. Finally, in order to replace asphalt large equipment is necessary and replacement involves replacing large slabs.

Durability: When looking at durability in terms of material, rip-rap is the strongest because it suffers less from abrasion compared to concrete blocks. Blocks come after rip-rap, because they don't suffer from fatigue like asphalt.

4.3.5 Step 4: Making a weighted MCA

In order to see what alternative is best, the unweighted MCA has to be multiplied by the weight factors found in step 2.

Doing this gives the result which can be found in Table 4-3.

Weighted	Asphalt	Blocks	Rip-rap
Maintenance	8	12	4
Constuctability	3	6	9
Building time	4	2	6
Ecology	1	2	3
Repairment	6	12	18
Durability	5	10	15
Total points	**27**	**44**	**55**

Table 4-3 Weighted MCA

From the MCA it follows that when all the scores are added and weighted, that rip-rap is the best option for the second test dike. Blocks are the second best alternative. Asphalt is the worst alternative.

Rip-rap is also a good option for the second test dike because this way an alternative already used in Vietnam can be compared to a new revetment.

4.4 MCA: Comparing Basalton® to other blocks

4.4.1 What to compare

This MCA will look at different kinds of blocks. There will be two friction Dutch blocks, Basalton® and Hydro-blocks, and two Vietnamese interlocking blocks: TSC and Am Duong. Although one of the dikes will already have Basalton® blocks, making it take part in the MCA gives a good comparison to the other blocks. Below is a short summary per alternative.

4.4.1.1 Basalton®

Basalton® columns are made up from 18 blocks with a total area of 1,3 m² . The blocks are made in factory in a controlled environment. The blocks are made by machine and under pressure. This way, no water has to be added; all the moisture comes from the sand. Because of this, high quality concrete can be achieved. Blocks are placed by a crane. On straights different sets of columns can interlock on 4 sides, in corners the top and bottom are flat so the columns only interlock on 2 sides. The height of the blocks ranges from 15 to 50cm. The strength of the concrete, when produced in a factory, is about C60-C75.

The Basalton® blocks that are used for the MCA are the so called *starting* and *continuation sets*. The *end sets* are not taken into account.

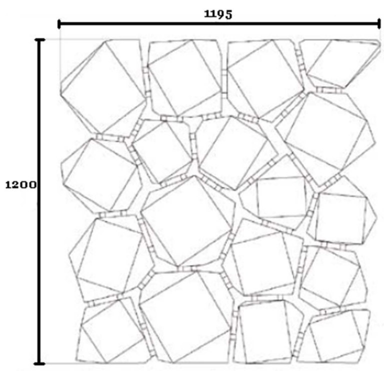

Figure 4-1 – Set of Basalton® corner blocks

4.4.1.2 Hydro-blocks

Hydro-blocks lend their strength from friction. The blocks are made in a factory under a controlled environment. The shapes of the blocks allow them to easily follow the outline of the dike. For an area of 1,3 m^2, 25 Hydro-blocks are needed. The height of the blocks ranges from 15 to 50cm. The strength of the concrete, when produced in a factory, is about C60-C75.

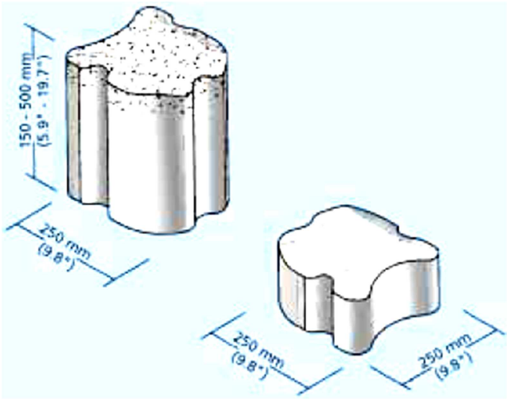

Figure 4-2 – Hydro-blocks

4.4.1.3 TSC:

TSC uses the interlocking system to withstand wave attacks. The blocks are made on site. The precast concrete blocks have the surface shape similar to bee hive. The blocks can have a thickness of 18, 26 and 36 cm. The strength of the concrete, when produced on site, is about C15-20.

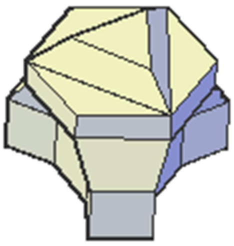

Figure 4-3 – TSC block

4.4.1.4 Am Duong

The Am Duong block relies on its interlocking system with other blocks to withstand wave attacks. The interlocking system is just like the tiles on the roof, the blocks are made on site. The blocks are 40x40x24 cm and the strength of the concrete, when produced on site, is about C15-20.

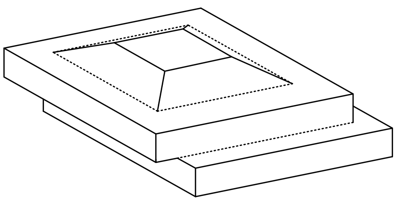

Figure 4-4 – Am Duong block

4.4.1.5 Assumptions

For the Dutch blocks, Dutch quality is expected. That means the blocks are made in a factory under controlled conditions. For the Vietnamese blocks, it is assumed that they are made on site. Of course, the Dutch blocks could be produced on site resulting in a lower concrete quality. Likewise, the Vietnamese blocks could be produced in a factory resulting in a better concrete factory. However, for this MCA, these possibilities are not looked at.

Basalton® and Hydro-blocks are compared. There are more blocks, such as Ronaton and C-star, but they have basically the same characteristics as Hydro-blocks, so only Hydro-blocks are looked at. For the Vietnamese blocks TSC and Am Duong are compared.

It is assumed that the Basalton® blocks are placed by a machine, the Hydro-blocks can be placed either by machine and hand. The Vietnamese blocks are placed by hand.

Besides these assumptions the dike build up just like with the first MCA is considered to be properly designed, meaning that the choice for the best block revetment alternative is not bound by the dike build up.

4.4.2 Step 1: List of criteria

The second MCA is used to check whether Basalton® is a good solution, compared to the Hydro Block and the two Vietnamese blocks.

The blocks are compared to the following criteria:

- *Stability of blocks:* How blocks resist uplift forces between them during storms.
- *Placing blocks:* How easily the blocks can be placed correctly.
- *Repairment:* How easy the blocks can be repaired or replaced when broken.
- *Use of material:* How much material per m^3 of revetment.
- *Making bends:* Whether the blocks are suitable to be placed in bends.

4.4.3 Step 2: Determining weight factor

The importance of the different criteria is determined by comparing each one to the other. When comparing, one of the criteria will get the advantage above the other. It is important to note that this process has been done as objective as possible. It might well be that someone else favor other different criteria than the one below.

Table 4-4 gives the results and can be read as follow:

- The *stability of blocks* is more important than *placing blocks,* resulting in a (1) in the row of *stability of blocks*. Consequently, in the row *placing blocks* with column *stability of blocks*, a (0) will be placed. Comparing all the alternatives will result in a matrix with (0)'s and (1)'s.

By counting the (1)'s per row, will result in the weight factor which will be used later to determine the weighted MCA's.

All the criteria are taken into account for the weighted MCA, that's why all criteria are awarded 1 point to start with.

MCA blocks	Stability of blocks	Placing blocks	Repair-ment	Use of material	Making bends	Weight factor
Stability of blocks		1	1	1	1	**5**
Placing blocks	0		0	0	1	**2**
Repairment	0	1		1	1	**4**
Use of material	0	1	0		1	**3**
Making bends	0	0	0	0		**1**

Table 4-4 Determination of the weight factor for the criteria

4.4.4 Step 3: Making un-weighted MCA

In this step the three different revetments will be compared. The alternative which scores the best on a criterion will be rewarded with a (4), the second best alternative will get a (3) and ultimately the worst two alternatives will get a (2) and (1), the last one being the worst.

Unweighted	Friction		Interlock	
	Basalton	**Hydro**	**TSC**	**Am Duong**
Stability of blocks	3,5	3,5	2	1
Placing blocks	3	4	1	2
Repairment	4	3	1	2
Use of material	1,5	1,5	4	3
Making bends	3	4	2	1
Total points	**15**	**16**	**10**	**9**

Table 4-5 Unweighted MCA

4.4.4.1 The unweighted criteria explained:

Stability of blocks: Basalton® and Hydro-blocks gain their strength with friction, these two blocks provide holes where water can freely flow through from the wave impact. The TSC and Am Duong gain their strength by interlocking with each other. Due to this interlocking system sea water cannot easily flow around it after a wave impact. Therefore uplift forces can become high under this interlocking system.

Placing blocks: The placement of Hydro-blocks is relatively easy because the blocks consist of just one shape, which doesn't have to be interlocked and can be placed next to each other. The Am Duong block also has the same shape, but its interlocking system gives it the second place for this criterion. The third easiest block to place correctly is the Basalton® block, this consists of 18 small pieces which have to be placed by machine to guarantee its strength. The TSC block also has the same shape, but has more interlocking points per block. Also, the TSC and the Am Duong needs to be placed by hand, the Hydro-blocks can be either be placed by hand or machine.

Repairment: Because Hydro-blocks consist of columns with the same shape, replacing blocks that are worn down or broken can be done quite easily. The second one is the Am Duong block, the interlocking system does make it more difficult than a friction system. However the Am Duong is only interlocked on 2 sides, meaning that it can be replaced easily, just likes roof tiles. Basalton® is a friction system, the reason for the third place is because of the 18 different blocks per column of 1,3 m². If one should break or get damaged, this puzzle needs to be restored which can be difficult for unskilled workers. Last is the TSC block because it has 3 interlocking places, this makes it difficult, almost impossible, to replace if only one get damaged.

Use of material: The use of material is higher for friction blocks than for interlocking blocks. The reason for this is because interlocking blocks are much lower than the column blocks which are higher. Therefore the material use to produce one m² of interlocking blocks is smaller than one m² of friction blocks. The Am Duong has a size of 40x40x24 cm and the dimensions of TSC are 40x40x24 cm. Meaning that de Am Duong needs more material then the TSC. Basalton® and Hydro are more or less the same, seen from the open spaces between the blocks, Basalton® has around 10-12% of open spaces between the blocks. The Hydro-block has about 10-15% of open spaces. This means that with the same height, the material use over one m² that Basalton® is equal to Hydro-blocks.

Making bends: Hydro-blocks can easily be placed in bends, using the same blocks as for the straights. Basalton® can also be placed in bends by using special bend patterns with a flat top and bottom row. It does, however have to be placed by machine. TSC and Am Duong blocks are not suitable for making bends. Instead a reinforced concrete slab is used to fill up the gap between two straights.

4.4.5 Step 4: Making weighted MCA

Now by using the weight factors, a weighted MCA can be found. This gives the following result:

Weighted	Friction		Interlock	
	Basalton	Hydro	TSC	Am Duong
Stability of blocks	17,5	17,5	10	5
Placing blocks	6	8	2	4
Repairment	16	12	4	8
Use of material	4,5	4,5	12	9
Making bends	3	4	2	1
Total points	**47**	**46**	**30**	**27**

Table 4-6 Weighted MCA

From Table 4-6 it follows that Basalton® and Hydroblocks are both equally good alternatives. It is also clear that the Vietnamese interlocking blocks are not as good as the friction blocks with high strength.

The difference within the friction blocks and interlock is very small. This is because they share a lot of characteristics.

5 Geometry: dimensions of the dike body

In the current situation most of the Vietnamese sea dikes are designed like river dikes with a heavy revetment (Figure 5-1). The sea dikes have an inner berm instead of an outer berm. Inner berms are effective for river dikes, but for sea dikes outer berms are far more efficient. With knowledge of the current situation, it will be interesting to design a sea dike based on a better geometry.

Figure 5-1 Sea dike at Cat Hai Island.

The geometry of a dike has influence on wave run-up, wave overtopping and sliding/instability. Because of the close relations between those failure mechanisms and the geometry, it is interesting to take a closer look to an optimization of the geometry instead of taking standard dimensions.

The geometry of the dike can be divided into different parts of the dike, see Figure 5-2. Among the different parts the crest level is the most important. What the required height of the crest level will be depends, beside the given boundary conditions like set-up and waves, on the geometry of the dike. An outer berm and the inclination of the slopes have influence on the needed crest height.

Of course safety is the most important goal for the dike design, but an unnecessary high dike will result in high costs. Within this chapter an optimization will be made between, crest level, the use of an outer berm, the inclination of the slope and the amount of materials needed to build the dike. The optimization will be made with the calculations of the wave run-up and wave overtopping.

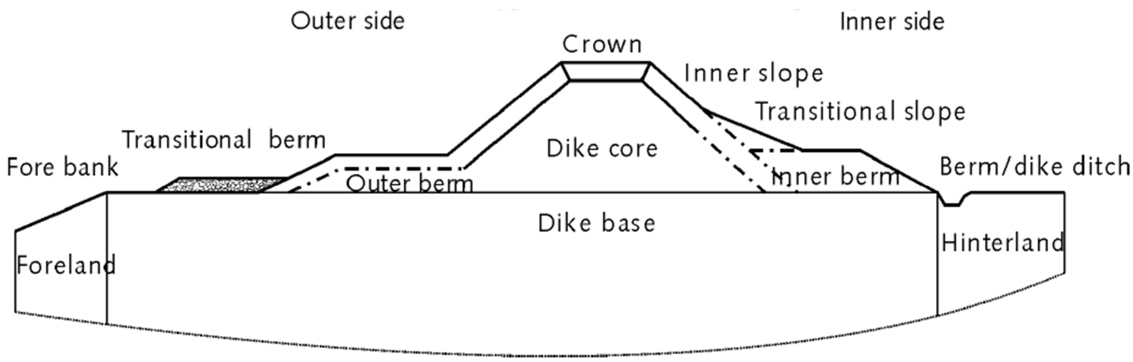

Figure 5-2 General geometry of a dike[8]

5.1 Calculations

The calculations of the wave run-up and wave overtopping are based on the Vietnamese guideline ("Technical standards in sea dike design") and on background information from "Technical report wave run-up and wave overtopping at dikes" (TAW, May 2002).

Four Matlab scripts are made to determine the optimized dike design, the scripts can been found in appendix 15. The first two scripts are optimizations of the geometry, where no wave overtopping is allowed. The other two are optimizations with an allowed wave overtopping discharge.

[8] (Tonneijck & Weijers, 2009)

The first and third script determines the influence of the berm on the needed crest level and as a result of that also the needed material for construction. The second and fourth script calculates the same things however this time the berm is not variable but the inclinations of the slopes.

The script calculates the crest height with the next formula:

$$Without\ overtopping: Z_{dp} = Z_{tk} + R_{up} + a$$

$$With\ overtopping: Z_{dp} = Z_{tk} + R_{cp} + a$$

Z_{dp} = Crest height
Z_{tk} = Design water level
R_{up} = Wave run-up
R_{cp} = Crest freeboard
a = Safety height increment

To determine the wave run-up or crest freeboard, the chosen variables, width of the berm, berm level and the inclinations of the slope are needed. The Matlab file calculates the wave run-up / crest freeboard and with that the needed crest level for a range of values for the variables. In combination with the use of material an optimal can be chosen.

For the calculation of the optimal berm width and berm level, the inclination of the slopes is needed and the other way around. So an optimum has to been found by iteratively calculating optimal values in the Matlab files.

5.2 Boundary conditions

For the calculations, boundary conditions are needed. These are discussed in chapter 3.

In chapter 3.1 is explained that the dike design has to meet the requirements for a class IV dike. This means that it has to withstand a storm with a frequency of once in 30 year.

It results in the following hydraulic boundary conditions (see paragraph 0):

$Z_{tk} = 3.17m + M.S.L.$ $H_{m0} = 1.70m$ $T_p = 11.16s$ $\beta = 0°$
$Z_{tk} = Design\ water\ level$ $T_p = Peak\ period$
$H_{m0} = Design\ wave\ height\ at\ the\ toe$ $\beta = Angle\ of\ incident\ waves$
Some of the geometric boundary conditions are given by the Vietnamese guideline. Those are related to the dike class. Table 5-1, from the guideline, give the following conditions:

- Safety height increment: $a = 0.3\ m$
- Minimal width of the crest: $B_{crest} = 4\ m$

In Vietnam the crest of the dike is used as place for a road. On Cat Hai Island the road will only be used for maintenance, but on many places the road on top of the dike is used as an important connection. In those cases it has to be possible for two vehicles to pass each other. In that case a wider crest will be necessary; the effect of such a wider crest will be discussed later on.

Safety grade	I	II	III	IV	V
Safety height increment (a)	0,5	0,4	0,4	0,3	0,3
Dike crest width B_0 (m)	6~8	6	5	4	3

Table 5-1 Safety height increment and minimal crest width[9]

When wave overtopping will be allowed, it is important that the wave overtopping doesn't damage the dike. The overtopping waves can damage the inner slope, to prevent damage to the inner slope the amount of water that will overtop the crest may not be too large. In our design the inner slope of the dike has a grass cover on a clay layer. There is a lot of research done to the strength of grass revetments in the last years. The strength of the grass depends on the quality of the grass layer. The discharge that can be handled by a grass layer is 1 till 10 l/s/m, see Table 5-2 (results based on Dutch tests). Because the grass layers in Vietnam are not always of a good quality and the types of grass differs from the tested Europe's kinds of grass, the lower limit is taken as an allowable overtopping discharge:

Allowable overtopping discharge $q = 1\ l/s/m$

Quality of the inner slope	Allowable average overtopping discharge q (l/s/m)
Undefined quality, non-protected slope	< 0,1
Well-grown grass on clay base layer slope	< 1,0 ~ 10,0
Reinforced slope	< 50,0 ~200,0

Table 5-2 Allowable overtopping discharge[10]

The last parameter that has to be determined is the roughness of the revetment. The roughness factors of all the different revetments are determent by TAW and can been found in; "Technical report wave run-up and wave overtopping at dikes", Appendix 1. Besides Basalton®, the second type of revetment is rip-rap, the roughness of both types of revetments is given in Table 5-3.

Type of revetment	Roughness factor (γ_f)
Basalton	0.90
Rip-rap	0.55

Table 5-3 Roughness factors[11]

The optimization is related to the amount of material needed for the construction of the dike (per meter). Because it is hard to find an optimum when looking at the different materials, the use of materials is translated to costs. A first estimation of the prices for the different elements can been found in Table 5-4. Within this optimization a simplification is made by taking a fixed price for the block revetment. The price of the blocks change when they become thinner or thicker in reality.

Element	Price
Soil (for the dike body)	130.000 VND/m^3
Block revetment	570.000 VND/m^2
Rip-rap	270.000 VND/m^3
Buying land	140.000 VND/m^2

Table 5-4 First estimation of the prices of the different elements of the dike

[9] (CWRCT, 2010 march)
[10] (CWRCT, 2010 march)
[11] (Technische Adviescommissie voor de Waterkeringen, 2002)

5.3 Effect of the outer berm

When a berm is designed well, the berm will break the incoming waves. So when a berm will be constructed the needed crest level can be lower. Within the analyze of the effect of the berm, two variables are analyzed;

- The width of the berm;
- The level of the berm (in comparison with the design water level).

Within the optimization calculations have been made for a berm width of 0,0 m (no berm), till 20,0 m. This range holds all the realistic widths.

The optimal effect of a berm will be reached when the berm is constructed at the water level of the design storm. When applying a lower or higher berm level, the berm width need to be longer to get the same crest level. An optimization with respect to the level of the crest can been found in

Figure 5-3.

When a shorter berm (or no berm) will be constructed, the crest level needs to be higher to get the same safety level. When space is limited it can be interesting to construct a higher dike with a smaller berm. But within the optimization made, the berm width will be chosen on behave of an optimization of the cost of the dike (see

Figure 5-3).

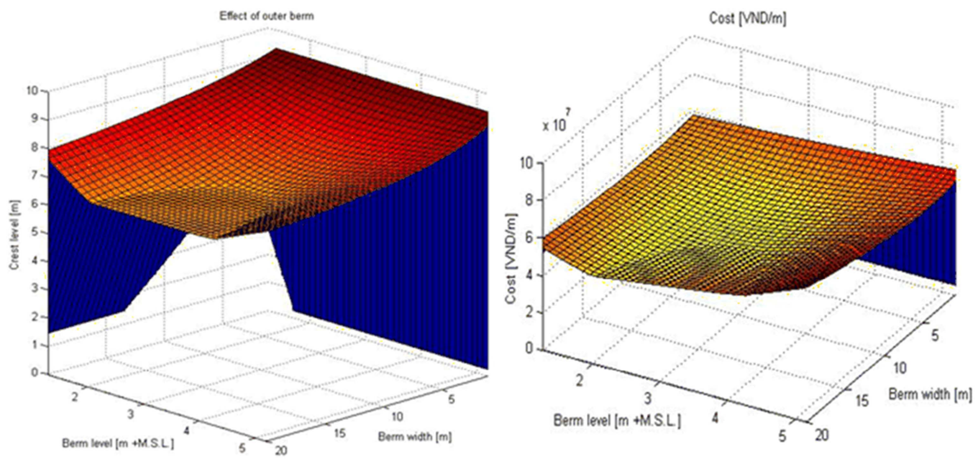

Figure 5-3 Effect of the berm on the crest level (left) & Cost optimization (right), for Cat Hai Island pilot project

5.4 Effect inclinations of the slopes

The outer slope of the dike is divided in two parts;

- The inclination of the slope beneath the berm;
- The inclination of the slope above the berm.

A smoother/longer slope results in a lower wave run-up, but will result in a wider dike. To minimize the crest level, the slopes need to be as gentle as possible (see Figure 5-4). But from the perspective of cost the optimization looks somewhat different. An optimal cost will be reached when the slope above the berm is steep and the slope beneath the berm is gentle (seeFigure 5-4). The total slope absorbs wave energy, but it is cheaper to make a long slope on a low level than on a high level (because of the amount of soil needed).

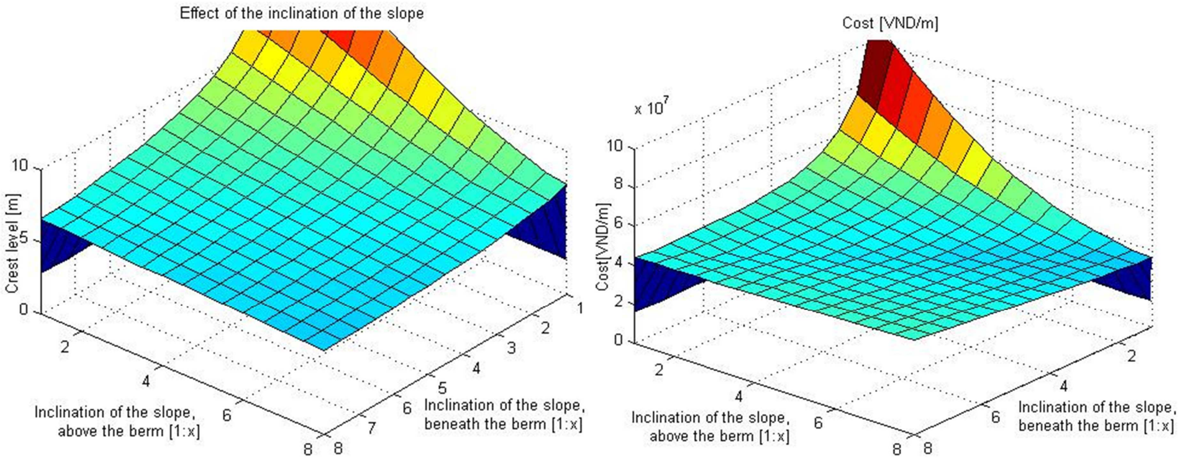

Figure 5-4 Effect of the inclination of the slope on the crest level (left) & Cost optimization (right) , for Cat Hai Island pilot project

From a practical point of view the inclination may not be too steep because it must be possible to place the revetment on the slope. Besides that, a steeper slope will result in heavier blocks, because the main stability of revetments comes from the own weight of the stones. The distribution of the forces to the subsoil depends on the inclination of the slope, see Figure 5-5. From technical point of view a maximum steepness of 1:2.5 is allowed. But when this steepness would be applied, the needed revetment will become extremely thick. What will result in high costs. Extreme thick blocks are hard to make and to place. From this last point of view the maximum allowable steepness of the slope is chosen as 1:3.5. With this maximum the optimization is made.

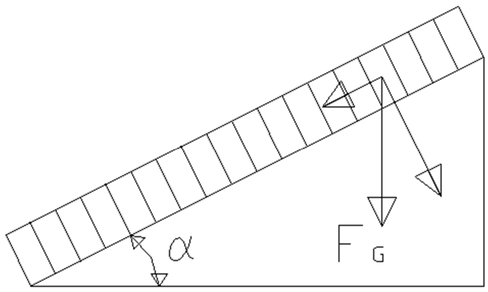

Figure 5-5: Distribution of forces

5.5 Conclusion

An optimization of the geometry is made for different situations, from two different points of view. The following designs are optimized:

- Dike with block revetment, designed with guideline;
- Dike with block revetment, with extra wide crest;
- Dike with rip-rap revetment

The optimizations of these three different designs are made for a situation with and without allowable overtopping. The results of the optimizations can been found in Table 5-5, Table 5-6 and Table 5-7.

Within the new Vietnamese guidelines two design methods are mentioned, with and without allowing overtopping. When (limited) overtopping is allowed, the dikes can be lower. This can be seen when comparing Table 5-5 and Table 5-6. But also in the rip-rap design, the crest level of the design with allowable overtopping is considerably lower, see Table 5-7.

When for example a busy road has to be constructed on top of the dike, a wider crest will be needed. An option is to just make the crest wider, but this will not result in an optimal design (from cost perspective). A wider crest means more material under the crest. Because the amount of sand becomes larger it becomes interesting to build a lower dike. To build a lower dike, but with the same safety level, a wider berm and/or smoother slopes are needed (see Table 5-5 and Table 5-6).

To get a better impression of the differences between the optimizations, drawings are made for the six situations (see Figure 5-6 and Figure 5-7).

Basalton dike without overtopping	Guideline	Extra crest width (Crest width = 7m)
Crest level (+ VN 2000)	7.10 m	6.55 m
Berm width	10.0 m	11.5 m
Berm level (+ VN 2000)	2.90 m	2.90 m
Inclination of the slope beneath the berm	1 : 5	1 : 6.5
Inclination of the slope above the berm	1 : 3.5	1: 3.5

Table 5-5 Optimization of the geometry, without overtopping (for Cat Hai Island pilot project)

Basalton dike with overtopping	Guideline	Extra crest width (Crest width = 7m)
Crest level (+ VN 2000)	6.05 m	5.95 m
Berm width	9.5 m	10.5 m
Berm level (+ VN 2000)	3.00 m	2.90 m
Inclination of the slope beneath the berm	1 : 5.5	1 : 5.5
Inclination of the slope above the berm	1 : 3.5	1: 3.5

Table 5-6 Optimization of the geometry, with allowable overtopping (for Cat Hai Island pilot project)

Rip rap dike	Without overtopping	With allowable overtopping
Crest level (+ VN 2000)	6.15 m	5.40 m
Berm width	0.0 m	0.0 m
Berm level (+VN 2000)	1.2 m	1.2 m
Inclination of the slope beneath the berm	1 : 8	1 : 8
Inclination of the slope above the berm	1 : 3.5	1 : 3.5

Table 5-7 Optimization of the geometry, for a revetment of rip-rap (for Cat Hai Island pilot project)

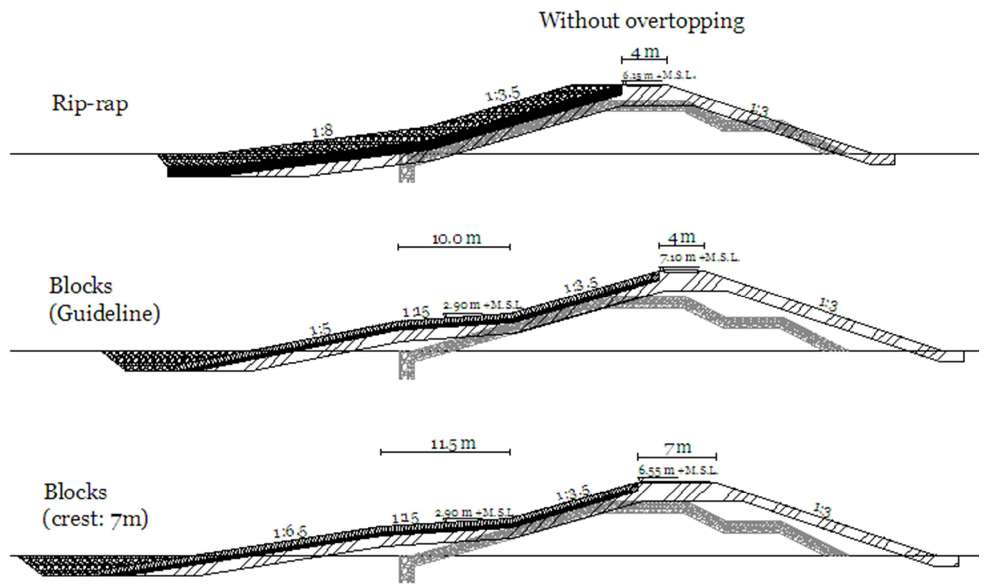

Figure 5-6 Drawings of the three optimizations without overtopping (for Cat Hai Island pilot project)

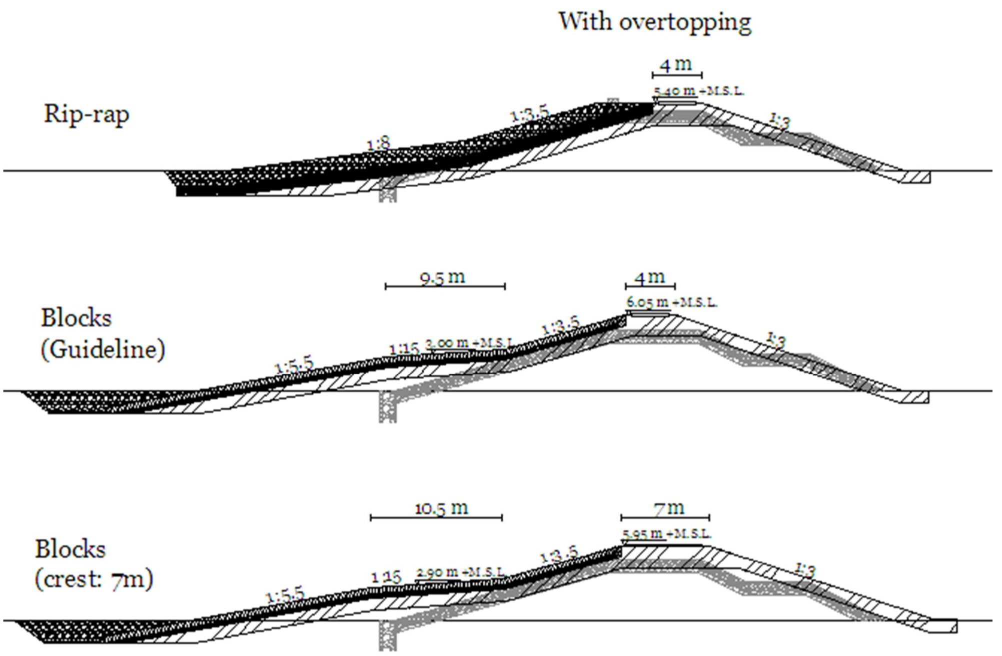

Figure 5-7 Drawings of the three optimizations with allowable overtopping (for Cat Hai Island pilot project)

At this moment there are still uncertainties about the quality of the inner slope. For this reason we do not allow overtopping and the design further on in this report is based on the optimizations of the geometric situations without allowable overtopping.

The guideline requires a minimal crest width of 4m, what will be enough for a road for the limited amount of traffic on Cat Hai Island. So the two designs which are designed further on are:

- Dike with block revetment (crest width, 4m), without overtopping;
- Dike with rip-rap revetment (crest width, 4 m), without overtopping.

6 Geotechnical aspects

One of the main parts of the dike design is the soil body. Within this chapter the geotechnical part will be designed and tested for the failure mechanisms; macro stability and settlement (see Figure 6-1).

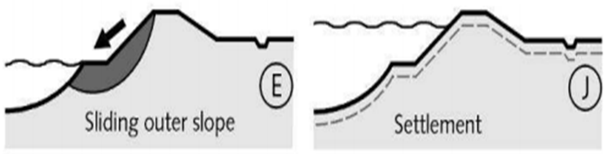

Figure 6-1 Possible failure mechanism[12]

6.1 Geotechnical design

In chapter 5, the geometry of the dike is designed. The buildup of the soil is defined with the following steps;

- The current dike is re-used as much as possible. The already existing soil layers will be used as base for the new dike. The revetment and filter layers will be removed;
- On the seaside a hard revetment (with filter layers) will be placed (the design of the revetment can been found in chapter 7), under this revetment a clay layer of 1 meter will be placed;
- On the landside of the dike an 1 meter thick clay layer will be placed, with on top a grass layer;
- Between the filter layer and the clay layer a geotextile will be placed;
- The core of the dike, the space between the old soil layers en the clay layer, will be filled with sand.

The mentioned steps results in the following geotechnical design of the dike (for the block revetment), see Figure 6-2. The geotechnical design of the dike with the rip-rap revetment is made with the same principles. Drawings can been found in appendix 16.3 to appendix 16.6.

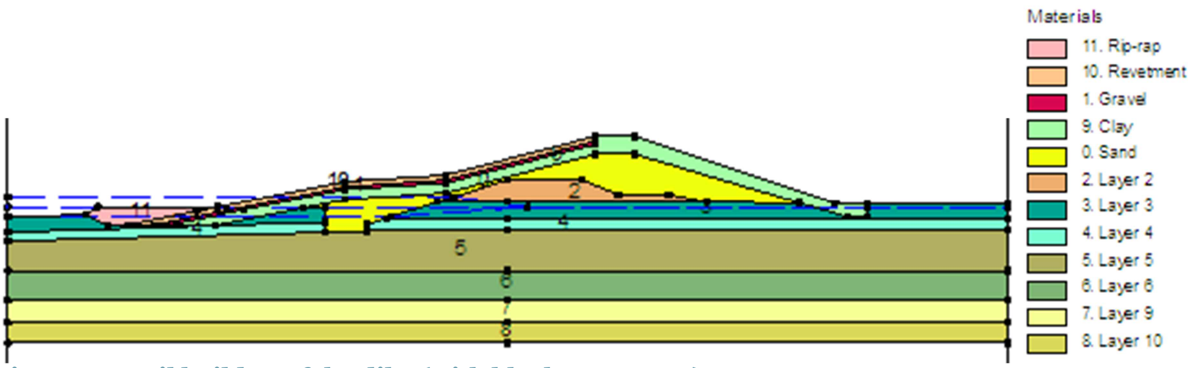

Figure 6-2 Soil buildup of the dike (with block revetment)[13]

6.2 Introduction to the calculations

The design is tested on stability and settlement. For the calculation of the stability the program MStab is used and for the calculation of the settlement MSettle. Both programs are from the M-series, a series of programs for geo-technical calculations. The programs are

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[12] (Tonneijck & Weijers, 2009)

[13] Figure created with the program MStab

developed by the company Deltares (formerly GeoDelft). Explanations of the calculations and the programs are explained in the paragraphs about the stability and settlement.

6.3 Properties and loads

For the calculations of the stability and the settlement several parameters are needed. The current soil buildup and the properties of the current soil layers are explained within the boundary conditions, see appendix 14. For the new soil layers the parameters are determined on standard values and can been found in Table 6-1.

Soil type	γ_d [kN/m³]	γ_n [kN/m³]	c [kN/m²]	Φ [°]	k [m/dag]	C'p [-]	C's [-]
Clay	14	14	16	20	8,5 x10^{-6}	15	160
Sand	18	20	3	29	4	600	-

Table 6-1 Soil parameters, for the new layers

The maximum load on the dike will occur during the building phase. This load mainly exists out of the weight of the equipment (mobile cranes, trucks, etc.). The time that the equipment will be at the same place on the dike will take at most a week. To determine the load during the building phase there is made assumptions about the amount and weight of the equipment. .In the building phase the maximum load will occur when there will be a truck with a maximum loading weight of 10 ton, a crane with a weight of 10 ton, building materials with a total weight of 5 ton and several people on a minimum area of 40 square meters. This means that the normative load will be a uniform load of 250 kN, this gives 6,25 kN/m².

6.4 Stability

Macro stability of the soil structure is defined as the resistance against shearing of large sections of the soil structures along curved slip planes. When the slopes are failing by macro instability, slaps of ground will slide along a curved plane, see Figure 6-3.

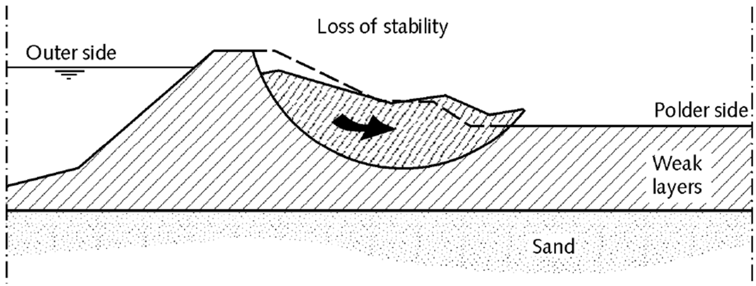


Figure 6-3 Sliding of the slope[14]

The safety against sliding is given by the relation between load and strength. The calculations for safety against sliding are good approximations, but use some simplifications. Because of this reason and the fact that ground layers are not uniform and can differs from place to place, a minimal safety factor is needed. The allowed minimal safety factors depends on the dike grade, the minimal safety factors for the different dike gradings are given in the Vietnamese guidelines, see Table 6-2. For the dike on Cat Hai Island the following minimal safety factor is needed:

$F = 1.15$

---

[14] (Tonneijck & Weijers, 2009)

Dike Grade	I	II	III	IV	V
Base load combination	1,30	1,25	1,20	1,15	1,10

Table 6-2 Minimal safety factors against sliding[15]

For the calculation of the safety against sliding, the program MStab is used. The program can use different methods for calculating the stability, the calculations of the used method (the method of Bishop) is explained in the next paragraph.

6.4.1 Calculations

For the analyses of the stability of slopes several methods have been developed. Most of these methods assume a circular slip surface. Using a number of simplifications a safety factor (F) is determined. The safety factor is the ratio between strength and load. The circle giving the smallest F, is considered to be critical.

For the calculations with MStab, the method of Bishop is used. Bishop's method is more consistent than the method of Fellenius. Various more sophisticated methods are developed, but the results often differ only slightly from those obtained with Bishop's method. Because of those two reasons the stability calculations are made with the method of Bishop.

The method of Bishop assumes that the soil fails along a circular slip surface, see Figure 6-4. The soil above the slip surface is subdivided into a number of slices, bounded by vertical interfaces. At the slip surface the shear stress is τ, which is assumed to be a factor F smaller than the maximum possible shear stress:

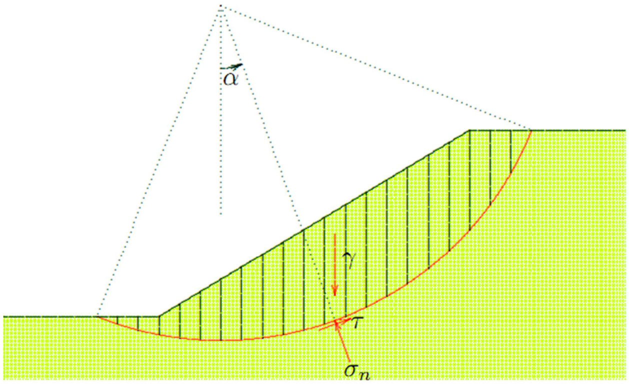


Figure 6-4: Circular slip surface

$$\tau = \frac{1}{F}\left(c + \sigma'_n * tan(\phi)\right)$$

$c = cohesion$
$\sigma'_n = normal\ effectif\ stress$
$\phi = angle\ of\ internal\ friction$

The equilibrium equation for a circular slip surface is the equation of equilibrium of moments with respect to the center of the slip circle. This equation is given by:

$$\sum \gamma hbR * sin(\alpha) = \sum \frac{\tau bR}{cos(\alpha)}$$

$h = height\ of\ the\ slice$ $b = width\ if\ the\ slice$
$\gamma = volemetric\ weight\ of\ the\ soil\ in\ the\ slice$ $R = radius\ of\ the\ circel$

When all the slices have the same width and the two equations are combined, it results in the following equation:

$$F = \frac{\sum \frac{c + \sigma'_n * tan(\phi)}{cos(\alpha)}}{\sum \gamma h * sin(\alpha)}$$

15 (CWRCT, 2010 march)

This is the basic method for calculating the safety against sliding along circular planes. The differences between the methods depend on the calculations of the normal effective stress.

Bishops method of calculating the normal effective stress is illustrated in Figure 6-5. Vertical equilibrium of a single slice is given by:

$$\gamma h = \sigma'_n + p + \tau \frac{sin(\alpha)}{cos(\alpha)}$$

When substituting τ into this equation and then substituting this equation in the calculation of F, it results in the final equation of Bishop:

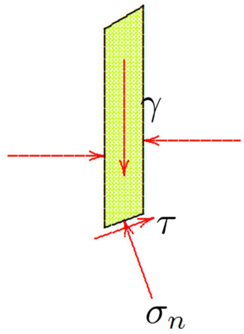


Figure 6-5: Bishops method for calculating the normal effective stress

$$F = \frac{\sum \frac{c + (\gamma h - p)tan(\phi)}{cos(\alpha) * \left(1 + \frac{tan(\alpha) * tan(\phi)}{F}\right)}}{\sum \gamma h * sin(\alpha)}$$

Because the factor F also appears in the right hand side of the equation, the safety factor F must be determined iteratively. The calculation has to been done for a large number of possible sliding circles. The calculations are done by the program MStab.

6.4.2 Results and conclusions

The stability of the geometry of the dike is checked with the program MStab. The critical slip circles of the geometries are given in Figure 6-6 and Figure 6-7. The safety values of those critical slip circles are:

- Geometry with block revetment: F = 2.03
- Geometry with rip-rap revetment: F = 2.31

The minimal needed safety against sliding is F = 1,15. The results of the calculations gives that sliding in the geometry will not occur, so the geometries are save. Complete results can been found in appendix 16.

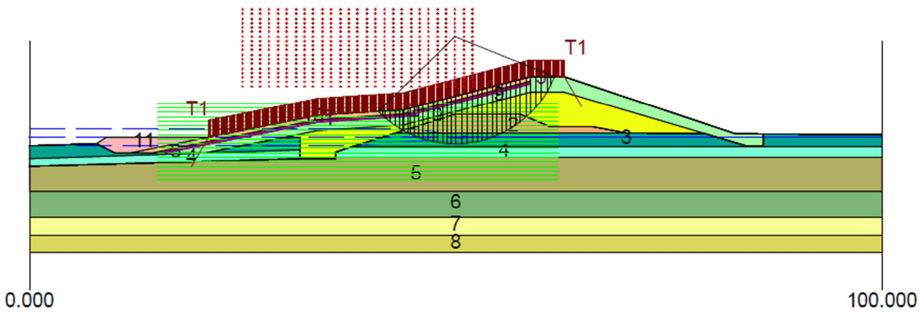


Figure 6-6: Critical slip circle for the geometry with block revetment

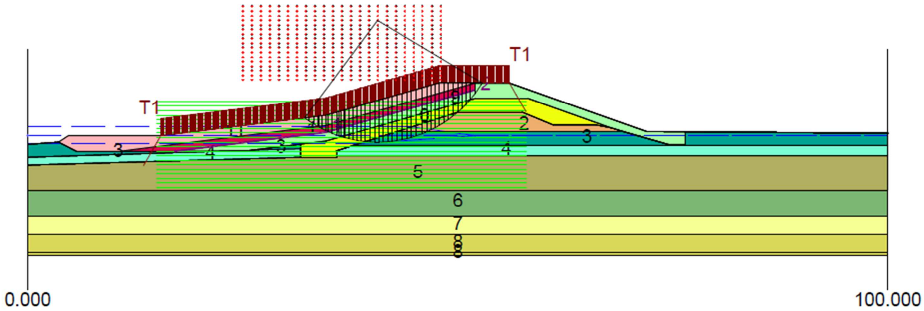


Figure 6-7: Critical slip circle for the geometry with rip-rap revetment

6.5 Settlement

In the design phase is it necessary to know how much the soil of the dike will settle. Settlement can be a failure mechanism and happens very slowly. If the crest settles too much, there is a change of overflow or the overtopping during storms becomes too large, which can result in failure of the inner slope. The consolidation process is when the soil decreases in volume, for example by its own weight or an external load. When the soil is saturated with water, the water will be squeezed out of the soil and then the soil particles will pack together more tightly. Calculations of the settlement will be made during the design phase, to be sure that the dike has the correct height when it is constructed. So the construction height must be higher than the design height, for a graphic overview see Figure 6-8.

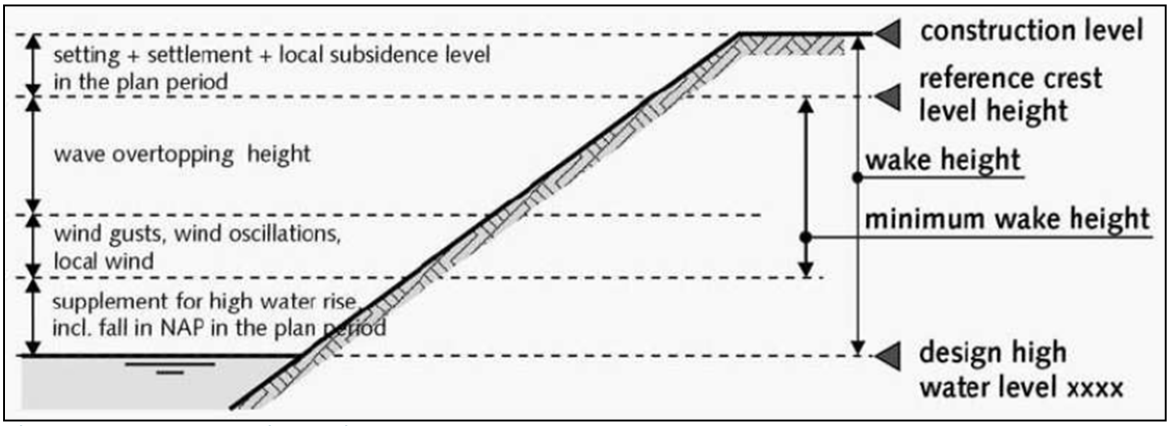


Figure 6-8: Construction height[16]

To predict the settlement of the soil there is information needed about the different ground layers. For rough predictions of the settlement in the Netherlands, it is usual to take for clay a decrease of height of 10 %. This value can drop to 5 % if there is a careful implementation and densification. Moreover, higher percentages could be assumed when unripe clay is used. One has to be careful when using unripe clay, because serious cracking can happen which can form a serious problem. For sand layers a value of 5 % settling is generally taken (for the increase in height). In Vietnam the standard is to take a settlement of 5% to 7% for the increase in height.

Settlement is a time-dependent process. The lifetime of the dike must be taken into account in the calculations. In the Netherlands, the lifetime of the dike is 50 years. The lifetime will be the same as in the Netherlands, at least 50 years. The total settlement consists out of primary settlements and secondary settlements over a period of 50 years. The primary settlement is the one that occurs during the hydrodynamic period and the secondary settlement over the period of 50 years.

[16] (Tonneijck & Weijers, 2009)

The new dike will be built on a location where already an existing dike is located. To determine the settlements of our new dike, information is needed about the settlements in the past. For most of the input standard values are taken for the different types of ground. With the information about the fact a new dike will be built on an old one, the settlement will be a lot less then when a completely new dike will be built.

MSettle is a program to make predictions of the soil settlements by external loading, typically for embankment construction or soil raise. MSettle produces results related to settlements and stresses per vertical[17]. In our design, there is decided to use the formula of Koppejan and the law of Darcy for the calculations of the settlement. The calculation of the used method is explained in the next paragraph.

6.5.1 Calculations

In the Netherlands the formula of Koppejan is often used to calculate the settlements. This equation gives the total settlement as a result of the primary settlement (after the completion of the consolidation) and the secondary settlement. The secondary settlement is the result of creep of the soil. The formula below is a simplified version of the Koppejan formula. Because the effects of the over consolidation is not used in this formula. Over consolidation is when stress is removed from a consolidated soil. The soil will expand but not to the original volume, because it is lost in the consolidation process. When this happens a couple of times, the soil will consolidate along a recompression curve over consolidation will be different than with only once a load. This means that the real grain tension will be higher than in the calculation on the basis of the volume weights and the depth[18].

$$\varepsilon = \left(\frac{U}{C_p} + \frac{1}{C_s} log(t)\right) ln\left(\frac{\sigma}{\sigma_1}\right)$$

ε = Settlement of the layer
U = consolidation grade
C_p = Koppejan: primary compression coefficient below pre-consolidation ($\sigma < \sigma_{grens}$)
C_s = Koppejan: secondary creep compression coefficient below pre-consolidation ($\sigma < \sigma_{grens}$)
σ = Stress after distributing the load
σ_1 = Stress before distributing the load
t = Time in days
C'_p = Koppejan: primary compression coefficient above pre-consolidation ($\sigma > \sigma_{grens}$)
C'_s = Koppejan: secondary creep compression coefficient above pre-consolidation ($\sigma > \sigma_{grens}$)

The law of Darcy is a description of groundwater flow. Henry Darcy explains with his experiments that the flow of water through a soil body saturated with water is proportional with the relative difference in water height. This is well known as the law of Darcy that is presented below. This law is valid for laminar flow. If the flow velocity will become too big, the flow will be turbulent and the formula is no longer applicable. Nevertheless, in the ground the flow velocity is so slowly that the groundwater flow is always laminar.

Figure 6-9 Color bar materials

[17] (Deltares, 2006)
[18] (Veruijt & Baars, 2001 & 2010)

$$q = -K * i$$

q = Specific discharge
K = Permeability coefficient
i = Gradient (dh/dl)

6.5.2 Results and conclusions

In the program MSettle the old and new dikes are imported. The problem of the program was in this situation that parts of the old dikes will be removed and new soil will be laid to form the new dike. The program however, makes no different between old and new layers. Therefore, for each dike the program has to run two times. Once with the load in the building phase on the new dike, see Figure 6-10 and Figure 6-12. And the second time with the load of the extra soil on the old dike, see Figure 6-11 and Figure 6-13. The legend of the figures is presented here on the right. The total settlement of the new dike exists of both settlements multiply by each other.

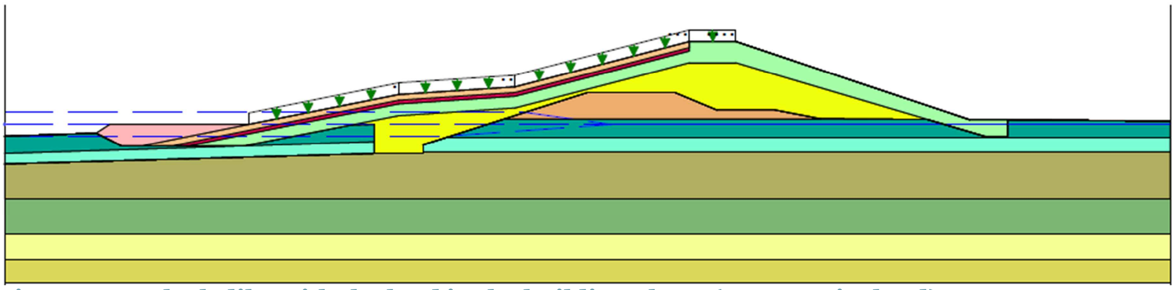


Figure 6-10 Block dike with the load in the building phase (at Cat Hai Island)

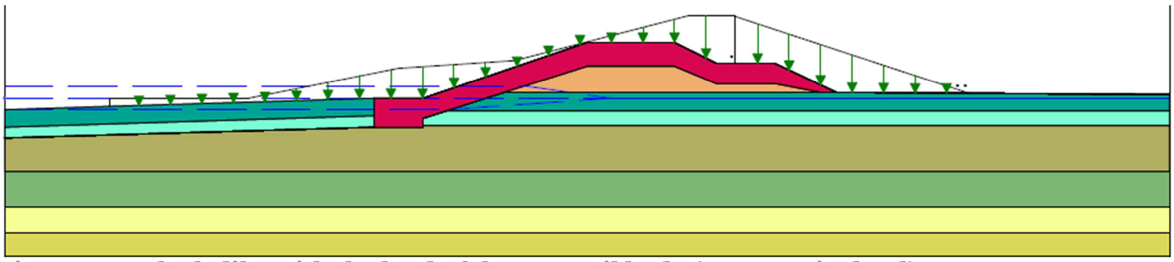


Figure 6-11 Block dike with the load of the new soil body (at Cat Hai Island)

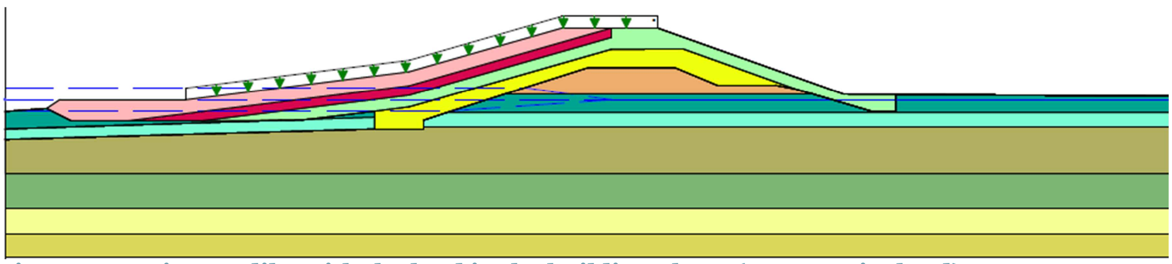


Figure 6-12 Rip-rap dike with the load in the building phase (at Cat Hai Island)

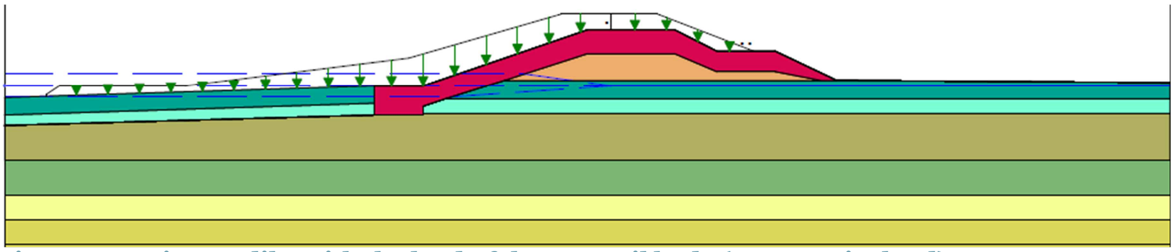


Figure 6-13 Rip-rap dike with the load of the new soil body (at Cat Hai Island)

The maximal settlement of the geometry of the dike is 20 centimeters. The critical settlement of the geometries is given in Figure 6-14 and Figure 6-15.

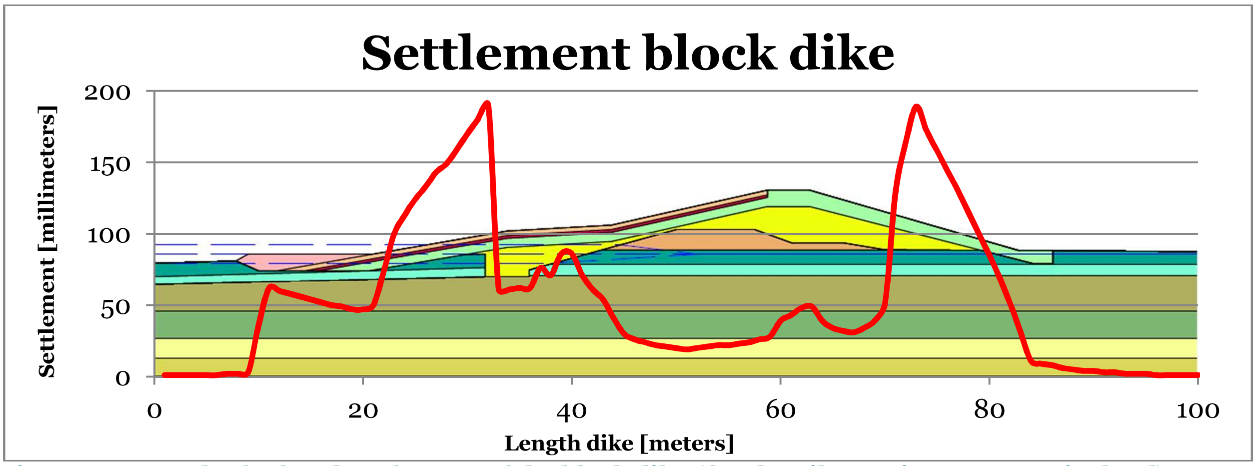


Figure 6-14 Total calculated settlement of the block dike (for the pilot project at Cat Hai Island)

The new design of the block dike has a total width in the cross-section of 62 meter, whereas the old dike length is 39 meter. At the places where the new dike is longer than the old one, the soil under the dike will settle more than were the new dike fit on the old one (see Figure 6-14). In additions to that, at the places were relatively more new sand and clay are added a bigger settlement will occur than the places were less new sand is placed. In reality the steep peaks in the settlement will not occur. Instead of these peaks a smooth transition between the different amounts of settlement will occur.

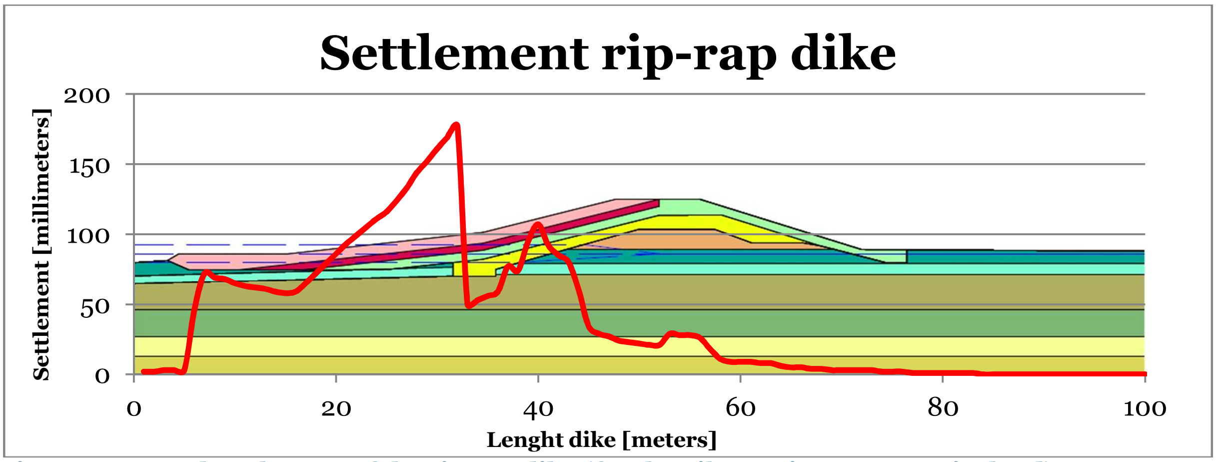


Figure 6-15 Total settlement of the rip-rap dike (for the pilot project at Cat Hai Island)

In Figure 6-15 can be seen that the settlement on the right side is small. This is realistic because the differences between the soil buildup of the old and new dike at this place is very small, as can be seen in Figure 6-13. A lot of settlement will occur at the left side because here the new dike will be a lot longer than the old one and the dumped rip-rap is relatively heavy.

In the building phase it is important to consider this extra settlement caused by the un-even distribution of the new soil over the old soil. The total settlement will not be more than 7 cm. Because of this the crest must be designed 7 cm higher. This amount of settlement can neglected, because within the design of the crest height an extra safety height of 30 cm is taken into account. Complete results can been found in appendix 16.1 & 16.2.

7 Revetment & Geotextile

One of the key points in designing a dike is the revetment type. This layer needs to withstand the wave action during storm conditions. Stability and permeability are essential factors that have to be considered when designing revetments.

The revetment layers of a dike consist of an armor layer, a filter layer and the base layer, which is also a part of the dike body. On Cat Hai Island, the base layer will be a clay layer. The filter layer can consist of a granular layer, a geotextile or a combination. For this pilot project, a geotextile fabric is chosen. The reason for this is that a good granular layer consists of several stone layers which for Vietnam standards is rather difficult to obtain. It is not that natural rocks in Vietnam are not available. In the contrary, Vietnam is rich in natural rocks. However, they are poor in processing the rocks for construction purposes. Therefore constructing a good granular layer will be costly business and therefore not preferred.

Nevertheless, Vietnam does have some grading of stones. To get a better overview of the stone grading of Vietnam, Table 7-1 gives a comparison between the Vietnam stone classes and the Dutch standard stones grading.

Grading NL	Dn50 of Netherlands [m]		Dn50 of Vietnam [m]		Grading VN
30/60 mm	0,04	0,06			
40/100 mm	0,05	0,08	0,05	0,10	***8***
50/150 mm	0,08	0,11			
80/200 mm	0,11	0,15	0,10	0,20	***7***
10 - 60 kg	0,22	0,26	0,20	0,30	***6***
60 - 300 kg	0,39	0,44	0,40	0,50	***4***
300 - 1000 kg	0,61	0,66	0,60	0,70	***2***
1000 - 3000 kg	0,88	0,95	0,80	1,00	***1***
3000 - 6000 kg	1,19	1,25			
6000 - 10000 kg	1,45	1,51			

Table 7-1 Dutch versus Vietnamese stone grading

7.1 Geotextile on the Base Layer

To choose a geotextile, first the base layer needs to be known, because without a good knowledge of the subsoil, it is impossible to choose the right kind of geotextile. The geotextile will be direct placed on the subsoil. The washout of the subsoil depends on the openings of the geotextile fabric and on the characteristic grain size of the subsoil. The subsoil of the base layer will be medium to heavy loam. This is a mix of sand, silt and clay, the smallest particle in this soil is that of clay. This soil type has a characteristic particle size of $d_{15} = 6\ \mu m$. The largest soil type of the subsoil is fine sand. The characteristic particle size of this soil type is $d_{90} = 63\ \mu m$. The rule is that the smallest particles may not pass the largest opening in the geotextile. However, some loss of fine material is acceptable, because the washout material is not detrimental to the function of the filter layer. A small layer under the geotextile can act as part of the total filter layer.

For stability of the geotextile, a rule of thumb for stationary flow is that the opening size should be smaller than two times the characteristic soil size of the largest particle:

$$O_{90} < 2 * d_{90B}$$

This means that $O_{90} < 126\,\mu m$. The possible geotextiles, which are used in Vietnam, are the Non-Woven TS-series, which is a product of *Ten Cate*: Polyfelt.

Properties [Standard]		Unit	TS 10 (4.01)	TS 20 (4.01)	TS 30 (4.01)	TS 40	TS 50	TS 60	TS 65	TS 70	TS 80
Mechanical Properties											
Tensile strength	MD	kN/m	7.5	9.5	11.5	13.5	15.0	20.0	21.5	24.0	28.0
[EN ISO 10319]	CD	kN/m	7.5	9.5	11.5	13.5	15.0	20.0	21.5	24.0	28.0
Elongation at max.	MD	%	90	90	90	100	100	100	100	100	100
load [EN ISO 10319]	CD	%	75	75	75	40	40	40	40	40	40
Static puncture resistance (CBR-Test) [EN ISO 12236]		N	1200	1500	1750	2100	2350	2900	3300	3850	4250
Cone drop test (hole-Ø) [EN ISO 13433]		mm	28	24	20	25	22	19	17	15	14
Hydraulic Properties											
Permeability vertical [EN ISO 11058] (Δh = 50 mm)		l/m²s (mm/s)	130	115	100	100	90	80	70	60	55
Opening size O_{90} [EN ISO 12956]		µm	105	105	100	100	100	95	95	90	90

Figure 7-1 Non-Woven Geotextile, TS-series

From the stability calculation, it shows that every geotextile is suitable to place directly on the sandy/clay loam subsoil. However, also the permeability factor needs to be checked. The reason to check on this is to prevent pressure build-up underneath the geotextile. A simple rule of thumb is that the permeability of the geotextile, k_f should be more than 10 times larger than that of the subsoil, k_s.

$$k_f > 10 * k_s$$

The permeability of sandy/clay loam has a k_s factor of around 10^{-2} mm/s (this is on the safe side of sandy/clay loam, meaning more permeable base layer). This means looking at the TS-series geotextile that every geotextile is at least 10 times as permeable as the base layer (see Figure 7-1).

The last thing that has influence on the choice of the geotextile is the tensile strength of the fabric and the cost per m². Both fall out of the scope of this report because the geotextile lies on a dike for which the tensile strength of the fabric is not a key factor. The second part is the cost factor that can still influence the choice of geotextile. From experience of the Vietnam engineers, they usually use TS-40 or TS-50 for marine works. TS-50 is chosen because of the higher tensile strength.

7.2 Design top layer

To calculate the revetment height several formulas are available. The most common ones to calculate the height are the Holcim[19] formula, which is used to calculate the size of their Basalton® blocks, and the Pilarczyk[20] formula to calculate column blocks.

[19] (Holcim Betonproducten B.V., 2009)
[20] (Pilarczyk, 1998)

For rip-rap stones two formulas are used, these are the “van der Meer” formula and again the Pilarczyk formula. First, the two column formulas will be explored. After that, the two rip-rap formulas will be further discussed to see which one is the most suitable.

7.2.1 Revetment height Basalton® blocks

The formula given by the Holcim brochure for the Basalton® blocks is actually a general formula for column blocks. This is because the special properties of Basalton®, such as friction between blocks, are not taken into account within this formula. It has to be said that the outcome of the height of a block could vary if friction and clamping between the blocks are taken into account. However, for this preliminary design these two factors are not taken into account.

The fixed parameters for these calculations are given in Table 7-2, some of them are explained in the boundary conditions.

General parameters		
Significant Wave Height	1,7	m
Peak Period	11,2	sec.
Gravitation	9,79	m/s^2
Density Concreet	2300	kg/m^3
Density Rip-Rap	2600	kg/m^3
Density Sea Water	1030	kg/m^3

Hudson Parameter		
Kd factor	3,5	-

vd Meer Parameters		
Average Period (T 1/3)	10,6	sec.
Storm Duration	4	hours
Permeability	0,1	-
Damage Factor	3	-

Pilarczy		
Psi - Rip-rap	1	-
Psi - Column Block	2	-
exponent - Rip-rap	0,5	-
exponent - Column Block	2/3	-

Table 7-2 The parameters for the calculations

Seen from this set of pre-determined parameters one can notice that the chosen peak period is extremely long. To see if it would make a difference in outcome if the peak period could be determined better, a re-calculation is done with a shorter peak period to see how sensitive this particular parameter is. First calculations are done with the parameters as determined from the boundary conditions and as shown in table 7-2. At the end of chapter 7.2 the same calculation will be done however this time with the ½ of the pre-determined peak period. To decide if it is wise to better research the extreme length of the peak period.

The first formula is given by Holcim, which is as follow:

$$\frac{H_s}{\Delta D} = 6 * \xi_p^{-\frac{2}{3}}$$

Rewriting this formula gives:

$$D = \frac{\left(H_s * T_p * \tan(\alpha) * \sqrt{\frac{g}{2\pi}}\right)^{\frac{2}{3}}}{6 * \Delta}$$

Within the formula, the Iribarren number is written in full, besides that the peak period is used and for the wavelength, the L_o equation. The α is based on the slope of the revetment and the Δ is calculated as follow:

$$\Delta = \left(\frac{\rho_s - \rho_w}{\rho_w}\right)$$

The second way to calculate the column height is by using the Pilarczyk formula this is as follow:

$$\frac{H_s}{\Delta D} = \psi * \phi \frac{\cos(\alpha)}{\xi_p^b}$$

Rewriting this formula will give the following:

$$D = \frac{H_s * \xi_p^b}{\Delta * \psi * \phi * \cos(\alpha)}$$

With ψ as a system-defined stability-upgrading factor and the Iribarren exponent b to represent whether a block is smooth or not. Both of these parameters are given constants, which are chosen by experiments and then curve fitting. The ϕ parameter is a stability factor, which is compost partly from the "van der Meer" formula: $\phi = 6{,}2 * P^{0{,}18}\left(\frac{S^2}{N}\right)^{0{,}1}$ Within the formula the P is the permeability, S is a subjective chosen damage factor and N is the number of waves hitting the revetment. The N is calculated as follow:

$$N = \frac{T_{storm}}{T_{average\ wave}}$$

With this calculation, the needed period is the average period. Unfortunately, due to the lack of good background data only the peak period is known. To get as close to the average period the $T_{1/3}$ is used. The following equation can calculate the $T_{1/3}$: $T_{\frac{1}{3}} \approx 0{,}95 T_p$. It is known that this is still not the average period. Nevertheless, seen that the N is within the ϕ and it is to the power of 0.1 the significant change it will give to the outcome of the column height is negligible.

Calculating the column height with the given formulas gives the results shown in Figure 7-2.

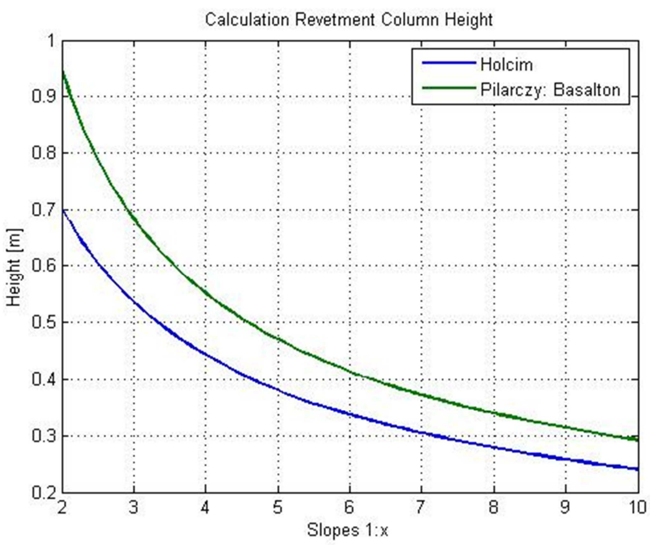


Figure 7-2 Colum height vs the slope for conditions formulated in Table 7-2

Looking at Figure 7-2, one can see that the result of the Holcim formula is smaller than that of the formula of Pilarczyk. The reason for this is that Pilarczyk takes much more physical aspects of the storm conditions into account than the Holcim formula. They both do consider the significant wave height, the slope of the revetment and the peak period via the Iribarren number. However, Pilarczyk also takes into account the permeability of the revetment, the number of waves during storm conditions and it provides, although rather subjective, a damage factor. However, it is still a number that can describe what damage is, to an extent that can help the decision makers which level of maintenance is necessary after each storm.

The drawback of the Pilarczyk formula is that it is an overestimation because it does not take into account the clamping/friction between the blocks. This argument also holds for the Holcim formula. Besides that, the Pilarczyk formula is actually invented for breakwaters and not as a formula for dike revetments. Therefore, the problem rises that Pilarczyk is only suitable for situations when waves are plunging. However, this design is based on storm conditions where by the wave action on the dike is surging and not plunging.

Because of the above-mentioned reasons, the Holcim formula gives a good first estimation. For this preliminary design phase, the safe, upper side of the Holcim formula is chosen. This means that the necessary block height for a slope of 1 : 3.5 will be a standard height of 0,50 m and for a slope of 1 : 5 it will be the standard height of 0,40 m.

7.2.2 Revetment height rip-rap stones

To calculate the stone size of rip-rap it is possible to use two kinds of formulas. The most common used formula is the Hudson formula. This formula is famous because of its simplicity but not suitable to use for these types of calculations. It leaves out a lot of the physical background of storm conditions and this formula may not be used in situations with an impermeable core like a dike revetment. Besides that it is not applicable for waves with a low steepness. Therefore, this formula will not be used as a possible design formula.

The first formula used is again the Pilarczyk formula. This is the same one as for the column blocks; however, the ψ and the exponent b of the Iribarren number in the formula differs for rip-rap.

The second formula is the "van der Meer[21]" formula, which also considers several aspects of the storm conditions. Just like the Pilarczyk formula, it also makes use of a damage number, the permeability of the revetment and wave numbers. Besides that, it also makes a distinction between surging and plunging waves.

For plunging breakers, the formula is as follow:

$$\frac{H_s}{\Delta D_{n50}} = 6{,}2 * P^{0{,}18} * \left(\frac{S}{\sqrt{N}}\right)^{0{,}2} * \xi^{-0{,}5}$$

For surging breakers, the formula looks quite different:

$$\frac{H_s}{\Delta D_{n50}} = 1.0 * P^{-0{,}13} * \left(\frac{S}{\sqrt{N}}\right)^{0{,}2} * \xi^{P} * \sqrt{\cot(\alpha)}$$

[21] (Schiereck G. J., 2004)

The transition between the two equations can be found by equating them, this gives a transition point that shows if a wave is surging or plunging, the transition formula is as follow:

$$\xi_{transition} = \left(6{,}2 * P^{0{,}31} * \sqrt{\tan(\alpha)}\right)^{\left(\frac{1}{P+0{,}5}\right)}$$

Giving that if $> \xi_{transition}$, the equation for surging breakers will be used, when $\xi < \xi_{transition}$ the plunging breakers of the “van der Meer” formula is used. Just like the column blocks, the different formulas can be filled in to calculate the nominal diameters of rip-rap. The results can been found in

Figure 7-3.

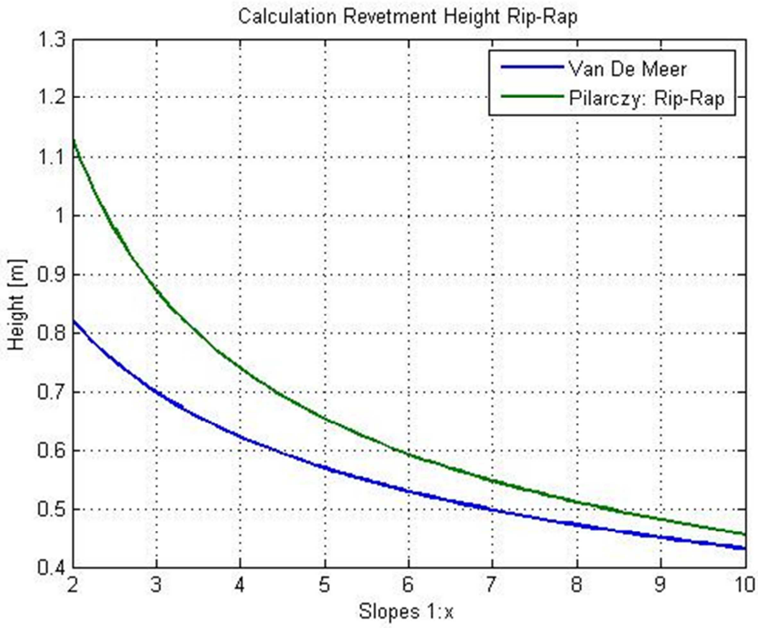


Figure 7-3 Rip-rap height vs the slope for conditions formulated in Table 7-2

The green line in the figure is based on the Pilarczyk formula given for rip-rap dikes. It is an upper limit because the Pilarczyk formula does not make a distinction between plunging and surging waves. The reason for this is also stated in the above paragraph.

The last formula that is used is the “van der Meer” formula; this one does make a distinction between plunging and surging waves. Because the dike is designed to cope with storm surge conditions, the surging form of the “van der Meer” formula is therefore always chosen to calculate the height of the revetment. Besides this distinction, the “van der Meer” formula also takes many physical characteristics of the storm surge within its formula. Based on the above-mentioned reasons, this formula will be chosen for the design of the rip-rap revetment. This means that a slope of 1 : 3.5 needs a D_{n50} of 0,67 m, and the slope of 1 : 8 needs a D_{n50} of 0,47 m

Now that the height of the rip-rap armor layer is known, a bedding layer needs to be designed. This is because placing rip-rap stones directly on the geotextile could easily damage the delicate fabric. This statement also holds for column blocks however, this layer must be chosen carefully so that the stability and permeability will not be jeopardized.

The rules, which have to be satisfied to know if the chosen bedding layer is stable and permeable, are:

- For stability: $\frac{d_{15f}}{d_{85b}} < 5$
- Furthermore for permeability: $\frac{d_{15f}}{d_{15b}} > 5$

As said before, the grading of Vietnam is not as good as the grading of the Netherlands. Therefore, the design will first be done with the use of the Dutch grading. Using Table 7-3 the Dutch grading can be translated to the Vietnamese grading. The reason for using the Dutch grading first is to be able to apply the stability and permeability rule, it is necessary to know the D_{n15} and D_{n85}. Due to the poor grading of Vietnam this vital data is missing.

Grading NL	W50 [kg]		Dn50 of Netherlands [m]		Dn50 of Vietnam [m]		Grading VN
30/60 mm	0,15	0,45	0,04	0,06			
40/100 mm	0,4	1,2	0,05	0,08	0,05	0,10	*8*
50/150 mm	1,2	3,1	0,08	0,11			
80/200 mm	3,1	9,3	0,11	0,15	0,10	0,20	*7*
10 - 60 kg	26	46	0,22	0,26	0,20	0,30	*6*
60 - 300 kg	150	220	0,39	0,44	0,40	0,50	*4*
300 - 1000 kg	595	760	0,61	0,66	0,60	0,70	*2*
1000 - 3000 kg	1800	2200	0,88	0,95	0,80	1,00	*1*
3000 - 6000 kg	4400	5050	1,19	1,25			
6000 - 10000 kg	7850	8900	1,45	1,51			

Table 7-3 The Dutch stone grading versus the Vietnamese

To determine which kind of stone class is needed the weight of the calculated stone need to be determined: $0{,}67^3\ m^3 \ * 2600 \frac{\text{kg}}{\text{m3}} = 751{,}9$ kg. This means that the Dutch stone grading of 300 – 1000 kg is needed.

The D_{n15} and D_{n85} are calculated using the following calculations, first a spread calculation based on the weight can be used:

$$Spread = \frac{W_{85}}{W_{15}}$$

These spread gradations are standard given values for the Dutch grading. From that point on the D_{n15} and the D_{n85} can be calculated using the following formulas: $D_{n15} = \frac{D_{n50}}{\sqrt{Spread}}$ and $D_{n85} = D_{n50} * \sqrt{Spread}$. The results of these to calculations can been found in Table 7-4.

Average W85/W15 [-]	Dn15 [m]	Dn85 [m]
	0,03	0,06
	0,04	0,10
	0,05	0,15
	0,08	0,20
5,5	0,10	0,56
4,4	0,20	0,87
3,1	0,37	1,11
2,9	0,54	1,56
1,9	0,88	1,68
1,6	1,17	1,87

Table 7-4 Calculated D_{n15} and D_{n85}

Now that the D_{n15} and D_{n85} are known for all the Dutch grading, a bedding layer can be composed. Based on the stability and permeability equations the bedding layer between the armor layer and filter layer will be 40/100 mm, this has a D_{n50} between 0,05 – 0,08 m. The results are shown in the table below. The factors have to be smaller than 5 for the stability and larger than 5 for the permeability. The chosen filter layer is made bold in Table 7-5.

Grading NL	W50 [kg]	Dn50 of Netherlands [m]	Dn50 of Vietnam [m]	Grading VN	Dn15 [m]	Dn85 [m]	Stab < 5	Perm > 5
30/60 mm	0,15 - 0,45	0,04 - 0,06			0,03	0,06	6,09	12,17
40/100 mm	**0,4 - 1,2**	**0,05 - 0,08**	**0,05 - 0,10**	**8**	**0,04**	**0,10**	**3,65**	**9,13**
50/150 mm	1,2 - 3,1	0,08 - 0,11			0,05	0,15	2,43	7,30
80/200 mm	3,1 - 9,3	0,11 - 0,15	0,10 - 0,20	7	0,08	0,20	1,83	4,56
10 - 60 kg	26 - 46	0,22 - 0,26	0,20 - 0,30	6	0,10	0,56	0,66	3,58
60 - 300 kg	150 - 220	0,39 - 0,44	0,40 - 0,50	4	0,20	0,87	0,42	1,86
300 - 1000 kg	**595 - 760**	**0,61 - 0,66**	**0,60 - 0,70**	**2**	0,37	1,11	0,33	1,00
1000 - 3000 kg	1800 - 2200	0,88 - 0,95	0,80 - 1,00	1	0,54	1,56	0,23	0,68
3000 - 6000 kg	4400 - 5050	1,19 - 1,25			0,88	1,68	0,22	0,41
6000 - 10000 kg	7850 - 8900	1,45 - 1,51			1,17	1,87	0,20	0,31

Table 7-5 Calculation of the bedding layer

This means that the Vietnamese stone grading class 2 and 8 are needed to build a stable rip-rap dike revetment on a slope of 1 : 3,5. For the slope of 1 : 8 the rip-rap stone revetment will also be from the stones of 300 – 1000 kg. The reason for that is that stones from 0,47 m give a needed weight of 270 kg, which also fall in the category 300 – 1000kg. Because the W_{50} limits of the category 60 – 300 kg lies between the 150 – 220 kg, therefore a higher stone class is needed to gain enough stability.

7.3 Conclusion

For the two test dikes geotextile TS-50 will be used. This geotextile will be placed directly on the base layer that consists of a clay layer. For both the column blocks and rip-rap the bedding layer consists, based on the Vietnamese stone class, of a stone class of number 8 which will have a nominal diameter between 0,05 and 0,08 m. To consider the height of the bedding or the rip-rap layer a rule of thumb is used. The rule of thumb is 2 to 3 times the D_{n50}. This means for the bedding layer $2 * 0{,}065 = 0.13\ m$. Due to constructability, this layer thickness will be 0.15 m. The rip-rap layer has a thickness of $2 * 0.67 = \ 1.34\ m$, this will be stated to a thickness of 1,35 m. The column blocks, which are Basalton® block within this design, consist out of two standard Basalton® heights. For the slope of 1 : 3.5 the necessary height is 0.50m for the slope of 1 : 5 the required height is 0.40 meters.

7.4 Sensitivity analysis of the peak period

In the previous chapter the calculation are being made based on a peak period which with common sense and a good trained engineer, should know that this is extremely large. To see if the peak period should get better researched the above calculations will be done again. However, this time 50% of the pre-calculated peak period will be taken. This means a peak period of 5,6 seconds. All the other parameters will be as given in table 7-2 these will be kept the same as the previous calculation, to only see the effect of the smaller peak period.

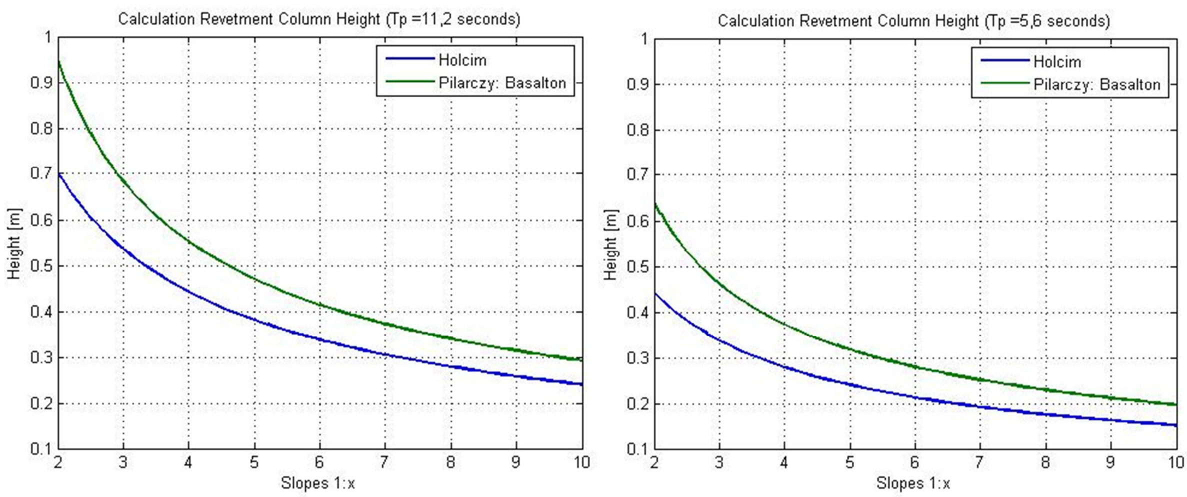


Figure 7-4 The column height vs slope for T_p=11,2 & T_p=5,6

Seen from Figure 7-4 a smaller peak period that does come in line with a value which one would expect the height of the column blocks are considerably smaller. For this case a slope of 1 : 3.5 gives with the smaller peak period a column height of 0,3 m. This is around 20 centimeters lower in height. For the slope of 1:5 it only needs a height of 25 centimeters which is 15 cm smaller that with a T_p of 11,2 seconds.

The results for the Rip-rap dike are as follow:

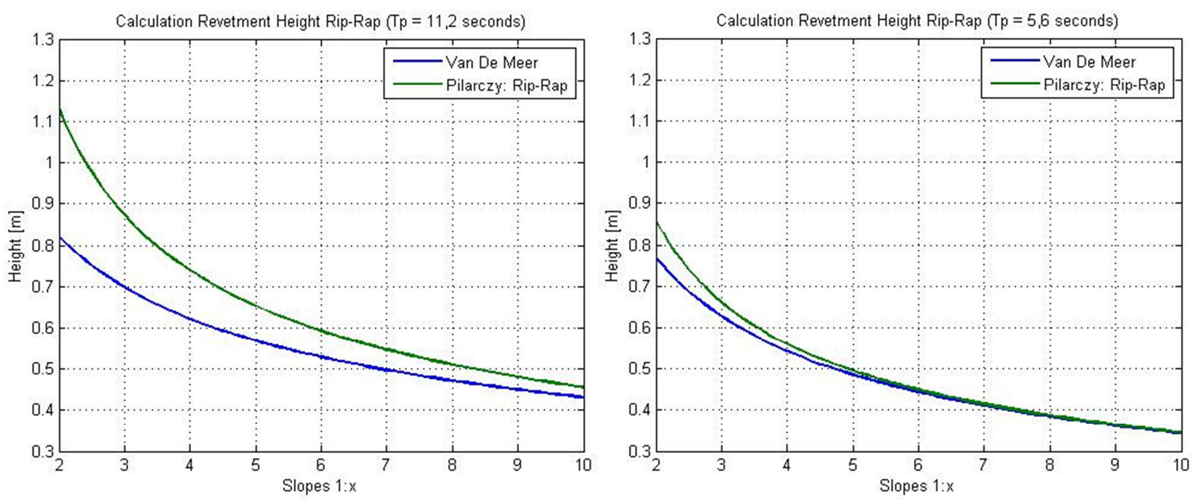


Figure 7-5 The stone size of rip-rap height vs slope for Tp=11,2 & Tp=5,6

From this figure the stone dimensions can also become smaller with a better researched peak period. The stone size for a slope of 1 : 3.5 the stone size can be reduced to 58 cm, this means a reduction of almost 10cm and for a slope of 1 : 8 with a T_p of 11,2 seconds is 0,47 m the stone size for T_p is equal to 5,6 gives of a round 0,39 cm. Meaning also a reduction of almost 10 cm.

The conclusion of this sensitivity analysis is that the peak period is a very vital parameter. In this case the peak period is far too high. Investing in researching this parameter is highly recommended. This will be cheaper in executing, because the needed revetment will be smaller. So investing in this particular parameter could earn its money back.

7.5 Placing the revetments

Rip-rap stones need to be dumped and not be placed by hand. The reason for this is that a rip-rap dike gains its internal strength due to the best possible natural position, which it will take when it is dumped. Basalton® needs to be placed. With Basalton® however, a problem can occur due to the difference of slope and therefore a difference between the required Basalton® heights. It is strongly advised that a column block revetment does not have any transitions, especially not around the area where the wave impact is the strongest. Therefore, a solution is to choose for only one kind of revetment height. In this project the minimal slope is 1 : 3.5 meaning that a minimal column height is needed of 0,50 m. Therefore, the whole dike could be covered with only these kinds of blocks. The advantage of this solution is as stated before that there are no transition points. In this project the weak point exists at the transition of the berm which has a slope of 1 : 15 and the slope of 1 : 3.5. The drawback of this solution is that the cost of the revetment would be a lot higher if one should only use column blocks of 0,50 m. This means an increase of concrete volume of 25%, meaning an increase in cost of 25%. A small preliminary calculation is given:

The cost of 1 m^3 Basalton® concrete is € 235,- (This value is based on a m^2 of the Dutch price of Basalton® with a height of 20 cm this cost € 42,00. The m^3 of Baslaton® is including the open spacing between the column blocks). This means that for a block with a height of 0,40m the cost of 1 m^2 of 0,40 m height block the cost are 85 €/m^2. For blocks of 0,50m the cost for 1m^2 is 106 €/m^2.

This means that for an area of around 8.000 m^2, which is the area of the berm and the lower slope, the costs are as follow:

- For 0,40m the cost are € 677.000,-
- For 0,50m the cost are € 846.000,-

The difference is the 25% which is € 169.200,- meaning 4,23 billion VND. This means that the cost, to extend the 0,50 m Basalton® columns to the full length of the dike will be quite expensive.

A second method is to place the Basalton® on the same height by placing the smaller Basalton® higher on the subsoil of the bedding layer then the longer Basalton® blocks. This is shown Figure 7-6:

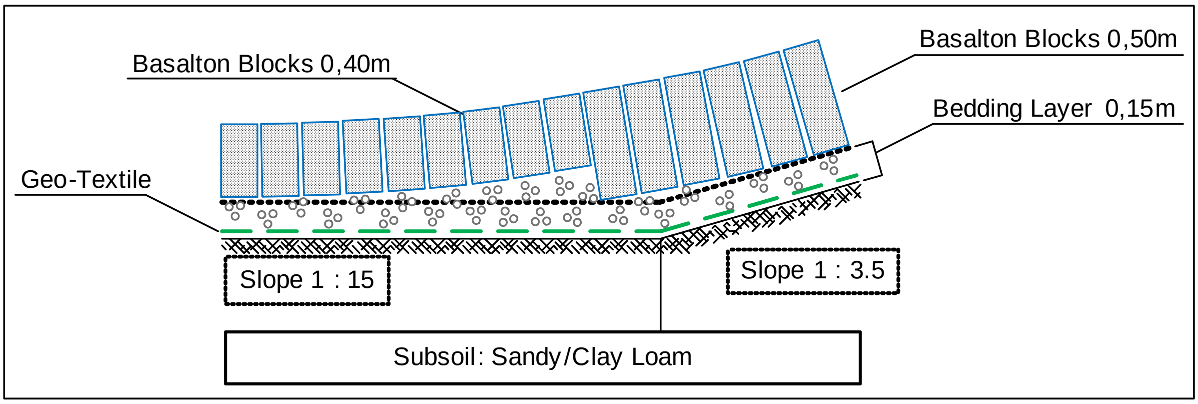


Figure 7-6 Transition between Basalton® blocks of 0,40 to 0,50 m

As seen in Figure 7-6, a thicker filter layer is needed at the transition point from the berm to the slope. However, in comparison to Basalton®, the material for the filter layer is a lot cheaper. Thus by using this method of construction a lot of expenditure can be saved. It is unknown of this design is possible to make for the construction workers on the field, because of the jump of 10 cm in the bedding layer. Nevertheless, if it is constructible by the workers on the field this design could be a good solution to overcome the height difference.

7.6 Tolerable damage

From the above paragraphs, one can imagine that with good grading and good quality blocks/rocks an excellent dike is quite easy to design. Unfortunately, in Vietnam this good processing of rocks and making good quality controlled blocks is not so obvious. As stated at the beginning of this chapter, Vietnam does have many natural stones quarries, but the processing of the stones for construction purposes is very poor. When asked if it is possible to gain rip-rap stones with a D_{n50} of 0.67m the answer was "no". If they want, they can gain these large size stones. However, the cost of gaining these stones is considered too high. The maximum stone sizes that they can easily gain access to are stones from around a D_{n50} of 0,40m. With that in mind, designing a dike with a rip-rap revetment becomes hard and it could eventually cost a lot more than when using column blocks.

There are two ways, which can be done to still use the stone size of 0,40m. The drawback of this solution is that the tolerable damage is excessively high and more maintenance after each storm is needed. In addition, the largest acceptable slope is chosen to be the design dike slope. When taking a very smooth slope of 1 : 8 meaning that with a height of 7m the width of the dike foot would be larger than 100 meters.

The first way to use stones with a D_{n50} of 0,40m is to bump up the damage factor, within the used "van der Meer" formula. The damage level for the optimal design has a factor 3. Given that, maintenance is necessary on chosen intervals and small maintenance will be needed after each big storm. The damage number can be varied between 2 to 10. With 10 being a damage level whereby the geotextile is completely exposed after a storm and high maintenance is necessary.

If one should double the damage factor from 3 to 6, the result is shown in Figure 7-7.

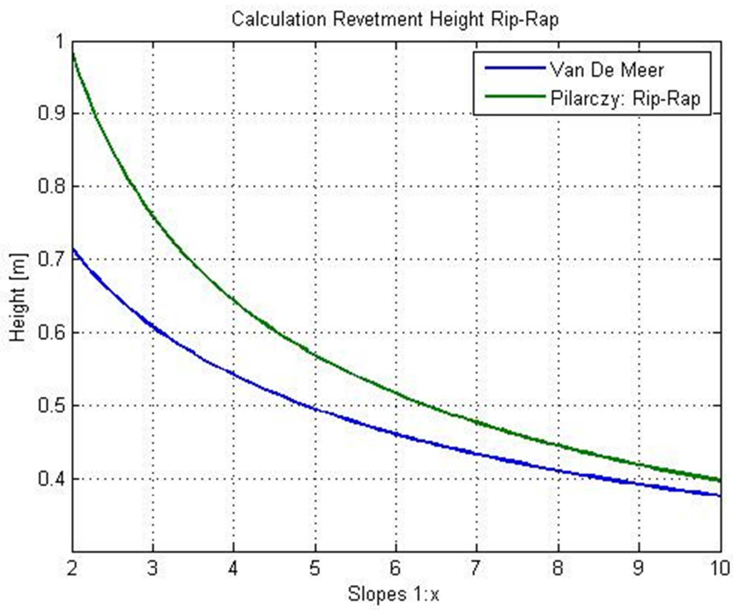


Figure 7-7 Rip-rap height with damage number 6 futher conditions formulated in Table 7-2

Looking at Figure 7-7 and at a slope of 1:8 the needed stone size is now 0,41 m. In the optimal situation whereby the damage factor was not doubled, the necessary stone height was 0.47 m.

Although in the previous chapter, the Pilarczyk formula was one of the formulas that was not chosen, this one is still going to be used here. The reason for this is that this formula has one extra parameter, which can be altered. The great thing about this is that the damage level which was set on 6 can be set back to level 3. The extra parameter is the ψ value, which is in fact also a kind of damage number. For rip-rap normally the ψ value is put to 1. However, if one is willing to tolerate more damage, which is not a wise decision, ψ can also have a value of 1,33 (Table 7-6). If one should use a ψ value of 1,33 the result is shown in Figure 7-8.

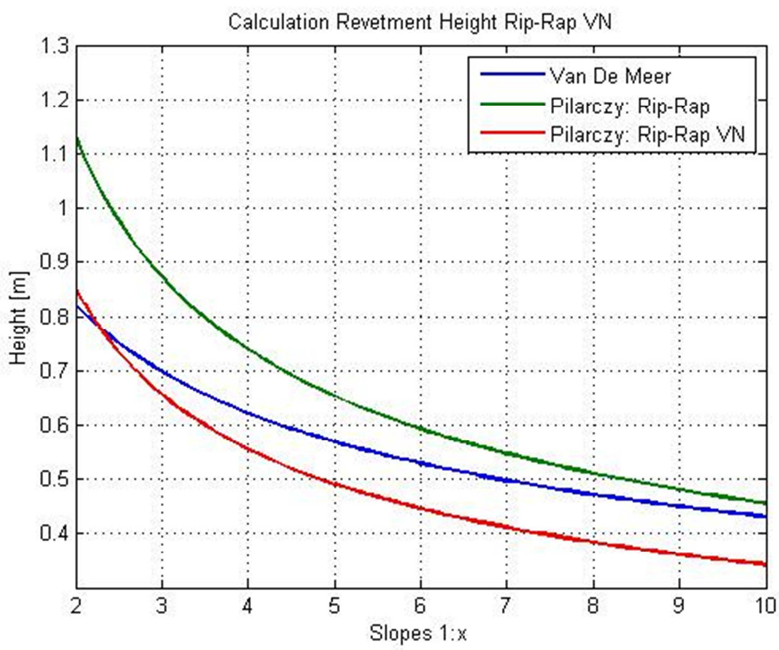


Figure 7-8 Pilarczyk formula with a ψ-value of 1.33 futher conditions formulated in Table 7-2

$\frac{H_s}{\Delta_m D} = \psi_u \cdot \Phi \frac{\cos\alpha}{\sqrt{\xi_p}} = \psi_u \cdot 2,25 \frac{\cos\alpha}{\sqrt{\xi_p}}$ (Criterion)		$\Phi_{(rock)} = 2,25$ $\cot\alpha \geq 2$ & $\xi_p < 3$ (Limits)	
System	ψ_u	**Description**	**Sublayer**
Ref.	*1,0*	*Rip-rap (2 layers)*	*Granular*
Rock	*1,33*	*Rip-rap (tolerable damage)*	*Granular*

Table 7-6 ψ-value based on tolerable damage

Figure 7-8 shows what the difference will be between the Pilarczy formula with a ψ of 1,33 (red line), the "van der Meer" formula (the blue line) and the normal Pilarczyk formule (the green line). It will be at a slope of 1 : 8 almost 10 cm lower than the used "van der Meer" formula. The required stone height is now 0,39 m instead of 0,47 m.

Concluding from this is that based on the known formulas and a grading of 0,40 m, it is possible to construct a rip-rap dike with a slope of 1 : 8. However, this way of designing a dike is not the best method. Taking the limits of the parameters will result in a relative risky design and in more maintenance.

8 Toe

In designing a dike, the toe construction is an important part. The revetment on the slope is supported by the toe, which is placed in front of the dike structure. The design of the toe depends mainly on the scouring process. There are two types of scouring, a structural scouring and scouring due to extreme conditions. When a wave extends down rush on the structure face to the toe and/or the wave breaks near the toe, the maximum scour force occurs.

Structural scouring in front of a toe is a very specific mechanism. It is a result from a gradient in the longshore sediment transport. This is not from extreme conditions it is due to the small frequencies of events, the majority of this structural erosion is due to moderate wave conditions, like the tidal influence or wind waves. Due to cross-shore re-distribution of sediment during high tide and the longshore sediment transport. Deepening in front of the sea wall, which is enlarged by wave attacks against the sea wall, can occur. This eventually can lead to damage or failure of the seawall structure. For that reason there is not an easy prediction of how large the scour will be, not even using scale testing or model results, because the results of different tests conflict with each other. The reason to provide this information here is because it is impossible to construct a revetment that can protect the coastline from structural erosion. It is possible to construct a toe that holds the scour hole sufficient far away from the structure so that the construction has no influence of the erosion. The toe is designed within this project to prevent storm waves from scouring and undercutting the dike.

8.1 Stability of the toe constructions

For the two test dikes, a toe needs to be constructed. The two test dikes consist of a Rip-Rap dike and a Basalton® dike. To design the toes of these two types of dikes some typical Dutch design rules of thumb are used together with the possibilities Vietnam has to offer. After the concept designs of the toes the new designs are being tested, using the available calculation methods based on breakwaters and one from the Vietnamese guidelines.

8.1.1 Design situation in the Netherlands

The significant wave height of the design wave is 1.7 meter at the toe. In the upcoming calculations this design wave is used. The maximum expected wave at the toe is assumed equal to the maximum scour depth at the natural bed, this means that the scour in this case will be 1.7 meter.

The concrete tubes currently used at the front of the toe are 2 to 2.5 meter high. To prevent an upcoming scour hole close to the tube, the soil must be kept in place in order to protect the stability of this tube. The toe protection is to prevent undermining of the dike; therefore, the scour must be kept far enough away from the dike. The extensions of the protection can be made in or above the natural surface of the seabed with a certain length. An option is to burying the extension of the toe. When this is not completely possible, the apron can be places on the existing bottom or partially buried under the natural seabed. However, the width of the extension should not be less than twice the local wave height[22]. The situation sketch of the concrete tube is described in paragraph 2.5 and can be seen in Figure 2-25. The apron is in current situation not buried and has a length of about 5 meters. The minimal

[22] (Pilarczyk, 1998)

length must be 3.4 meters (two times H_s). In the Netherlands the same kind of toe is used, which is shown at the left side of

Figure 8-1. In this situation, the length of the toe is 3 H_s - 4.5 H_s, this means for the situation at Cat Hai Island a toe length of 5.1 - 7.65 meters.

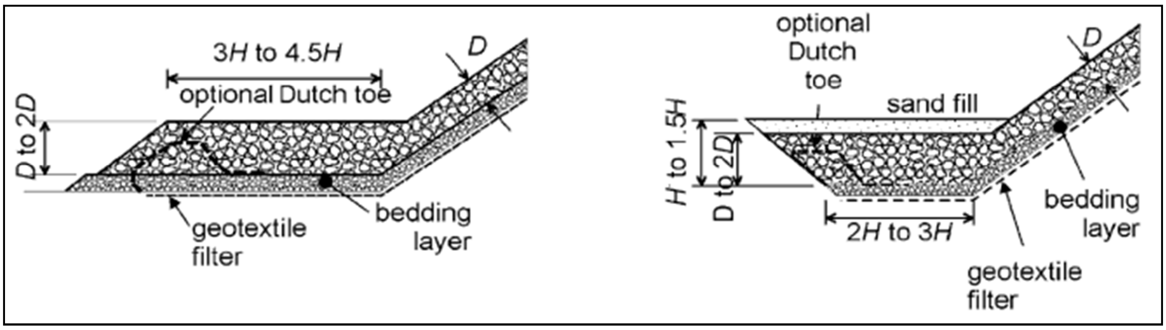


Figure 8-1 Dutch toes

Experience of the Vietnamese engineers learns that the current toes, does not work well. The current toes only just meet the Dutch standards. Besides that, conditions in Vietnam are a lot heavier than in the Netherlands. Looking from that point of view the new toes must be stronger than the current toes. For this design, the assumption is made that there is a strong near shore current and if the water is in calm conditions, the foreshore is stable. (See Figure 8-2)

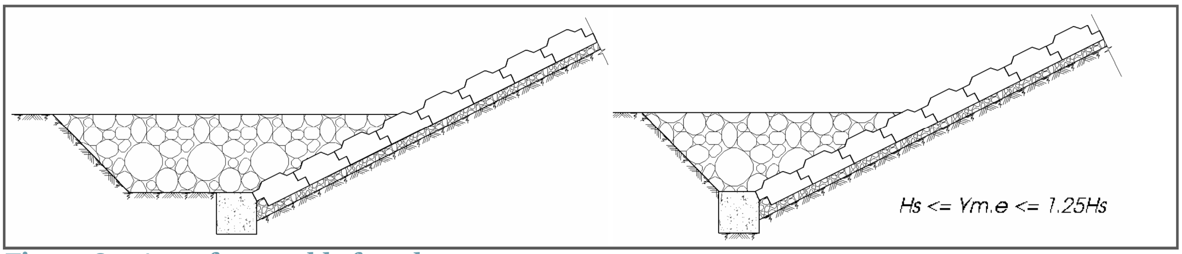


Figure 8-2 A toe for a stable foreshore

8.2 Toe for the Basalton® dike

For this project, a new toe is designed using the Dutch and the Vietnamese standard design guidelines. A stable toe-confinement with the placed Basalton® revetment is the basis for a good functionality of the revetment. In the Netherlands, it is common to use sheet piles to construct a toe. An example of this method is shown in Figure 8-3, a toe design is presented for a foreshore that is strongly eroded or when the foreshore consists out of cohesive soil.

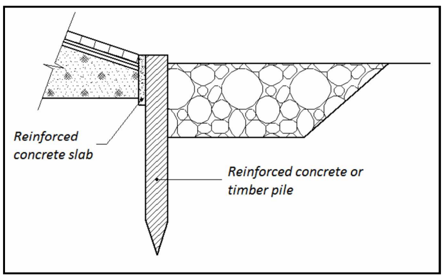


Figure 8-3 A toe with a driven pile structure

For the construction of sheet piles, heavy machinery is necessary. This machinery is not standard available in Vietnam. Work force in Vietnam is very cheap and the dikes are not easy to reach. That is why the Vietnamese do not want to use sheet piles for their toe protection. However, if it is possible and economically feasible, it will be a very strong and good option. With the current possibilities of Vietnam, it is preferred to have a design, which does not require a lot of heavy machinery.

Therefore, the Vietnamese now use concrete tubes as toe protections. This method is not well known in the Netherlands. Nevertheless, it could be a good solution/starting-point for a toe design. An example is shown in Figure 8-4. This design is used for areas where the near shore currents are strong.

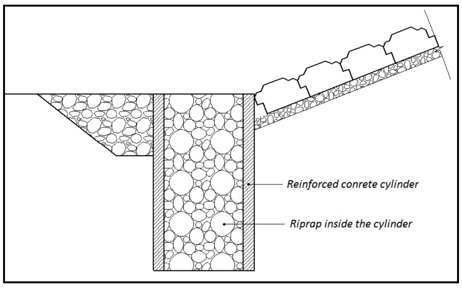


Figure 8-4 A Vietnamese toe

In the new design, the standard Vietnamese hexagonal tubes are used because the weakness of the current toes does not lie within the strength of the tubes itself. Its failure is within the placing of these concrete tubes. The reason they fail is that the needed buried depth in front is not sufficient in comparison with the acting scour depth in front of the toe. Therefore, the tube must be placed deeper into the ground, a result the toe gets more support from the surrounding soil. The concrete tube will be used to support the Basalton® blocks, therefore acting as a retaining wall.

If the foreshore in the tidal zone lies, at least 50 cm above the mean low water level[23] a buried toe can be used. In this project, the tidal zone lies 1 meter above the mean low water level so a buried toe structure can be used. The construction method for a buried toe structure is more difficult than for a non-buried toe. Building a buried toe takes more time; the reason for this is that it is needed to dig a hole before it is possible to place the stones. However, using this method requires less stones, this is because a toe protection on top of the sea bed needs at least a length of 3 to 4,5 times the wave height and a delved in toe only needs a length of around 2 to 3 times the wave height see Figure 8-1. Furthermore, a buried toe needs less maintenance and has a better resistance against storms than a non-buried toe under the same conditions. The sketch of the new toe is shown in Figure 8-5. In this figure, dumped rip-rap can be seen over the concrete tube and the lowest part of the Basalton® revetment. The reason that the Rip-Rap is also dumped on the transition between the revetment and the toe is because the transition is a one of the weak points in the structure. By dumping rip-rap on the transition and the lower part of the revetment, this weak point gets more support.

[23] (CUR, 1991)

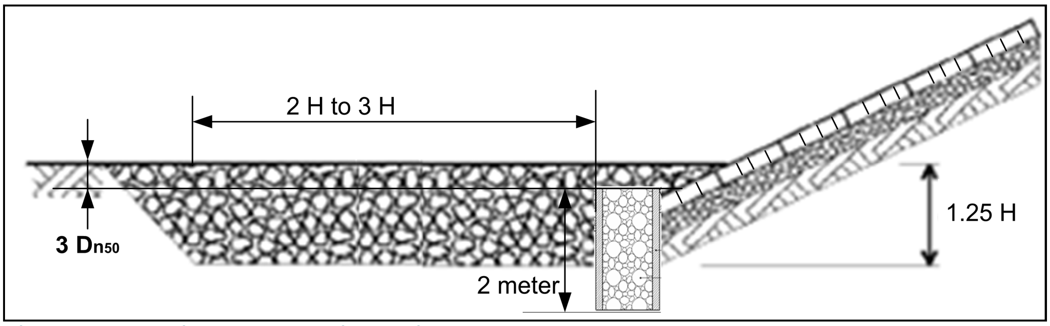


Figure 8-5 Design toe, the dimensions are correct but not on scale

8.3 Toe for the rip rap dike

Rip-rap is a revetment that has been used for a long time, also in Vietnam. For the rip-rap toe, a standard Dutch design is used that is shown at the right side of Figure 8-1. The inclination of the slope of the rip-rap revetment connected with the toe is 1 : 8, this is a very mild slope. Because of this mild slope, most of the wave forces will be dissipated on the structure face. Stones from the quarry are placed on a very flat slope and because of this the diameter of the stones can be reduced to $^1/_2$ or even $^1/_3$ of the primary cover. The minimum thickness of the cover layer as a toe apron should be two times the stones diameter of the calculated revetment. The new dike revetment has a design stone of D_{n50} = 0,47 meter. This means that the stones for the Rip-Rap toe apron should lie between the D_{n50} of 0,16 to 0,24 meter. This corresponds to a Vietnamese stone class of 6 based on the guidelines.

8.4 Calculations for the maximum possible scour depth

Below three different formulas are presented to calculate erosion near different hydraulic structures. However, none of the three formulas is particularly for dike revetments. The reason for this is that erosion in front of a revetment is quite un-familiar terrain. Nonetheless, formulas for breakwater and for vertical walls are used because no better alternative are available to calculate a possible scour depth.

As said before, based on the Dutch rules of thumb, the maximum scour depth will be 1.7m.

Second the formula of Fredsøe and Summer[24] (1997) is used. This is a formula, which is design for breakwaters. The formula of them is based on an empirical expression for the maximum depth of a scour hole at the lee side of a rubble-mounded breakwater.

$$\frac{S_{max}}{H_s} = 0{,}01\left(\frac{T_p\sqrt{gH_s}}{h}\right)^{3/2}$$

Within this formula the only unknown is S which is the, to be calculated scour depth. The h stands for the shallow water depth at the toe, T_p is the peak period and H_s the significant wave height.

By filling in the Fredsøe and Summer formula, the result will give a scour depth of 0,91 m. This is in comparison with the other formulas the lowest scour depth. This is likely because this formula is designed for breakwaters, which have large gaps between the Rip-Rap breaker zone. Most of the wave energy is absorbed by the stone layers of the breakwater and not reflected. What results in a lower amount of turbulence at the bottom and so less scour.

[24] (Kim)

The third possible method, which can be used to determine the possible scour depth, is the Xie[25] (1981) formula. He has found on empirically bases an equation to calculate the maximum scour depth at $\frac{1}{4}L$ of an impermeable vertical structure with regular incoming waves that has a normal incident and are non-breaking.

$$\frac{S_{max}}{H_s} = \frac{0{,}4}{[\sinh kh]^{1{,}35}}$$

This formula also has an S, which is the maximum scour depth. The h stands for the water depth at the toe. The k is the wave number, which is calculated by $(2\pi/L)$. Unfortunately, this formula is for calculating scour depths in front of rigid, massive structures. Incoming waves tens to fully reflect against this seawall. This reflection will in his turn increase the turbulence and stimulates the erosion of a deepening trench before the sea wall. In this project, a dike with a slope is designed. The erosion is different for a dike structure then for a vertical wall. Nonetheless, filling in the Xie formula gives a scouring depth of 2,89 meters.

The last equation that could help to determine a scouring depth in front of the dike is the equation from the Vietnamese guideline[26]. Based on some research this equation is created by Fowler (1992). Like the other formulas above it is empirically found, filling it in will give the maximum scour depth in front of a vertical sea wall. The formula is as follow:

$$\frac{S_{max}}{H_0} = \sqrt{0{,}25 + 22{,}71h/L_0}$$

The Fowler formula gives an extreme high scour depth (see Table 8-1), which is not very realistic. When the scour is in the order of 6 meter, most of the dikes even in the Netherlands shall collapse.

The results of the different methods are spread over a large domain (see Table 8-1).

Method	Rule of thumb from The Netherlands	Freds øe and Summer	Xie	Vietnamese guideline formula
Scour depth	1,7 meter	0,91 meter	2,86 meter	6,55 meter

Table 8-1 Results of the erosion formulas

For the erosion on the toe, a realistic value has to be chosen. Based on the rule of thumb of the Netherlands the maximum scour depth will be 1.7 m. However, the weather conditions in Vietnam are a lot heavier than in the Netherlands. In line with this, the erosion in front of the toe is considered larger than that is calculated with the rule of thumb. Based on the Xie formula and the knowledge that erosion in front of a vertical wall will be larger than the erosion in front of a water-retaining dike, the maximum scour depth must lie in between the rule of thumb of the Netherlands and the erosion formula of Xie.

$$rule\ of\ thumb < S_{max} < Xie$$

This means that the maximum scour should be between $1{,}7 < S_{max} < 2{,}86$. For this preliminary design, a S_{max} of 2,2 m is chosen.

[25] (Xie, 1981)
[26] (CWRCT, 2010 march)

8.5 Calculation of the stone size of the toe revetment

Based on the pervious chapter a scour depth has been chosen to act as the maximum scour depth that could occur within this preliminary design. Now that the maximum scour is known, the toe revetment stone size needs to be calculated. To do so, three different formulas can be used. However, just like the calculations for the scour depth in front of the revetment structure, these formulas are designed for breakwaters. Nevertheless, no better alternative are available to calculate the needed stone size, therefore these breakwater formulas will be used.

The first formula is the Hales & Houtson[27] (1983). They have done several tests with regular waves to determine the stability of a rock blanket extending seaward with a permeable rubble foreshore and a slope of 1:25. This empirical experiment resulted in the formula below, which can be used to determine the required stone size (D_{n50}) with respect to the needed length of the toe revetment B_p. H_b is the breaker wave height ($H_b \approx 0.7\ H_s$ for regular waves).

$$\frac{H_b}{\Delta D_{n50}} = 20\left(\frac{B_p}{T\sqrt{gh}}\right)^{2/3}$$

With the standard grading of Vietnam, the seaward extending of the toe protection is calculated. In Table 8-2 the results are given. It can be seen that when the stone size increases, the extending of the toe revetment will decrease.

Grading VN	D_{n50} [m]	B_p [m]
6	0,20 - 0,30	13 - 8,6
5	0,30 - 0,40	8,6 - 6,5
4	0,40 - 0,50	6,5 - 5,2
3	0,50 - 0,60	5,2 - 4,3
2	0,60 - 0,70	4,3 - 3,7

Table 8-2 Relation between B_p and D_{n50}

Unfortunately, the formula of Hales & Houtson is designed for breakwaters whereby the toe revetment always lies on top of the natural bed, which means that the calculation is only valid for a revetment type that lies above the seabed. However, the toe revetment design, situation on Cat Hai is below the natural seabed. First a calculation is done with the design rules of a toe that is buried. The buried toe, for a Basalton® dike, needs a length of 2 H to 3 H (see

Figure 8-1), this means a toe revetment with a length between the 3,4 to 5,1 meter. From Table 8-2 the minimal required stone size is a D_{n50} of 0,70m. However, as said before the empirical Hales & Houtson formula is carried out with a toe that is situated on the natural seabed. Thus, if the toe is not buried the extension of the toe has to be 3 H to 4.5 H which equals a toe revetment length of 5,1 to 7,65 m. Based on this the stone size of D_{n50} needs to be 0,50 m.

The formula of Hales & Houtson can only be used for toe revetments that are situated above the natural seabed. The stone size that is calculated for a toe revetment above the seabed has a D_{n50} of 0,5 m. However, the toe that is going to be used is the one that is going to be buried. The required stone size is thus not as said before a D_{n50} of 0,70 m but still the one that is used to calculate the toe revetment on top of the natural seabed, this has a D_{n50} of 0,50 m.

[27] (Pilarczyk, 1998)

The second formula which can be used to calculated the stone size of the toe revetment is the formula of “van der Meer”, d’Angermond and Gerding[28] (1995). They have investigated a relationship between the relative weight of the toe elements, the toe level and a damage factor (N_{od}) of the breakwater toe. This is valid for toes which are not too deep (only for $h_t/H_s < 2$). The formula is:

$$\frac{H_s}{\Delta D_{n50}} = \left(1{,}6 + 0{,}24\left(\frac{h_t}{D_{n50}}\right)\right) N_{od}^{0{,}15} \qquad 3 < h_t/D_{n50} < 25$$

Height toe [m] ($h - h_t$)	D_{n50}
0,5	0,29
1,0	0,37
1,5	0,44
2,0	0,52

Table 8-3 Relation between toe height and D_{n50} for a revetments/toe that is situated at the natural seabed

The bottom of this toe is located at the natural seabed. If the toe height rises, the D_{n50} also rises. If you construct a toe on the natural seabed, it must be 1 to 2 times the thickness of the primary cover layer (the design stone of the revetment are size D_{n50} = 0,47 m). In this situation the thickness of the layer will be approximately 1 to 2 meter thick. In the results are reflected. There is need for stones of D_{n50} = 0,37 to 0,52 meter.

Height toe [m] (h-ht)	Dn50
0,5	0,29
1,0	0,37
1,5	0,44
2,0	0,52

Table 8-4 Relation between toe height and Dn50 for a revetment/toe that is situated at the natural seabed

Height toe [m] (h- ht)	*Dn50*
0,5	0,29
1,0	0,37
1,5	0,44
2,0	0,52

Table 8-3 Relation between toe height and Dn50 for a revetments/toe that is situated at the natural seabed

The last possible formula that can be used is the one from the Vietnamese guideline:

$$V_{max} = \frac{\pi H_s}{\sqrt{\frac{\pi L_{sp}}{g} \sinh \frac{4\pi h}{L_{sp}}}}$$

This formula calculates a maximum velocity at the toe of the revetment and is based on the given parameters. The L_{sp} is the wave length at very shallow water which can be calculated with the wave length in shallow water ($T\sqrt{gh}$). By filling in the formula, the maximum velocity V_{max} is 1,43 m/s. Within Table 8-5 it is than possible to find which category of stones is needed to withstand this maximum velocity.

[28] (Verhagen, Roode, & d'Angeremond, 2009)

$V_{max}(m/s)$	2,0	3,0	4,0	5,0
$G_d(kg)$	40	80	140	200

Table 8-5 Stable weight of the stones per V_{max}[29]

This table comes from the Vietnamese guideline and the value G_d is equal to the W_{50} value in the Netherlands. The result is that a required stones weight of 40 kg is needed. Stones from 40 kilo with a density of 2600 kg/m^3 have a D_{n50} of 0,25 m and falls within the stone class from the Dutch standards of 10 to 60 kg.

The formula of Hales & Houtson is to calculate the stone size, based on the length of the toe revetment. The formula of “van der Meer” calculates the stone size of the toe based on the height of the toe revetment.

All three used formulas give different values, varying between D_{n50} of 0,25 to 0,52 meter. The largest value is a bit of an over estimation because this formula is for breakwaters with a toe revetment placed on the natural seabed. Within the design for Cat Hai Island the toe is designed as a buried one. The Vietnamese guideline formula gives a value that lies at the lower end of the needed toe revetment stone size. For the preliminary design stones will be chosen that lies between the D_{n50} of 0,39 to 0,44 which has a Dutch stone class of 60 to 300kg. This is for both types of revetment a good choice.

8.6 Conclusion

The depth of the scour hole and the stone sizes of the revetment are calculated with the formulas mentioned in the above paragraphs. The scour dept in front of the toe will be 2.2 m and a stone size, from the Dutch standard class of 60 to 300 kg, will be used within the finally design. An impression is given in Figure 8-6.

[29] (CWRCT, 2010 march)

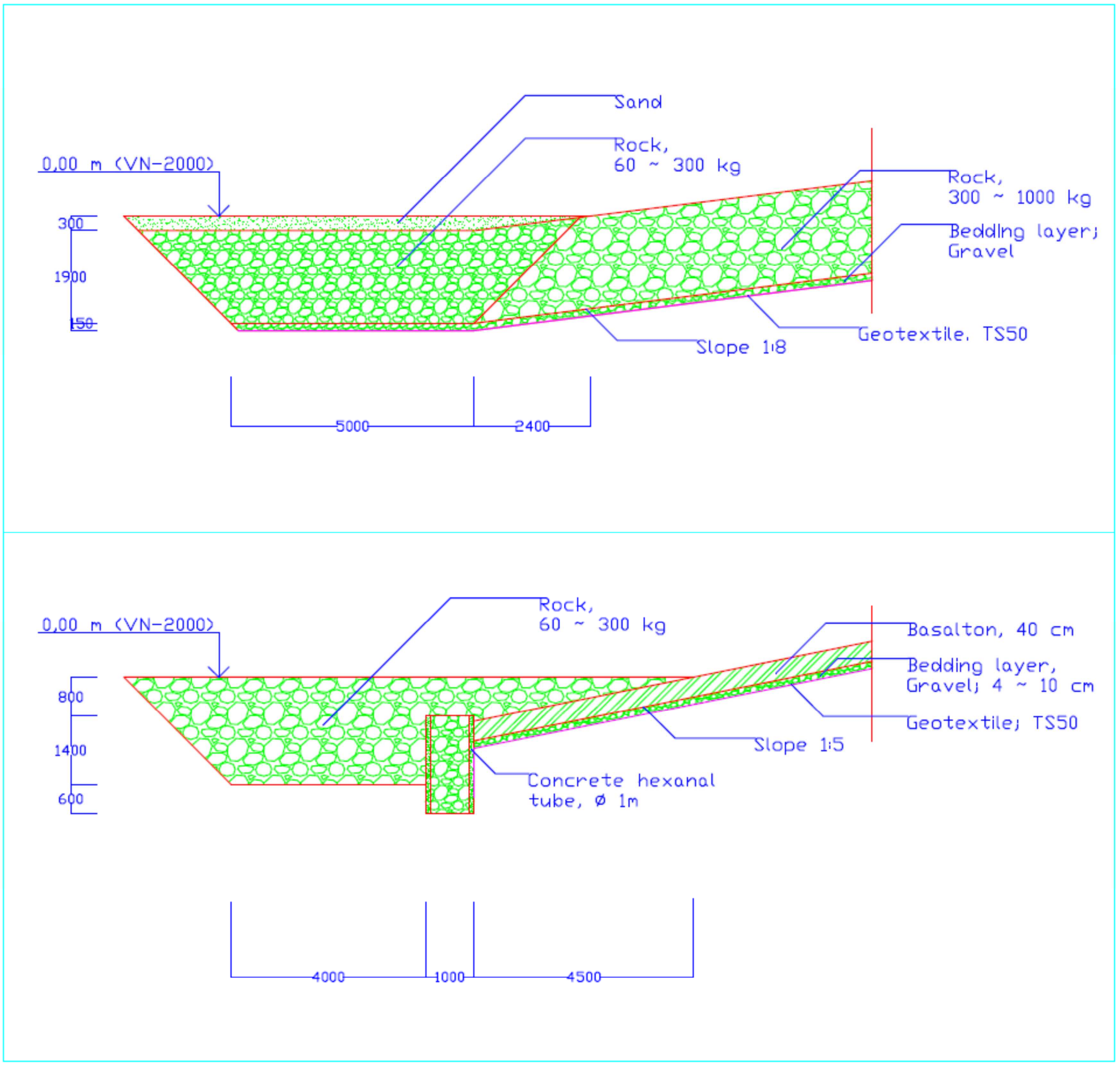


Figure 8-6 Final toe design

8.7 Toe parameters

H_b The breaker wave height (≈ 0.78 H_s for regular waves) **1,33** [m]

L The wave length in shallow water ($T\sqrt{gh}$) **60** [m]

h The shallow water depth at the toe **3,2** [m]

Δ Relative density ($\frac{\rho_s-\rho_w}{\rho_w}$) **1,52** [-]

ρ_s Density of stone **2600** kg/m³

ρ_w Density of water **1300** kg/m³

T_p Peak period **11,16** [s]

T Significant wave period [s] (for wind sea T ≈ 0.95 T_p) **10,6** [s]

H_s Significant wave height **1,7** [m]

g Gravity in Vietnam **9,79** [m/s^2]

N_{od} Character of damage (0.5 start of damage, **1,0** acceptable damage, 4.0 failure) [-]

k regular wave number $(2\pi/L)$ **0,11**$[m^{-1}]$

L_0 deep water wave length $(gT^2/2\pi)$ **194** [m]

H_0 deep water wave height **8,3** [m]

The variable parameters:

h_t The water depth above the toe [m]

S Scourer depth [m]

B_p The seaward extend of the toe protection [m]

D_{n50} Is the 50% passing nominal diameter with the size of the cube with equivalent volume to the block with median weight [m]

9 Logistics & Finance

9.1 Introduction

The construction of a dike is a large operation involving careful planning and placement. If the dike is not constructed correctly, the strength will be less than calculated in the design.

Currently the budget for dike construction in Vietnam is about 25 to 28 billion VND (= 25.000.000.000 VND) per kilometer. [30] This is 25 to 28 million VND per meter or about € 1.000,- per meter. For the test dike of 400 meters long the budget is 10 to 11.2 billion VND, or € 400.000,-. The budget for dike construction in Vietnam is in the order of magnitude twenty times lower than in the Netherlands.

Vietnam is divided in 58 provinces. Because there is no national contractor, most projects are done with local contractors, workers and materials. Thus, different costs for materials are applicable for each province. Furthermore the assumption that labour costs the same in every province is made.

For the dike construction at Cat Hai Island, the prices of the province of Hai Phong are used. A list of unit prices can be found in appendix 17.1. The Vietnamese use these prices, in combination with the productivity of its workers, to calculate the total cost of a dike. These prices are used in further calculations.

In this chapter an idea is sketched on the time and cost of the different phases of dike construction. In addition, the different possibilities of making and placing blocks are highlighted. The numbers and values are based on known knowledge and information given by the CWRCT and WRU.

In some cases the steps are quite simplistic, but this is the way the information has been published. Also, assumptions are made on production speed and cost by using common sense. Please note that because of this, the given time span and costs are just indications.

9.2 Cat Hai Island

Cat Hai is an island so materials must come by boat or ferry. The prices are nevertheless the same as in the rest of the Hai Phong province. During dike construction workers from Cat Hai will be used. In Vietnam it is common for workers to work, eat, sleep and live on the project site during the construction.

9.3 Assumptions

Before the different steps, possibilities and costs of the dike are explained, some assumptions are made. They are as follows:

- The dike dimensions used to calculate material needs and production speed is based on the Basalton® dike and rip-rap dike without allowable overtopping;
- One day/shift consists of 8 hours;
- A week consists of 5 working days;
- € 1,- = 25.000,00 VND.

[30] According to the CWRCT

9.4 Blocks fabrication/production

Blocks for the revetment can be produced in different ways. The supply chain gives an overview of what steps have to been taken for producing the blocks. For each of the different production methods, different ways of transport are required. Table 9-1 shows the universal supply chain; in the next paragraphs the different production methods will be explained. This paragraph will also give an overview about the assumptions made, possible production locations, prices and an advice on which method should be used.

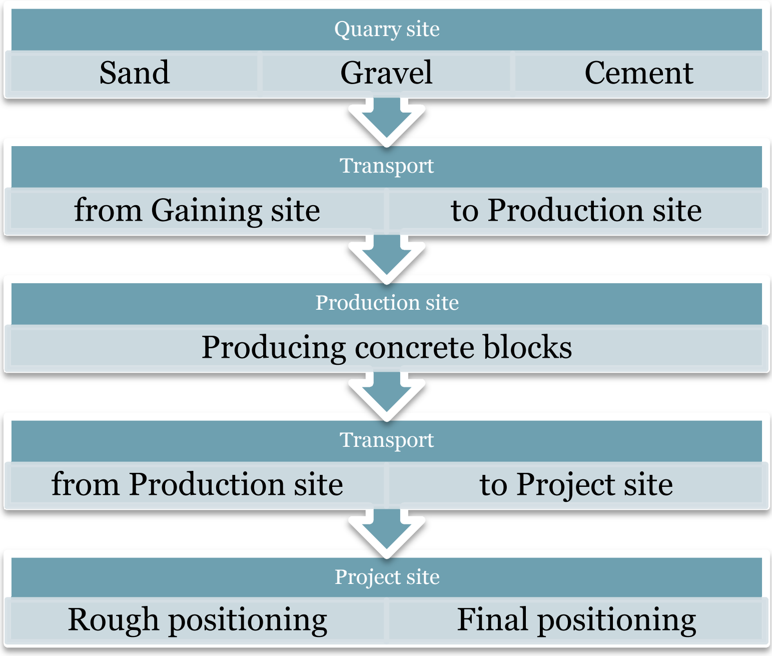


Table 9-1 Supply chain of producing the blocks

9.4.1 Production on site

For the production of Basalton® blocks on site, the Vietnamese use a norm to determine how many blocks can be made and at what cost. Taken into account are the necessary manpower, materials and the equipment, such as a concrete mixer, wheelbarrow and molds.

According to the norm, one 250 liter concrete mixing machine can produce 10,5 m^3 concrete per day when it is used non-stop. This is 1,3125 m^3 per hour. This amount also depends on the amount of workers on the job. The cost of producing 1 m^3 of Basalton® is about 1,15 million VND.

A detailed calculation is added in appendix 17.2.

The maximum amount of teams working to make blocks is limited by the available space to produce and store the blocks. This also depends on how fast the blocks will be placed, otherwise a situation could occur that blocks will have to be stored longer than necessary.

The advantage of producing the blocks on site is that the production speed can be adjusted to the placement speed of the dike. That means that the making of the blocks can be adjusted to the placement of the blocks more easily compared to blocks from a factory.

A disadvantage is that there is no quality control on the concrete mix and the curing process. Producing in open air doesn't help to improve the concrete strength either. Even though cement PC30 is used, after curing, the actual strength of the blocks is around 15-20 N/mm^2.

9.4.2 Factory in Vietnam

At the moment, in Vietnam blocks for dike revetment are not made in a factory. It is clear that producing blocks in a factory will have consequences for the price per m^3, such as:

- The amount of materials used will be different;
- Extra factory costs will have to be added to the price of the blocks;
- Technology cost for the factory will have to be added;
- Extra transport cost to site will have to be accounted for, roughly this is about 3,500 VND/kilometer.

It is assumed that when the production of blocks in a factory will be done on a large scale, the cost of the block will be lower than when the blocks are made on site due to the high efficiency of a factory. [31]

Because the blocks are made under a controlled environment, the quality of the blocks will be much better than when produced on site. The production speed of the Vietnamese factory depends on the design and investment of the factory. For a smooth placement of blocks on the dike, the blocks have to be delivered to the dike on time. This way, no blocks have to be stored longer than necessary and little storage area is needed on site.

Because so far blocks haven't been produced in a factory, there are no clear prices on how much it will cost to produce blocks in a factory in Vietnam. Further research will have to be done to determine how much it will cost to produce blocks in a Vietnamese concrete factory.

9.4.3 Movable factory

To produce high quality blocks, production has to been done in a controlled environment. This principle is contradictory to the current Vietnamese method of producing blocks in situ. When combining the benefits of the two methods, a transportable factory can be the solution. This way, the blocks are still produced on site, so the logistics don't have to change a lot, and a high quality concrete is achieved. The hard infrastructure in Vietnam isn't up to the level to carry heavy machines, so a movable factory by road isn't a realistic option.

The water infrastructure need almost no maintenance and is available for all dike projects, so a floating factory is more realistic. This factory can sail to every location where the dike will be made and deliver the blocks directly on site. Due to the revolutionary aspect of this principle, further research is necessary in order to design and make such a floating factory.

For the logistics of a floating factory, some parts have to be adjusted. Nowadays, the main transport is done by a truck, but for a floating factory the transport needs to be done by ship.

Figure 9-1 Example of a boat/barge to adapt for a the floating factory

[31] According to the CWRCT

One of the issues that has to be solved is the supply of the raw materials such as sand, gravel and cement to the floating factory. On the factory boat there will be very little storage area due to presence of the factory. However, it is possible to have a second storage boat which provides the raw materials. These storage boats can sail back and forth from the main supply location to ensure a continuous flow of raw materials. After producing, the blocks need to cure for about 3 weeks in a controlled environment. For this a third curing boat can be used. A fourth transport boat is needed for the transport from the curing boat to the shore. This boat requires a shallow draft to make sure the boat can moor at the shore. From the moored boat, manpower or machine can take over the prefabricated blocks and place them on the revetment.

9.4.4 Factory in NL

Basalton® is made in a Dutch factory in a controlled environment. The main materials needed to make Basalton® are sand, various kinds of gravel and cement. In the Netherlands these materials are delivered by ships and put into an automated concrete machine. The blocks are made by machine and hydraulic press. This way, no water has to be added; all the moisture comes from the sand. Because of this, high quality concrete with a high density can be achieved.

After the blocks are made, they are put in a curing shed for 24 hours. After these 24 hours the blocks are stored at a storage area and cured for a minimum of 28 days. Once cured, the blocks are ready to be put on trucks or ships and transported to their destination

During maximum production, 110 tons of material can be produced per hour. For blocks with a height of 25 cm, 180 m^2 can be produced per hour. For blocks with a height of 45 cm, this is around 100 m^2 per hour.

The strength of the blocks after 28 days of curing is about 60 N/mm^2. The standard asks for 50 N/mm^2 per stone and 45 N/mm^2 average for the set. This translates to a strength of about C60-C75.

For the test dike, the blocks will have to be shipped from the Netherlands to Vietnam. Based on the amount of m^3 that can be produced per hour, the production of the blocks for the pilot project can be done within one week. After the 28 days of curing, the shipping from the Netherlands to Hai Phong can be done within one month. The cost for shipping the blocks is unknown. For the long term, producing and shipping Basalton® from the Netherlands to Vietnam is not realistic.

To calculate the price for Basalton® blocks made in Holland, the Dutch price for one m^2 of Basalton® with a height of 20 cm is used. The 20 cm high blocks made in the Netherlands costs € 42,00, this is without transport and placement cost. Using this price the unit price per m^3 is calculated, from that the total costs are calculated. Using the exchange rate of 25000 VND per Euro the total price become 5,9 million VND.

9.4.5 Conclusion production blocks

It can be seen that producing blocks on site is the cheapest option, but the production speed and strength are lower than producing blocks in a (Dutch) factory. It is believed that in a Vietnamese factory, the same production speed and production strength can be achieved as in the Netherlands, as long as the production steps are carefully followed. Also, the production cost will be cheaper compared to a factory in the Netherlands.

Production method	Production speed (m^3/hour)	Productions costs (VND/m^3)	Production strength (N/mm^2)
On site	1,3	1.150.000	15
Vietnam Factory	?	?	?
Made in Netherlands	50	5.875.000	45

Table 9-2 Conclusion production blocks

It is recommended to make the blocks in a factory in Vietnam in a controlled environment.

9.5 Dike construction of Basalton®

The construction of the Basalton® dike includes a few phases, such as:

- Removal of old revetment;
- Placing of new, extra mountainous soil;
- Placing of clay;
- Placing of geotextile;
- Placing of stones;
- Placing of new revetment.

The different phases will be explained in the next few paragraphs. After this, a time schedule will be given.

Below is an overview of the different values that will be used in the calculations of the next paragraphs concerning the Basalton® dike. The construction of the rip-rap dike will be discussed in paragraph 9.7.

Parameter	Value	Parameter	Value
Length of dike	400 m	**Area of current dike**	6.550 m^2
Amount of soil – current dike	38.125 m^3	**Amount of soil – new dike**	87.800 m^3
Re-use of soil	80%	**Amount of soil necessary**	57.300 m^3
Thickness of clay layer	1 m	**Volume of clay**	15.000 m^3
Area of new dike	15.000 m^2	**Area of geotextile necessary**	15.000 m^2
Height of stone layer	0,40 m	**Volume of stone necessary**	6.000 m^2
Height of Basalton®	0,50 m	**Volume of Basalton®**	7.500 m^3

Table 9-3 Overview of different values used for calculations

9.5.1 Removal of old revetment

On the current dike at Cat Hai Island, there is 6.550 m^2 of blocks with a height of 28 cm. According to the Vietnamese norm, the removal of the old revetment is done by hand. The norm indicates that one man can remove 15 blocks per day. Thus to remove the old revetment, 3045 man days are required.

As can be seen in Table 9-4, this can be done in a short amount of time, requiring a lot of workers on site, or in a longer period meaning fewer man are needed.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	610	305	203	153	122	102	87	76	68	61

Table 9-4 Speed of removal the old revetment

Assuming the amount of workers needed and can be contracted is unlimited, the cost of removing the revetment by hand is 256 million VND, regardless of the necessary time.

The calculation can be found in appendix 17.3.

The Vietnamese norm does not give information on how the old revetment is processed once it has been removed. Neither are there any cost indications of this process available.

9.5.2 Placing of new, extra mountainous soil

The dike body is made of so-called 'mountainous soil'. It is cheaper to use clay but sticking to the Vietnamese guideline mountainous soil is used. When budget costs are needed, 1.700 million VND can be won to use clay instead of mountainous soil. According to the Vietnamese guideline, the soil is excavated from mountains and then transported to the location of the dike. The excavation is done by '1,25 m^3 excavators' and 'bulldozers 108CV'. The transport is done by 'automatic pouring trucks of 7 tons'.

One m^3 of mountainous soil costs 130.000 VND. The cost of transport is included in the material price, but to give an idea: transportation by a 10 ton truck costs about 3.500 VND/km. [32]

The placement of the soil can be done in two ways:

1. By manpower with a shovel and compactor;
2. By machine with a bulldozer 108CV and a 9 ton compacter.

With manpower, around 1,5 m^3 of soil can be placed per day, with a machine about 200 m^3 of soil can be placed per day.

The current body of the dike on Cat Hai Island has a volume of 38.125 m^3.

The new Basalton® dike will have a volume of 87.800 m^3.

Assuming 80% of the current dike body can be re-used, about 57.300 m^3 of new soil has to be placed on the dike. The amount of sand that can be re-used depends on the quality of the sand. The advantage of re-using the sand from the current dike for the new dike, is that the sand of the current dike is already well-packed, so little settlement will occur.

In Table 9-5 an overview of the time needed to place the soil compared to the amount of man, compactors and bulldozers per day is given.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	200	100	67	50	40	34	29	25	22	20
# Compactor	48	24	16	12	10	8	7	6	6	5
# Bulldozer	24	12	8	6	5	4	4	3	3	3

Table 9-5 Speed of placing new soil

The cost of the placement of the extra soil is 500 million VND. The cost for the soil itself is 7.450 million VND.

A detailed calculation can be found in appendix 17.4.

9.5.3 Placing of clay

On the dike, 1 meter of clay will be placed. This will have to be done with good care because it has to be carefully compacted. It is assumed that the placement of the clay layer will take twice as long per m^3 as the placement of small stones on the filter layer, which will be discussed in paragraph 9.5.5, due to the stickiness of clay. With a thickness of the clay of 1

[32] According to CWRCT

meter, a total of 15.000 m³ of clay is needed. This is two and half times more than the amount of small rocks needed. Thus by multiplying the placement cost of the stones by 5 (two times longer placing time and 2,5 times more material), the cost for the placement of clay is derived. This is 335 million VND.

The placement speed is a total of 5 times as long as placing stones. This result in the following amounts of trucks compared to the number of construction weeks.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Trucks	47	24	16	12	10	8	7	6	6	5

Table 9-6 Speed of placing clay

For the placement of small stones, only the use of trucks is considered. One can imagine also manpower or hydraulic cranes are needed to correctly place the clay layer. However, no numbers are available on this.

The cost of the clay itself is 100.000 VND/m³. Multiplying this with the total amount of clay needed, gives the cost for the clay: 1,5 million VND.

9.5.4 Placing of geotextile

According to the guideline, geotextile is placed by manpower and using bamboo piles. In total, about 15.000 m² has to be placed. The timescale in which this can be done compared to the amount of workers can be found in Table 9-7.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	43	22	14	11	9	7	6	6	5	5

Table 9-7 Speed of placing geotextile

The material cost for 15.000 m² of geotextile is 310 million VND, the cost of placing the geotextile is 18 million VND. Thus the total cost for placing the geotextile, including manpower, bamboo sticks and geotextile, is 328 million VND.

A detailed calculation can be found in appendix 17.5.

9.5.5 Placing of stones

To protect the geotextile from possible damage due to moving blocks, a layer of 40 cm with small rocks will be placed between the geotextile and the blocks.

In the Vietnamese guideline there is not a clear calculation on how long this should take or how much it costs. That is why the estimation has been made. To start, the total volume of stones necessary is found by multiplying the area of the dike with the height of the stone layer. This gives a volume of 6.000 m³.

According to the Vietnamese guideline, for carrying 100 m³ of soil a 7 ton truck is needed for 77% of its 8 hour shift.

To place 6.000 m³ of stones, 60 trucks are necessary. By multiplying the amount of truck necessary with the amount of truck shifts per 100 m³, which is 0,77, gives a total of 46 truck shifts to place the needed amount of stones. By multiplying the unit price for one truck per shift with the amount of shifts, the cost for the placing of the stones is found: 47 million VND.

After the stones are dumped, they have to be spread out by hand. However, there is no information on how many people are necessary for this.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Trucks	10	5	4	3	2	2	2	2	1	1

Table 9-8 Speed of placing stones

The stones cost 270,000 VND/m³. For a volume of 6,000 m³ this makes a total material cost of 1,62 billion VND.

9.5.6 Placing blocks

The blocks can be placed by either hand or machine. In Vietnam they are used to placing blocks by hand, because labor is cheap. In the Netherlands blocks are placed by machines because of strict rules on working conditions and expensive labor. In the next paragraphs, the different options for the placement of blocks will be explained. After that, they will be reviewed and the values used for calculations will be determined.

9.5.6.1 Vietnamese norm by hand

The Vietnamese guideline uses a norm for the placement of blocks by hand. The guideline is based on a maximum weight (100 kg) a worker can handle and a certain distance (30 meters) the block has to be moved before being placed.

Based on the standard, one worker can place 0,4 m² Basalton® per day. It is necessary to use 5 workers per day to place 2 m², costing 56.500 VND per m².

A detailed calculation of this can be found in appendix 17.6.

9.5.6.1.1 Remarks on this method

The current Vietnamese guideline assumes one worker can place just 4 blocks per day. This means a worker has 2 hours to place one block. It is believed that this is a very low production level, even when considering one man has to carry one block of 100 kg for 30 meters. It is assumed no mechanical help is used whatsoever.

9.5.6.2 Dutch way by hand

The Dutch way is based on the area one stone setter can place per day by hand, with the help from a small machine.

A worker can place around 30 m² per day at a cost of 3.300 VND per m². This price does not include the helping machine. This machine might add a substantial amount to the price of the cost of placing the blocks.

Figure 9-2 Placing by hand with small machine

A detailed calculation can be found in appendix 17.7.

9.5.6.2.1 Remarks on this method

It is assumed the labor price is of the Vietnamese level; this is less than the Dutch wage level. Comparing the Dutch and Vietnamese placing speed by hand, a Dutch worker is 75 times more effective. The price of the helping machine is not included.

9.5.6.3 Observation in Nam Dinh province

During a site visit to the Nam Dinh province, dike constructions were going on: Am Duong blocks were being placed by 10 workers with the support of 3 cranes.

One crane was used to place the blocks to the location they had to be placed, so the workers didn't have to carry them. The second and third cranes were used to dig away soil.

In total 450 Am Duong blocks of 140 kg per piece could be placed per day, equaling 72 m² per day. When looking at the price, the cost is 81.000 VND per m².

In case no extra soil has to be excavated, one crane will be enough. In this case, the cost of placing one m² by 10 workers and 1 crane is 35.400 VND.

The exact calculations can be found in appendix 17.8.

Table 9-9 Placing blocks with hand and machine

9.5.6.4 Vietnamese way placing by machine

For the placement of blocks by machine, the Vietnamese guideline is used. In this guideline the necessary manpower and equipment are defined. Using the Vietnamese guideline, one crane can place 20 m² per day at a total cost of 4,29 million VND. This is 214.000 VND per m².

A detailed calculation can be found in appendix 17.8.

9.5.6.4.1 *Remarks on this method*

The Vietnamese guideline uses an interesting approach for determining the placement speed and cost of the revetment. For the placement of the blocks, it is assumed that one crane needs 1/20 shift to place one block. This is one block per 24 minutes. It is faster than placing by hand but still very slow in comparison to the Dutch machine standard.

9.5.6.5 Dutch way placing by machine

In the Netherlands blocks are placed by machine with a special clamp for Basalton® blocks. An experienced team can place around 500 m² per day. In total one crane and three people are necessary: one crane operator, one person at the pallets and one person at the placement of the blocks. Using the Vietnamese price levels, without the clamp, the cost is 3.900 VND per m². This is without a hydraulic clamp. In the Netherlands, a Basalton® clamp to mount on a hydraulic crane costs 35.000 euro per piece.

A detailed calculation can be found in appendix 17.10.

9.5.6.5.1 *Remarks on this method*

It is assumed the labor price is of the Vietnamese level; this is less than the Dutch wage level. The mentioned 500 m² per day is based on an expierenced team. Because the method is new to Vietnam, the production level will probably be lower than 500 m² per day. For the placement of Basalton® blocks a special clamp is necessary. The cost of a Basalton® clamp in Vietnam is unclear. For placing the so-called end sets (see paragraph 2.8.5.4), a regular clamp can be used. This is a lot cheaper than having to use a special clamp. Further research is necessary to see whether the whole revetment can be made of these end sets.

9.5.6.6 Overview different methods

Now that the five methods have been described, an overview of the placing speed and placing cost can be made. This overview can be found in the table below.

Placing method	Placing Speed [m²/day]	Placing Costs [VND/m²]	Remarks
By hand norm	0,4	56.500	Per worker, per day
By hand Netherlands	30,0	3.300	Per worker per day, price of helping machine not included
By mix hand/machine	72,0	35.400	For 10 workers and 1 crane
By machine Vietnam	20,0	214.000	By assuming 1 lift per 24 minutes
By machine Netherlands	500,0	3.900	For an experienced team

Table 9-10 Overview of different placing methods

From the table it can be seen that the placing cost the Dutch way, by either hand or machine, is very cheap compared to the Vietnamese norm and observation. However, for the cost of the Dutch method the price of Vietnamese labor is used. In the Netherlands the placement of blocks by hand is much more expensive because of higher labor cost. Also, the cost of extra equipment such as the helping machine and the clamp are not taken into account.

9.5.6.7 Determining method for calculating speed and cost

To determine what placement speeds are realistic, the different methods for placing blocks described above are reviewed.

9.5.6.7.1 By hand, no help

From the observation of the dike construction in Nam Dinh province, 450 blocks were placed by 10 men who were helped by a crane to get the blocks to the workers, thus limiting the transport by hand. A productivity of 450 blocks by 10 men per day is equal to about 45 blocks per man per day. This is around 5-6 blocks per man per hour.

Assuming the blocks also have to be carried to the location, it is supposed that the placing of blocks will be reduced to 3 blocks per hour. This is 24 blocks per day, totaling around 4 m² per man per day costing 90.600 VND. This is 22.650 VND per m².

9.5.6.7.2 By hand, with a small machine

This option is not looked at because it is not interesting from a Vietnamese point of view. This is because in Vietnam the rules on labor are not as strict as in the Netherlands. The Vietnamese rather carry and place the blocks by hand than placing them with this machine.

9.5.6.7.3 By combining hand and machine power

Using the same method as in 9.5.6.3, the cost of placing one m² by 10 workers and 1 crane is 35.400 VND. Per day 72 m² can be placed. A calculation of the cost can be found in appendix 17.8.

9.5.6.7.4 By machine

For the machines, there is a big difference in production speed and price level between the Vietnamese guide (20 m² per day, 214.000 VND per m²) and the Dutch experience (500 m² per day, 3.900 VND per m²), when based on Vietnamese prices.

There is a different approach in the calculations. For the Vietnamese level the cost of the material, manpower and construction cost are based on the fact that they could be used elsewhere. This way, per lift an amount of manpower or crane is necessary. For the Dutch approach, it is assumed one day of placing blocks 1 crane and 3 workers are necessary.

For calculations purposes the Vietnamese and Dutch methods are combined. Assuming every 2,5 minutes a block can be placed, the production level is raised by almost 10 times compared to the Vietnamese guideline. This means 192 blocks of Basalton®, equaling 250 m³, can be placed per day.

Adjusting the Vietnamese guideline to the new production level, results in a cost of 21.600 VND per m². For a detailed calculation, see appendix 17.11.

9.5.6.7.5 Summary

In the following table, all the alternatives are shown.

Placing method	Placing Speed [m²/day]	Placing Costs [VND/m²]
By hand	4	22.600
By mix hand/machine	72	35.400
By machine	250	21.600

Table 9-11 Placing speed and placing cost of reviewed alternatives

9.5.6.8 Time span for placing by hand

From the values in Table 9-11, the amount of weeks and number of manpower can be found. An overview for the placement by hand can be found in Table 9-12.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	744	342	248	186	149	124	106	93	87	75
# Weeks	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
# Man	68	62	57	53	50	47	44	41	39	37

Table 9-12 Time span for placing by hand

The total cost for placing by hand is 337 million VND.

9.5.6.9 Placing by mix

From the values in Table 9-11, the amount of weeks versus the number of manpower and equipment can be found. An overview for the placement by hand and machine can be found in Table 9-13.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	413	207	138	103	83	69	59	52	46	41
# Machine	41	21	14	11	9	7	6	6	5	5

# Weeks	11	12	13	14	15	16	17	18	19	20
# Man	38	34	32	30	28	26	24	23	22	21
# Machine	4	4	4	3	3	3	3	3	3	3

Table 9-13 Number of weeks versus the number of man and machine per week

The total cost for placing by hand is 527 million VND.

9.5.6.10 *Placing by machine*

From the values in Table 9-11, the amount of weeks versus the number of manpower and equipment can be found. An overview for the placement by machine can be found in Table 9-14.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	48	24	15	12	9	9	6	6	6	6
# Machine	16	8	5	4	3	3	2	2	2	2

# Weeks	11	12	13	14	15	16	17	18	19	20
# Man	3	3	3	3	3	3	3	3	3	3
# Machine	1	1	1	1	1	1	1	1	1	1

Table 9-14 Number of weeks versus the number of man and machine per week

- Note: For every machine, 3 workers are necessary to operate the machine.

The total cost of placing by machine is 321 million VND. This is the price assuming no special clamp has to be bought, thus all the cranes cost the same per day.

9.6 Summary/conclusion on placement speed and cost

In this paragraph a summary is given on the placement speed, placement cost and material cost of the Basalton® dike.

9.6.1 Placement speed

During the construction, there are different phases. To construct as efficiently as possible, different phases can be done at the same time. This way, the working time of the project can be optimized. However, because so many assumptions have been made about the placement speed, further research is necessary before this can be done.

9.6.2 Placement cost

An overview of the cost for placement by hand, mix and machine is given in Table 9-15:

[in million VND]	By hand	By mix	By machine
Removal of old revetment	256	256	256
Placing of new, extra mountainous soil	500	500	500
Placing of clay	235	235	235
Placing of geotextile	18	18	18
Placing of stones	47	47	47
Placing of new revetment	337	527	321
TOTAL (for 400 m)	**1.393**	**1.583**	**1.377**
TOTAL (per meter)	*3,5*	*4,0*	*3,4*

Table 9-15 Cost of placement by hand, mix and machine

9.6.3 Material cost

The material cost can be found below:

[in million VND]	On site	In Dutch factory
Revetment	8.600	44.000
New, extra mountainous soil	7.450	7.450
Clay	1.500	1.500
Geotextile	310	310
Stones	1.620	1.620
TOTAL (for 400 m)	**19.480**	**54.880**
TOTAL (per meter)	***49***	***137***

Table 9-16 Material cost

By comparing Table 9-15 and Table 9-16 Material cost it follows that the material cost largely exceeds the placement costs. It is also clear that producing blocks on site is almost 3 times cheaper than in a Dutch factory, but bear in mind that the Dutch factory is based on Dutch labor costs.

It can be seen that placement by machine and placement by hand are the cheapest options. The most expensive option is placement by hand with the help from a crane.

The cost for upgrading the current dike on Cat Hai Island is higher than the current Vietnamese budget for dike construction. This is because the new dike is designed following the new guidelines, which means the dikes are stronger, but also bigger. This requires more materials, manpower and equipment to construct the dikes.

9.7 Placing rip-rap

Using loose rock as revetment can be done almost anywhere along the coast. Only when accessibility is an issue loose rock becomes a bit more difficult. In a mountainous country, such as Vietnam, rocks are usually easily available. In Vietnam rip rap often means placed rock. This requires a lot of man power which is abundantly available in Vietnam. For the pilot dike the assumed method is to dump the rock on place. For this method relatively a lot of machines are needed with little manpower. When dumping the rock, it will get a stable natural shape. From this natural shape rip-rap gets its strength. Underneath the heavy top layer a few filter layers are essential. These filter layers consist of smaller grading rock which provide stability and permeability. Finally, a geotextile is used to separate the stones and gravel from the clay. Due to the natural shape of the dumped stones, the revetment is easy to repair by dumping some stones on placed where it’s needed.

The rocks in Vietnam come from the quarries in the middle of Vietnam. The rocks have to be shipped or trucked to the north of Vietnam. Shipping is the cheapest solution but requires some more transfer from one transport device to the other and waterways near the quarry. Because there are no waterways near the quarry the transport will be done by truck.

9.8 Dike construction

The construction of the rip-rap dike includes a few phases, of which some are the same as the Basalton® dike. The phases are listed below:

- Removal of old revetment;
- Placing of new, extra mountainous soil;

- Placing of clay;
- Placing of geotextile;
- Placing of stones and rip-rap.

The different phases will be explained in the next few paragraphs. After this, a time schedule will be given.

Below is an overview of the different amounts that will be used in the calculations of the next paragraphs.

Length of dike	400 m	**Area of current dike**	6.550 m^2
Amount of soil – current dike	38.125 m^3	**Amount of soil – new dike**	64.225 m^3
Re-use of soil	80%	**Amount of soil necessary**	33.725 m^3
Thickness of clay layer	1 m	**Volume of clay**	13.160 m^3
Area of new dike	13.160 m^2	**Area of geotextile necessary**	13.160 m^2
Height of stone layer	0,15 m	**Volume of stone necessary**	2,000 m^3
Height of rip-rap layer	1,5 m	**Volume of rip-rap**	19.740 m^3

Table 9-17 Dike properties of the rip-rap dike

9.8.1 Removal of old revetment

The removal of the old revetment is the same as for the dike with blocks, see paragraph 9.5.1.

9.8.2 Placing of new, extra mountainous soil

The dike body is made of 'mountainous soil'. The placing is the same as the Basalton® dike, see paragraph 9.5.2. However, the volume of the new rip-rap dike is different is that of the Basalton® dike.

The new rip-rap dike will have a volume of 64.225 m^3.

Assuming 80% of the current dike body can be re-used, about 33.725 m^3 of new soil has to be placed on the dike.

In Table 9-18 an overview of the time needed to place the soil compared to the amount of man, compactors and bulldozers per day is given.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	118	59	40	30	24	20	17	15	14	12
# Compactor	29	15	10	8	6	5	5	4	4	3
# Bulldozer	15	8	5	4	3	3	3	2	2	2

Table 9-18 Number of weeks versus the number of man and machines per week

The cost of the placement of the extra soil is 290 million VND. The cost for the soil itself is 4,4 billion VND. The same calculation has been done as in appendix 17.4, but now with a different value.

9.8.3 Placing of clay

The assumptions made in paragraph 9.5.3 were also made in this paragraph. The clay layer has again a thickness of 1 m therefore, a total of 13.160 m^3 of clay is needed. This is 2,2 times more than the amount of small rocks needed. Thus by multiplying the placement cost of the stones by 4,5 (2 times longer placing time and 2,2 times more material, rounded from 4,4 to 4,5), the cost for the placement of clay is derived. This is 210 million VND.

The cost of the clay itself is 100.000 VND/m³. Multiplying this with the total amount of clay needed, gives the cost for the clay: 1,32 billion VND.

The placement speed is a total of about 4,5 times as long as placing stones for the Basalton® dike, because it is assumed it takes twice as long to place the same amount of m³, and also 2,2 times more clay than sand has to be placed.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Trucks	41	21	14	11	9	7	6	6	5	5

Table 9-19 Number of weeks versus the number of trucks working simultaneously

For the placement of small stones, only the use of trucks is taken into account. One can imagine also manpower or a hydraulic crane is needed to correctly place the clay layer. However, no numbers are available on this.

9.8.4 Placing of geotextile

Geotextile is placed by manpower and using bamboo piles. In total, about 13.160 m² has to be placed on the rip-rap dike. The timescale in which this can be done compared to the amount of workers can be found in Table 9-20.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Man	38	19	13	10	8	7	6	5	5	4

Table 9-20 Number of weeks versus number of man working simultaneously

The material cost of the geotextile is 275 million VND, the cost of placing the geotextile is 15,7 million VND. Thus the total cost for placing the geotextile, including manpower, bamboo sticks and geotextile, is 290 million VND.

The same calculation is used as in appendix 17.5, only with different values.

9.8.5 Placing of stones

To protect the geotextile from possible damage due to moving blocks, 15 cm of small rocks will be placed between the geotextile and the blocks.

In the Vietnamese guideline there is not a clear calculation on how long this should take. That is why an estimation has been made. The total volume of stones necessary is found by multiplying the area of the dike with the height of the stone layer. This gives a volume of 2,000 m³.

For carrying 100 m³ of soil by a self-dumping truck of 7 tons, 0,77 shift of trucks is necessary.

There are 20 trucks necessary to place the 2.000 m³ of stones. By multiplying this with 0,77 gives 16 truck shifts. This gives a price of 16 million VND for the placing of the stones. After the stones are dumped, they have to be spread out by hand.

The stones cost 270,000 VND/m³. For a volume of 2.000 m³ this costs 540 million VND.

# Weeks	1	2	3	4	5	6	7	8	9	10
# Trucks	3	2	1	1	1	1	1	1	1	1

Table 9-21 Speed of dumping stone layer on the rip rap dike

9.8.6 Placing of rip-rap

For the rip-rap, a diameter of 60 cm is used. Assuming the rocks have a cubical shape, the volume is $r^3 = 0{,}60^3 = 0{,}216\ m^3$. With a density of 2600 kg/m³, the rocks weight about 550 kg. Assuming a 7 ton truck is used, a truck can carry about 12 rocks.

Assuming the amount of space the rocks use can be characterized by a box, this is 0,6³ = 0,216 m³ per block. A total of 19,740 m³ of rip-rap is necessary, this is 91,400 rocks. For this 7600 truck dumps are needed.

Now it depends from where the rip-rap has to come, to determine the cost of placing the rip-rap. Assuming one truck needs a total of 8 hours (including the drive from and to the mountains) to dump, 12 rocks can be delivered per truck per day, thus a total of 7.600 truck shifts are needed.

Please note that this is just an assumption which has great influence on the final value, this can be seen in Figure 9-3, where the hours needed per dump are compared to the total cost of placing the rip-rap. For further use it is assumed the trucks need 8 hours to make one delivery of 12 rocks.

Hours per dumps	Total # of shifts	Cost (million VND)
2	1.904	VND 1.950
4	3.808	VND 3.890
6	5.712	VND 5.840
8	7.616	VND 7.790
10	9.520	VND 9.730
12	11.424	VND 11.680
14	13.328	VND 13.630
16	15.232	VND 15.570

Figure 9-3 Hours per dump of rip-rap versus the number of truck shifts and cost

Because the blocks are quite big, they have to be placed by a crane.

Following the assumption made in paragraph 9.5.6.7.4, a crane can place 1 rip-rap stone per 2,5 minutes. This leads to the following cost calculation:

A crane can place 200 rocks per day, this is 43,2 m³ per day per crane. One manpower can place 2,05 m³ per day. Thus in total 460 cranedays and 9600 mandays are necessary.

By multiplying the shifts and man days with the costs of the shifts per day, results in the total costs for the placement of rip-rap. This is 9.400 million VND.

The cost for buying rip-rap is 7.000 million VND. A calculation can be found in appendix 17.12.

9.8.6.1 Placing rip-rap from water

It is assumed that the rip-rap is transported by trucks and then placed by a crane and manpower. Because of the large amounts of rip-rap, it might be easier to deliver the rip-rap by a barge. From the barge the rip-rap can be placed by a crane, which is also situated on a barge. At the moment, there is no information on the cost for a barge or transport over water, but chances are this is cheaper and easier than transport over land. It is suggested to do further research on this.

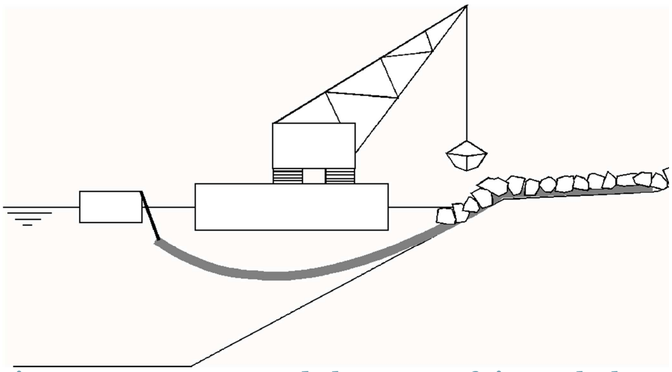


Figure 9-4 Transport and placement of rip-rap by boat

9.8.7 Summary/conclusion on placement speed and cost

In this paragraph a summary is given on the placement speed, placement cost and material cost of the designed rip-rap dike.

9.8.7.1 Placement speed

During the construction, there are different phases. To construct as efficiently as possible, different phases can be done at the same time.

9.8.7.2 Placement cost

The placement cost for the new rip-rap dike can be found below:

[in million VND]	
Removal of old revetment	256
Placing of new, extra mountainous soil	290
Placing of clay	210
Placing of geotextile	16
Placing of stones	16
Placing of rip-rap	9.400
Total (for 400 m)	**10.188**
Total (per meter)	*26*

Table 9-22 Placing costs rip rap dike

9.8.7.3 Material cost

The material cost for the new rip-rap dike can be found below:

[in million VND]	
Rip-rap	7.000
New, extra mountainous soil	4.400
Clay	1.316
Geotextile	275
Stones	540
Total (for 400 m)	**13.531**
Total (per meter)	*34*

Table 9-23 Material cost rip rap dike

9.9 Further research and advice

During the writing of the building process, speed and costs, it became clear that some information was missing or further research should be done in order to get a more accurate speed and cost indication. The things that should be researched are:

- Finding out the cost of producing Basalton® in a factory in Vietnam;
- Review the current norm to see whether the norm production can be upgraded, because now this is appears to be very low;
- Determine optimal dike construction time span;
- Split the producing and placing cost in start-up and variable cost;
- Research a movable factory on land or on water;
- Research cost and placement speed of clay, small blocks and rip-rap. In this report, these calculations are primarily based on assumptions;
- Research the possibilities of transport and placement of the dike by water;
- More tests to control the quality of the blocks which are produced.

9.10 Conclusion

The total price of the 400m dike for the different alternatives is as follows:

[in billion VND]	Basalton						Riprap
	On site			Dutch factory			
	By hand	By mix	By machine	By hand	By mix	By machine	By truck
Placement cost	1	2	1	1	2	1	10
Material cost	20	19	20	55	55	55	14
Total	**21**	**21**	**21**	**56**	**57**	**56**	**24**

Figure 9-5 Total price for 400m in million VND

This equals the following price per km dike:

[in billion VND]	Basalton						Riprap
	On site			Dutch factory			
	By hand	By mix	By machine	By hand	By mix	By machine	By truck
Placement cost	3	4	3	4	4	3	25
Material cost	49	49	49	137	137	137	34
Total	**52**	**53**	**52**	**141**	**141**	**140**	**59**

Figure 9-6 Price per km dike in million VND

9.11 Long term for Vietnam dikes

For the next 10 years, 1000 km of sea dikes in Vietnam will be rehabilitated by new blocks. This is 100 km per year and is a lot more than the 400 meter pilot dike project on Cat Hai Island.

The dike building season lasts for 6 months (December to May)[33]. Considering a month contains 20 working days and 28 days in total, this equals 120 weekdays and 170 days when the weekends are also included.

Once the Vietnamese are better known with placing blocks by machine, a crane can place 500 m^2 per day. This means during 6 months, a crane can place 60.000 m^2 when used during weekdays, or about 84.000 m^2 including weekends. With this placement speed, possible breakdowns is not taken into account. So working in the weekends can be the back up to make sure the availability of the machine will be up to level. Also maintenance can be done in the weekend to spare the precious time during the week.

The average dike height is 4,0 meters and has a slope of 1: 4[34]. This means that the width of the revetment is about 16,5 meters. When replacing 100 km per year for the next 10 years, a total of 1,65 million m^2 will be replaced.

According to the national program on rehabilitation of sea dikes from 2005-2015, it is planned to invest over 20.000 billion VND for 2.000 km sea dikes. However, in the first phases, from Quang Ninh to Quang Nam completed in 2010, only 3.000 billion VND was invested; this is 600 billion annually. It is expected that the annual investment level will be raised to 1.300 billion VND. [35]

When comparing this amount of money to the price of upgrading the current dike on Cat Hai Island to the dikes that have been designed, it is not nearly enough. The cheapest option, placing a Basalton® dike with blocks made on site and placed by machine, will require a total of 5.214 billion VND for 100km. The Basalton® dike, produced in a factory and placed by a machine will require an investment of 14.115 billion VND for 100 km. Upgrading the current dikes with a rip-rap dike will cost 5.950 billion VND for 100 km.

Thus for the safety of the Vietnamese hinterland during storms, it is advised that more money is invested for upgrading dikes in Vietnam by a substantial amount.

For the long term shipping the blocks from the Netherlands to Vietnam is not an option. That's why with the amount of dikes that will be upgraded in the coming years, there are a lot of possibilities for investing in new ways to make blocks and construct dikes in Vietnam. Setting up a Basalton® factory in Vietnam is not cheap, but the quality of the blocks and safety levels will greatly improve. It's definitely something worth considering.

[33] According to WRU
[34] According to WRU
[35] According to WRU

10 Monitoring plan

A monitoring plan describes how to inspect and maintain a dike. At the moment, the most important failure mechanism in Vietnam is wave overtopping. If there is monitoring plan in combination with maintenance, the lifetime of the dike will be expended. Also the investments needed for maintenance and repair due to monitoring & maintenance will lead on the long term to less cost. This can be characterised by the graph found in Figure 10-1.

The most important aspect of the monitoring plan is the execution of the periodic inspection and especially to pay good attention to the safety and maintenance requirements. The Dike Management Authority is responsible for the maintenance and monitoring of the dikes in Vietnam and has to follow the Dike-Code[36].

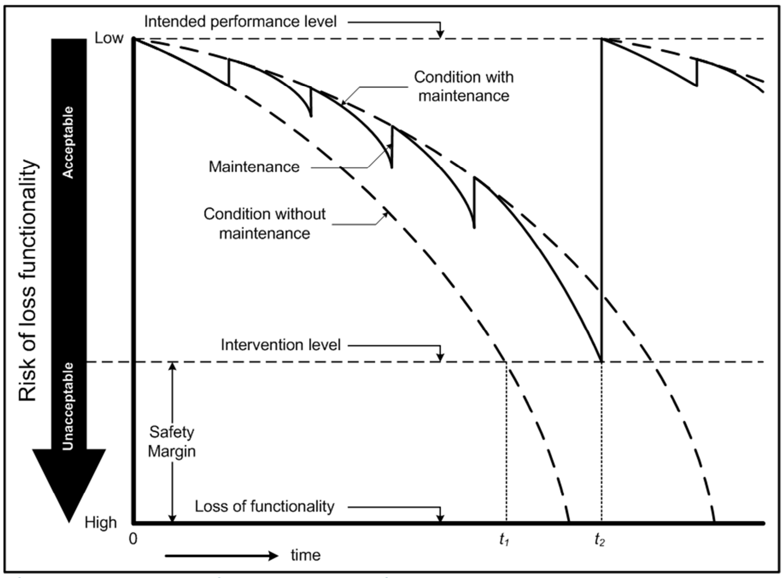


Figure 10-1 Overview of the ageing of a structure

10.1.1 Management strategy

This monitoring plan gives a suggested time interval for reparation and inspection of the dike. The plan is made for the pilot dike on Cat Hai Island, but can be implemented for all dikes in Vietnam. A good monitoring plan follows a cycle:

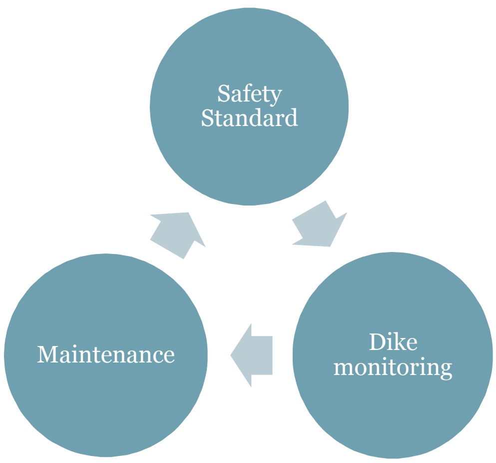


Figure 10-2: Monitoring Cycle

[36] As introduced on 1 july 2007

The first step is to determine the Safety Standard of the dike. Once the Safety Standard is identified the design parameters for the dike are known. After that, the dike can be monitored, this is the check whether the dike is consistent with the parameters. If so, no maintenance is needed. When the dike isn't in compliance with the parameters, maintenance is needed to get the dike back to the necessary dike condition. For Cat Hai Island the Safety Standard is a dike category IV as shown in paragraph 0. The design parameters are the requirements for the strength of the different parts of the dike taken into account the time span of the next monitoring moment.

For monitoring, the dike manager (from the Dike Management Authority) has to know the dike layout of each dike. Besides the layout, the manager needs to know what areas to inspect and how to label the area. Figure 10-3 shows an overview of the different elements of a dike cross section. Those labels can be used to pin point the exact location on the dike for inspection or maintenance.

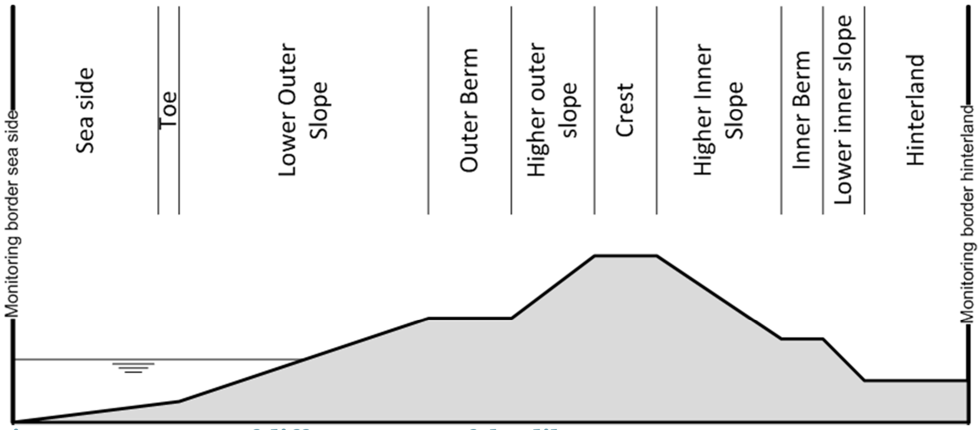


Figure 10-3 Names of different parts of the dike

10.1.2 Monitoring schedule

After the dike has been built, the project isn't completed. Monitoring and maintenance of the dike are very important to maintain the safety of the hinterland. Inspections to check the dike must be done regularly and event based. This means regular inspections before and after the typhoon season, and event based inspections before and after a heavy storm. Once in five years a big inspection and maintenance is necessary. Table 10-1 shows the maximum duration between two inspections:

Inspection type	Time interval
Geometry of the dike (Inner & Outer slope)	12 months
Positioning of stones on revetment slope	12 months
Physical-mechanical properties of the revetment	12 months
Foundation and scour	6 months
Total inspection	5 years
Visual inspection before and after storm	Event based
Visual inspection after reporting a problem by local people	Event based

Table 10-1 Monitoring intervals

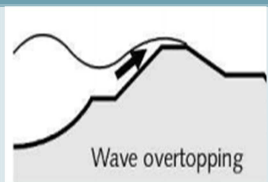
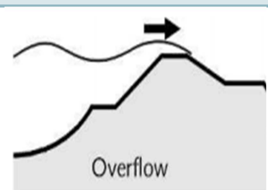
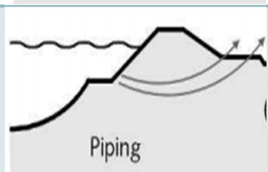
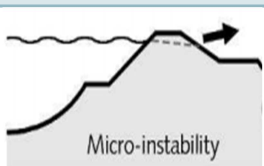
10.2 Monitoring methods (How to monitor)

There are a lot of different monitoring methods, varying from advanced measuring equipment to basic visual inspection. Advanced equipment is expensive, very accurate but is sensitive for break down. This method is not realizable in Vietnam. A better, cheaper and simpler solution is the visual inspection. The inspectors have to be trained in a visual inspection. The visual inspection could be a good tool if there are a good progressive scheme and checklist. The visual inspection is best suitable for Vietnam and Cat Hai Island because Vietnamese labour is a lot cheaper than in Western countries.

It is important to point out that the dike manager must have a good understanding of the dike, its behaviour, the possible failure mechanisms and the monitoring plan. Last but not least, the monitoring plan must be followed carefully.

10.2.1 Failure mechanisms

The understanding of failure mechanisms is essential for the quality of the monitoring. Table 10-2 gives an overview of the failure mechanisms. The table also gives an overview on how to recognize the different failure mechanisms and what to do if it happens. Appendix 16 gives a more detailed description of the failure mechanisms.

Failure mechanism	Visual description	How to recognize	How to solve
Wave overtopping		Waves are coming over the crest of the dike	• Increase roughness outer berm • Increase crest height
Overflow		Water is flowing over the crest during high water level (during a storm/typhoon)	• Increase crest height
Piping		Seepage water that reaches the inner slope of a dike. Erosion will occur underneath a cohesive layer in the subsoil.	• Increase the leakage length • Making tubes for drain off the water
Micro stability		Seepage water that reaches the inner slope of a dike.	• Making tubes to drain off the water
Macro stability Outer slope		The water in front of the dike drops very fast. Water inside the soil mass can't follow and the pressure of the water inside the dike will cause the outer slope to slide.	• Create a berm • Make the slopes less steep • Increase strength and stability of the filter layer, clay layer and inner slope • Increase permeability of the slope

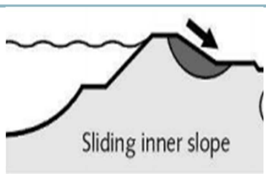
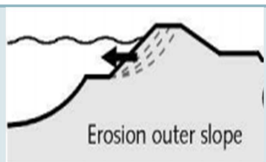
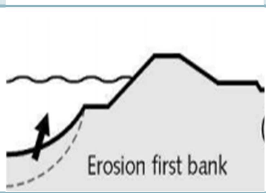
Macro stability Inner slope		When it is high water for a long time water can seepage to the dike body and create a pressure difference that will cause the inner slope to slide	• Make the slopes less steep • Increase strength and stability of the filter layer, clay layer and inner slope
Revetment stability		The revetment isn't at the right place anymore	• Repair (of a part) of the revetment
Toe stability		Scour hole in front of the toe.	• Dumping rock or sand in front of the toe for protection of scour

Table 10-2 Overview of failure mechanisms

10.2.2 Non water retaining objects

If there are non-water retaining objects on and around the dike, the situation has to be evaluated and a possible instability of the dike has to be carefully looked at. Non water retaining objects are:

- Sluices;
- Buildings;
- Cables and tubes;
- Cultivated land, trees and roads.

Buildings, sluices, cultivated land, trees and roads are easy to recognize and to report. Cables and tubes are difficult to see by visual inspection, but the dike manager has to know the total amount and location of the cables in and near the dike body. These locations need extra inspection. New buildings and cables have to be avoided.

10.2.3 Measuring hydraulic conditions

To get an indication of the failure mechanisms it is important that the dike manager measures the hydraulic conditions. There are simple methods to measure the hydraulic conditions during and after storms.

For overflow there is a method to measure the water level during a storm. Constructing a tube in the water filled by oil and after the storm measure the highest oil level in the tube. After the storm the tube has to be cleaned, otherwise it is not possible to measure the water level during the next storm if this is lower than the storm before.

For overtopping there is another method to measure the impact of this problem, like measuring the water level in a reservoir behind the dike before and after a heavy storm. By putting a reservoir in the field behind the dike, the rainfall can be measured. The difference between the two reservoirs is the amount over overtopping.

For measuring the erosion at the toe of the dike, the dike manager has to use GPS or other survey material to determine the exact place. From that point the dike manager can determine the erosion by using a dipstick to measure the water depth.By knowing the tide level at the moment of surveying the exact level of the toe above the nautical reference level (VN 2000) can be calculated. This gives a good indication of the total amount of erosion in front of the toe.

If a drain system is present, it has to be checked whether it isn't blocked due to too much sand in the system. To prevent the blockage the system needs to be flushed regularly.

10.3 Evaluation

After monitoring, the findings must be evaluated to know whether the dike is up to standard. Table 10-3 is a useful tool to evaluate the dike inspection. For each criterion the length of the dike which is sufficient and insufficient can be written down. Based on this overview, the Dike Management Authority can decide which part of the dike has the priority to be repaired or which dike is in need the most for maintenance.

Criteria	Total length (Km)	Good (km)	Insufficient (km)	No Rating	Sufficient (%)
Height					
Stability					
Macro stability Outerberm					
Macro stability Innerberm					
Piping/ Micro stability					
Revetment					
Toe					
NON WATER RETAINING OBJECTS	**TOTAL AMOUNT**				
Building					
Cables and tubes					
Cultivated land, Trees and roads					

Table 10-3 Evaluation table of failure mechanisms

If a part of the dike is found to be inadequate during an inspection, the dike manager has to write this down in Table 10-3. If the measure methods don't give a result, the dike manager gives the criteria 'no rating'. The result 'good' can only be given if the measurements meet all the requirements.

10.4 Judgment of evaluation

The dike meets its requirements when all criteria's are rated as 'good'. When one of the criteria isn't in compliance with the requirements, maintenance is needed.

Based on the evaluation the dike manager has to make a decision. What part needs to be repaired, how much does it cost and will it be within the budget? If there is a signal of weakness during monitoring the dike needs maintenance.

10.4.1 How and when to repair

Every repair is unique thus it is difficult to describe how to repair. The dike manager has come to a decision on how to repair the dike.

Table 10-4 shows repair options for every problem area.

Problem Area	Options (coastal structures)
Slope and crest repair	• Chinking (adjustment of the armour surface by surface with smaller material), resurfacing • Addition of armour stone • Layer reconstruction • Raising the crest • Burial of existing armour stone
Toe and foundation repair	• Reconstruction of the toe • Scour apron • Addition of a toe trench
Core repair or void sealing	• Precast concrete blocks • Filter cloth (geotextile) • Grout
Failure damage	• Replacement of original structure • Complete removal of existing structure

Table 10-4 Options for repairing the dike

After storms the dike has to be monitored, if there is damage, it has to be repaired as soon as possible. During repair the whole damage-area plus surrounding needs to be taken into account, this way, no hidden damage will be left behind.

10.4.2 Use of monitoring

When the two pilot dikes are constructed the dikes have to be compared with each other to determine which dike is better. This can be done based on the damage that occurs on the two dikes. The wave attack on both dikes is the same so the amount of needed maintenance is a good indicator for how good the dike is.

Besides that the data gathered during monitoring can be used for designing new dikes. For example the wave overtopping data is useful to get a better understanding of the strength of the inner slope and to check whether the crest height is high enough. And the scour data is useful to get a better insight in erosion along the coastline. The data has to be compared to the design standards and if needed, the data should complement the design guideline for new dikes.

11 Conclusions

During this project, there are two dikes designed for a pilot project on Cat Hai Island: one 200 meter Basalton® dike and one 200 meter rip-rap dike.

The current dikes and its fail mechanisms in Vietnam have been analyzed. Wave overtopping is the most frequently happened, fail mechanism with the biggest impact.

In order to design a dike it is vital to know what kind of forces it should be able to withstand. This is partly given by design guidelines and the return time of a certain storm, the other part is determined by the boundary conditions for the specific locations. The dike is designed to withstand a typhoon of a strength which occurs once every 30 years. This results in waves with a significant height of 1,70 meter and a peak period of 11,2 seconds.

At the start of the project it was clear that one revetment would be designed with Basalton® as revetment type. However, the second revetment was yet to be determined. This has been done using a Multi Criteria Analyses (MCA). From this MCA it follows that rip-rap is a good alternative.

After this, the optimized dike geometry has been determined. Three situations are optimized for two different kinds of revetment. The three different situations are; a dike with block revetment based on the guidelines, a dike with a block revetment with an extra wide crest and a dike with rip-rap revetment. These three situations are optimized for a situation without overtopping and with an allowable overtopping of 1 l/s/m. The two designs that are chosen to continue to calculate are the dike with block revetment based on the guidelines and the dike with the rip-rap revetment, both without overtopping. The needed crest heights are 7,10 m and 6,15 m respectively. It was also determined that for the Basalton® dike an outer berm has a positive effect.

Once the geometry is fixed, the buildup of the core has been determined and tested on stability and settlement. The buildup of the two dikes exists of a sand body of variable height as base, a layer of clay of 1 meter in between and finally a geotextile with on top a 0,15 meter of gravel. For the filter layer geotextile TS-50 is used. On top of the 0,15 meter gravel the Basalton® blocks for the block dike are placed. From calculations it follows that Basalton® blocks with a height of 0,40 meter is needed from the toe on to the lower slope which is below the berm. On the berm it is decided that also Baslaton blocks of 0,40 meters is used. On the rest of the slope above the berm 0,50 meter high Basalton® blocks are required. For the rip-rap dike on top the gravel layer comes the rip-rap stones. The D_{n50} of the stones is 0,67 meter. This is bigger than the current stones which are excavated from the mountains of Vietnam. In order to retain bigger stones, it is advised to change the current quarry process. The maximum settlement of the dike is estimated to be around 0,20 meter.

An important part of a dike is the toe. For the design of the toe the depth of the scour hole and the stone sizes of the revetment are calculated. The scour depth in front of the toe will be 2,20 meters and required stone size, taken from the Dutch standard class is 60 – 300 kg. The toe is buried into the natural sea bed and has a deep body length of 5 meter.

The logistics and finance of the dikes are investigated. Currently the blocks are made in-situ, but producing blocks in a factory instead of on the dike will greatly contribute to the quality of the blocks. At the moment, this is not the main reasons why dikes fail but for the future, once the dike design is proper, it is important to extend the lifetime of the dike. The current production of blocks in a factory in the Netherlands is about 5 times more expensive than producing on site in Vietnam. However, this is based on Dutch price levels. It also has to be noted that the quality of the blocks and the production speed are a lot higher than when the blocks are produced in-situ.

For the placement of the revetment several options are considered: placing by hand, placing by hand and machine the Vietnamese way or placing by machine the Dutch way. Carefully looking at these methods leads to the conclusion that placement with a machine the Dutch way is the cheapest and fastest option. From the estimated finances it became clear that the current level of investment for upgrading dikes in Vietnam is not sufficient to meet the desired safety standards.

After the dikes are constructed, they need to be monitored. This can be done with the suggestions given in the monitoring plan. If there are investments needed for maintenance and repair due to the “monitor & maintenance plan” it is advised to do so, because this will lead on the long run to lower costs.

Further research is necessary in order to improve the quality of dike designs in Vietnam in general. This means that information from the Netherlands cannot just be copied to Vietnam, the reason for this is that the conditions are too different compared to each other. It does mean that the knowledge level about designing dikes in Vietnam has to be improved. This can only be reached by means of thorough research done on site in Vietnam.

12 Recommendations

In the boundary conditions the peak period is calculated based on the wave height. The formula is known for the high outcomes so better research to the peak period is necessary to obtain a more reliable value. When the peak period is more reliable the whole design become more realistic.

The maximum settlement is 20 centimeter, at different places on the two dikes. When the dikes are constructed it is important that the new layers are consolidated by machinery or get a temporality extra load/soil placed on it. This is important for the stability and the eventual settlement will be less. It is also handy for the building phase of the following geotextile and revetment layers.

The calculated revetment size in this report gives reasonable sizes values for Basalton® as well as for Rip-Rap. However, a recommendation can be made for the fact that the stone grading system in Vietnam is very poor. A land like Vietnam could easily build cheap and strong Rip-Rap dikes. Especially if the land, like Vietnam, is rich in natural stones.

Recommendation for the logistic part is that the production speeds of placing blocks are in reality much higher than which is given within the norms. Besides that information from the norm is too detailed which does not help the processing of it. It is wise to see if this too detailed norm is changeable to a form which has a better fit with the reality. Financially the design can be cheaper by 2 billion VND to use clay as dike build up material instead of mountainous soil. So it is recommended to investigate this possibility.

For this project toe information was limited especially about the structural erosion, in the future it is wise to do investigate the sediment transport along the dike. If there is long shore erosion it is wise to keep the maintenance level high and look for other alternatives like constructing groins or sand nourishment. It is advised to make a good working registration system to investigate the toes behavior at different locations.

Which least to the monitoring plan and on his turn leads to a key point, which is maintenance. The costs of maintenance are variable and are depending on the evaluation of the monitoring. It must be considered as the optimization in terms of money and stability for major maintenance. Now it is estimated at 5 years. However, time can only tell if this estimation was feasible, this maintenance can also be slightly earlier or later depending on Mother Nature and the quality of the constructed dike. From the collected information of the monitoring plan can be used for future designs of dikes along the Vietnamese coast.

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14 APPENDIX Soil layers

Criteria	Unit	Layer1	Layer2	Layer3	Layer4	Layer5	Layer6	Layer7	Layer8	Layer9	Layer10	Layer11	Layer12
Granular composition													
+ Clay grain	%		20.0	24.60	8.0		9.5		13.30	9.8	12.4	6.0	20.80
+ Dust grain	%		19.9	25.30	14.5	1.0	20.6	1.8	21.70	19.2	18.7	16.40	29.00
+ Sand grain	%		60.1	50.10	77.5	97.2	69.9	98.2	65.0	71.0	68.9	77.60	50.20
+ Gravel	%					1.8							
Atterberg Limit													
+ Liquid limit	%		36.04	42.36	27.92				37.13	30.43	30.53		44.56
+ Plastic limit	%		23.49	26.47	19.60				24.63	20.98	21.04		27.78
+ Index of plasticity	%		12.55	15.89	8.32				12.50	9.45	9.49		16.78
Consistency B			0.500	0.860	0.952				0.861	0.671	0.790		0.998
Moisture content	%		29.76	40.13	27.52		29.94		35.16	31.43	28.54	28.48	44.53
Density													
+ Wet density g_w	x10^3 kg/m^3		1.76	1.71	1.85		1.82		1.70	1.77	1.81	1.82	1.62
+ Dry density g_c	x10^3 kg/m3		1.36	1.22	1.45		1.40		1.26	1.35	1.41	1.42	1.12
Specific weight	x10^3 kg/m3		2.70	2.69	2.68	2.68	2.67	2.68	2.69	2.67	2.68	2.67	2.68
Porosity n	%		49.76	54.64	45.87		47.6		53.39	49.45	47.46	46.95	58.18
Void ration e_o			0.991	1,024	0.847		0.918		1,148	0.991	0.903	0.885	1,391
Degree of saturation G	%		81.11	89.63	87.04		87.41		82.41	84.59	84.68	85.94	85.80
Average cohesion C_{tb}	kg/cm2		0.12	0.05	0.09	0.08	0.07	0.09	0.08	0.07	0.08	0.07	0.04
Internal friction angle f_{tb}	Deg		8,00	6,00	20,41	20,00	11,00	20,00	6,00	16,00	10,00	15,00	5,00
Compaction ratio a_{1-2}	cm2/kg		0.034	0.052	0.052	0.012	0.020		0.040	0.026	0.033	0.022	0.052
Permeability K	cm/s		1.2×10^{-5}	7.1×10^{-6}	3.3×10^{-4}	4.0×10^{-3}	6.9×10^{-5}	2.6×10^{-3}	8.1×10^{-5}	5.6×10^{-4}	8.5×10^{-5}	4.2×10^{-4}	8.2×10^{-6}
Koppejan													
Primary compression coefficient													
+ below preconsolidation Cp	-		270	19	1418	768	79			764	78		
+ above preconsolidation C'p	-		70	10	45	600	25			200	25		
Secondary creep compression coefficient													
+ below preconsolidation Cs	-		1900	40	1500	1500	650			1500	650		
+ above preconsolidation C's	-		1900	40	1500	1500	650			1500	650		
Primary swelling coefficient Ap	-		270	19	1418	768	79			764	78		

15 APPENDIX Matlab scripts

15.1 Optimazation berm without overtopping

```
clear all; close all; clc;

%% Effect of the outer berm (without overtopping).
    %What is the effect of a berm for different width and levels
    %Design in combination with calculation of the inclination, doc.:
    %inclination.
    %Itteration: bermwithd & bermlevel --> inclinations --> crestlevel -->
    %overtopping --> bermwithd & bermlevel --> etc.

%% Parameters

%Iteratiff parameters
%Results out of Inclination.m
m1 = 5            ;% [-]        Inclination of the outer slope, beneath the berm (1:x)
m2 = 2.5        ;% [-]       Inclination of the outer slope, above the berm (1:x)

%Parameters
B_crest = 4       ;% [m]        Width of the crest (see "Technical standards in sea dike design", table 5.4, dike grade IV)
Ztk = 3.17        ;% [m +M.S.L.] Design water level
a = 0.3           ;% [m]        Safety height increment (see "Technical standards in sea dike design", table 5.2, dike grade IV)
m3 = 3           ;% [-]        Inclination of the inner slope (1:x)
Hm0 = 1.70        ;% [m]        Design Wave height at the toe
Tp = 11.16        ;% [s]        Peak period
alfa = 1.15       ;% [-]        1.1 ~ 1.2
beta = 0         ;% [degree]   Angle of incident waves
gamma_f = 0.9     ;% [-]        Reduction factor roughness elements (see "Technical Standards in Sea dike design", Appendix A, Table 1)
g = 9.81          ;% [m/s2]

%Current situation
Ground_level = 0  ;% [m +M.S.L.] Ground level
%Current dike
B_h_crest = 7.5       ;% [m]        Width of the crest (current dike)
Crest_level_h = 4.80  ;% [m +M.S.L.] Crest level (current dike)
B_h = 5.0             ;% [m]        Width of the inner berm (current dike)
Berm_level_h = 3.00   ;% [m +M.S.L.] Berm level (current dike)
mh1 = 3.5                 ;% [-]         Inclination of the outer slope (current dike)(1:x)
mh2 = 2               ;% [-]        Inclination of the inner slope, beneath the berm (current dike)(1:x)
mh3 = 2                ;% [-]        Inclination of the inner slope, above the berm (current dike)(1:x)

%Costs
P1 = 130000        ;% [VND/m^3]   Price of sand
P2 = 570000        ;% [VND/m^2]   Price of block revetment
P3 = 140000        ;% [VND/m^2]   Price to buy agriculture land

%% Calculations: Effect of the outer berm

%Reduction factor for oblique incident waves (gamma_beta) (for wave run-up)
    gamma_beta = 1 - 0.0022*abs(beta);
```

```
%Reduction factor for oblique incident waves (gamma_beta) (for overtopping)
    gamma_beta2 = 1 - 0.0033*abs(beta);

%Wave steepness
    Tm = Tp / alfa;
    s0 = 2*pi*Hm0 / (g*Tm^2);

%Determenation of the surface area of the cross-section of the current dike
    h_hc = Crest_level_h - Ground_level;
    h_hb = Berm_level_h - Ground_level;
    O_h = 0.5*mh1*h_hc^2 + h_hc*B_h_crest + 0.5*mh3*(h_hc-h_hb)^2 + h_hb*(B_h +
mh3*(h_hc-h_hb)) + 0.5*mh2*h_hb^2;

%Width current dike
    B_hd = h_hc*mh1 + B_h_crest + mh3*(h_hc-h_hb) + B_h + mh2*h_hb;

%Berm level
    p=1;
for k = (Ztk-2):0.1:(Ztk+2);
    p=p+1;
    Berm_level(p) = k;

    %Berm width
    i=1;
    for j = 0:0.5:20
    i = i+1;
    B(i) = j;

    %Computation Wave Run-up
    n=2;
    Rup(n-1) = 999;
    Rup(n) = 0;
    while (Rup(n-1)-Rup(n)<-0.02) || (Rup(n-1)-Rup(n)>0.02 )

        %Breaker parameter (xi0)
        Lslope = B(i) + 1.5*Hm0*m1 + Rup(n)*m2;
        xi0 = ((1.5*Hm0+Rup(n))/(Lslope - B(i)))/sqrt(s0);

        %Reduction factor for a berm (gamma_b)
        Lb = B(i) + Hm0*m1 + Hm0*m2;
        dh = Ztk-Berm_level(p);
        if dh<0
            x=Rup(n);
        else
            x=2*Hm0;
        end
        gamma_b = 1 - B(i)/Lb * (0.5 + 0.5*cos(pi*abs(dh)/x));
        if gamma_b > 0.6
            gamma_b = gamma_b;
        else
            gamma_b = 0.6;
        end

        n = n+1;

        %Wave Run-up
        if 0.5 < gamma_b * xi0 < 1.8
            Rup(n) = Hm0 * 1.75 * gamma_beta * gamma_b * gamma_f * xi0;
```

```
        ELSEIF 1.8 < GAMMA_B * XI0 < 10
            RUP(N) = HM0 * GAMMA_BETA * GAMMA_F * (4.3 - 1.6/SQRT(XI0));
        ELSE
            DISP('ERROR, NON ALLOWED VALUE')
        END
    END

    RUP(P,I) = RUP(N);

    %DETERMINATION OF DIKE CREST LEVEL
    ZDP(P,I) = ZTK + RUP(P,I) + A; %[M]

    ZTK_M(P,I)=ZTK; %[M +M.S.L.]


    IF BERM_LEVEL(P)-GROUND_LEVEL<0;
        BG = 0;
    ELSE
        BG = BERM_LEVEL(P)-GROUND_LEVEL;
    END

    IF GROUND_LEVEL > BERM_LEVEL;
        ZB = ZDP(P,I)-GROUND_LEVEL;
    ELSE
        ZB = ZDP(P,I)-BERM_LEVEL(P);
    END

    %DETERMENATION OF THE AMOUNT OF NEEDED SOIL
    O(P,I) = 0.5*M1*BG^2 + BG*(B(I)+M2*ZB) + 0.5*M2*ZB^2 + B_CREST*(ZDP(P,I)-
GROUND_LEVEL) + 0.5*M3*(ZDP(P,I)-GROUND_LEVEL)^2 - O_H; %[M3/M]

    %DETERMENATION OF THE AMOUNT OF NEEDED REVETMENT
    REV(P,I) = SQRT(BG^2+(M1*BG)^2) + B(I) + SQRT(ZB^2+(M2*ZB)^2); %[M2/M]

    %WIDTH OF THE DIKE
    B_D(P,I) = M1*BG + B(I) + M2*(ZB-BG) + B_CREST + M3*ZB; %[M]

    %FIRST APROXIMATION OF THE COSTS [VND/M]
    COST(P,I) = P1*O(P,I) + P2*REV(P,I) + P3*(B_D(P,I)-B_HD);
    END
END
RUP(1,1)=0;

%% FIGURES

FIGURE(1)
SURF(B,BERM_LEVEL,ZDP)
    TITLE('EFFECT OF OUTER BERM')
    XLABEL('BERM WIDTH [M]')
    YLABEL('BERM LEVEL [M +M.S.L.]')
    ZLABEL('CREST LEVEL [M]')
    AXIS([1 20 ZTK-1.9 ZTK+2 0 10])

HOLD OFF

FIGURE(2)
SURF(B,BERM_LEVEL,RUP)
HOLD ON
```

```
SURF(B,BERM_LEVEL,ZDP)
HOLD ON
SURF(B,BERM_LEVEL,ZTK_M)
COLORMAP JET
    TITLE('EFFECT OF OUTER BERM')
    XLABEL('BERM WIDTH [M]')
    YLABEL('BERM LEVEL [M +M.S.L.]')
    ZLABEL('[M]')
    AXIS([1 20 ZTK-1.9 ZTK+2 0 10])
    LEGEND('WAVE RUN-UP','CREST LEVEL','DESIGN W.L.')

HOLD OFF

FIGURE(3)
SUBPLOT(1,2,1)
SURF(B,BERM_LEVEL,O)
    TITLE('AMOUNT OF SOIL')
    XLABEL('BERM WIDTH [M]')
    YLABEL('BERM LEVEL [M +M.S.L.]')
    ZLABEL('AMOUNT OF SOIL [M3/M]')
    AXIS([1 20 ZTK-1.9 ZTK+2 0 500])
SUBPLOT(1,2,2)
SURF(B,BERM_LEVEL,REV)
    TITLE('AMOUNT OF REVETMENT')
    XLABEL('BERM WIDTH [M]')
    YLABEL('BERM LEVEL [M +M.S.L.]')
    ZLABEL('AMOUNT OF REVETMENT[M2/M]')
    AXIS([1 20 ZTK-1.9 ZTK+2 0 100])

HOLD OFF

FIGURE(4)
SURF(B,BERM_LEVEL,COST)
    TITLE('COST [VND/M]')
    XLABEL('BERM WIDTH [M]')
    YLABEL('BERM LEVEL [M +M.S.L.]')
    ZLABEL('COST [VND/M]')
    AXIS([1 20 ZTK-1.9 ZTK+2 0 100000000])
```

15.2 Optimization slope without overtopping

```
CLEAR ALL; CLOSE ALL; CLC;

%% EFFECT INCLINATION OF THE OUTER SLOPE (WITHOUT OVERTOPPING).
    %WHAT IS THE EFFECT OF THE INCLINATION OF THE OUTER SLOPE
    %DESIGN IN COMBINATION WITH CALCULATION OF THE OUTER BERM, DOC.:
    %BERM.M
    %ITTERATION: BERMWITHD & BERMLEVEL --> INCLINATIONS --> CRESTLEVEL -->
    %OVERTOPPING --> BERMWITHD & BERMLEVEL --> ETC.

%% PARAMETERS

%ITERATIFF PARAMETERS
%RESULTS OUT OF BERM.M
B = 10          ;% [M]        BERM WIDTH
BERM_LEVEL = 2.9  ;% [M +M.S.L]  BERM LEVEL (REFFERENCE: THE MIDDEL OF THE BERM)

%PARAMETERS
```

```
B_CREST = 4       ;% [M]        WIDTH OF THE CREST (SEE "TECHNICAL STANDARDS IN SEA
DIKE DESIGN", TABLE 5.4, DIKE GRADE IV)
ZTK = 3.17        ;% [M +M.S.L.] DESIGN WATER LEVEL
A = 0.3           ;% [M]        SAFETY HEIGHT INCREMENT (SEE "TECHNICAL STANDARDS IN
SEA DIKE DESIGN", TABLE 5.2, DIKE GRADE IV)
M3 = 3            ;% [-]        INCLINATION OF THE INNER SLOPE (1:X)
HM0 = 1.70        ;% [M]        DESIGN WAVE HEIGHT AT THE TOE
TP = 11.16        ;% [S]        PEAK PERIOD
ALFA = 1.15       ;% [-]        1.1 ~ 1.2
BETA = 0          ;% [DEGREE]   ANGLE OF INCIDENT WAVES
GAMMA_F = 0.9     ;% [-]        REDUCTION FACTOR ROUGHNESS ELEMENTS (SEE "TECHNICAL
STANDARDS IN SEA DIKE DESIGN", APPENDIX A, TABLE 1)
G = 9.81          ;% [M/S2]

%CURRENT SITUATION
GROUND_LEVEL = 0  ;% [M +M.S.L.] GROUND LEVEL
%CURRENT DIKE
B_H_CREST = 7.5       ;% [M]        WIDTH OF THE CREST (CURRENT DIKE)
CREST_LEVEL_H = 4.80  ;% [M +M.S.L.] CREST LEVEL (CURRENT DIKE)
B_H = 5.0             ;% [M]        WIDTH OF THE INNER BERM (CURRENT DIKE)
BERM_LEVEL_H = 3.00   ;% [M +M.S.L.] BERM LEVEL (CURRENT DIKE)
MH1 = 3.5             ;% [-]        INCLINATION OF THE OUTER SLOPE (CURRENT
DIKE)(1:X)
MH2 = 2               ;% [-]        INCLINATION OF THE INNER SLOPE, BENEATH THE BERM
(CURRENT DIKE)(1:X)
MH3 = 2               ;% [-]        INCLINATION OF THE INNER SLOPE, ABOVE THE BERM
(CURRENT DIKE)(1:X)

%COSTS
P1 = 130000        ;% [VND/M^3]  PRICE OF SAND
P2 = 570000        ;% [VND/M^2]  PRICE OF BLOCK REVETMENT
P3 = 140000        ;% [VND/M^2]  PRICE TO BUY AGRICULTURE LAND

%% CALCULATIONS: EFFECT OF THE INCLINATION OF THE OUTER SLOPE

%REDUCTION FACTOR FOR OBLIQUE INCIDENT WAVES (GAMMA_BETA) (FOR WAVE RUN-UP)
    GAMMA_BETA = 1 - 0.0022*ABS(BETA);
%REDUCTION FACTOR FOR OBLIQUE INCIDENT WAVES (GAMMA_BETA) (FOR OVERTOPPING)
    GAMMA_BETA2 = 1 - 0.0033*ABS(BETA);

%WAVE STEEPNESS
    TM = TP / ALFA;
    S0 = 2*PI*HM0 / (G*TM^2);

%DETERMENATION OF THE SURFACE AREA OF THE CROSS-SECTION OF THE CURRENT DIKE
    H_HC = CREST_LEVEL_H - GROUND_LEVEL;
    H_HB = BERM_LEVEL_H - GROUND_LEVEL;
    O_H = 0.5*MH1*H_HC^2 + H_HC*B_H_CREST + 0.5*MH3*(H_HC-H_HB)^2 + H_HB*(B_H +
MH3*(H_HC-H_HB)) + 0.5*MH2*H_HB^2;

%WIDTH CURRENT DIKE
    B_HD = H_HC*MH1 + B_H_CREST + MH3*(H_HC-H_HB) + B_H + MH2*H_HB;

%INCLINATION OF THE SLOPE, BENEATH THE OUTER BERM (1:X)
    P=1;
FOR K = 1:0.5:8;
    P=P+1;
    M1(P) = K;
```

```
%Inclination of the slope, above the outer berm (1:x)
i=1;
for j = 1:0.5:8;
i = i+1;
m2(i) = j;

%Computation Wave Run-up
n=2;
Rup(n-1) = 999;
Rup(n) = 0;
while (Rup(n-1)-Rup(n)<-0.02) || (Rup(n-1)-Rup(n)>0.02)

    %Breaker parameter (xi0)
    Lslope = B + 1.5*Hm0*m1(p) + Rup(n)*m2(i);
    xi0 = ((1.5*Hm0+Rup(n))/(Lslope - B))/sqrt(s0);

    %Reduction factor for a berm (gamma_b)
    Lb = B + Hm0*m1(p) + Hm0*m2(i);
    dh = Ztk-Berm_level;
    if dh<0
        x=Rup(n);
    else
        x=2*Hm0;
    end
    gamma_b = 1 - B/Lb * (0.5 + 0.5*cos(pi*abs(dh)/x));
    if gamma_b > 0.6
        gamma_b = gamma_b;
    else
        gamma_b = 0.6;
    end

    n = n+1;

    %Wave Run-up
    if 0.5 < gamma_b * xi0 < 1.8
        Rup(n) = Hm0 * 1.75 * gamma_beta * gamma_b * gamma_f * xi0;
    elseif 1.8 < gamma_b * xi0 < 10
        Rup(n) = Hm0 * gamma_beta * gamma_f * (4.3 - 1.6/sqrt(xi0));
    else
        disp('ERROR, Non allowed value')
    end
end

Rup(p,i) = Rup(n);

%Determination of dike crest level
Zdp(p,i) = Ztk + Rup(p,i) + a; %[m]

Ztk_m(p,i)=Ztk; %[m +M.S.L.]

if Berm_level-Ground_level<0;
    BG = 0;
else
    BG = Berm_level-Ground_level;
end

if Ground_level > Berm_level;
```

```
        zb = Zdp(p,i)-Ground_level;
    else
        zb = Zdp(p,i)-Berm_level;
    end

    %Determenation of the amount of needed soil
    O(p,i)   =   0.5*m1(p)*bg^2   +   bg*(b+m2(i)*zb)   +   0.5*m2(i)*zb^2   +
B_crest*(Zdp(p,i)-Ground_level)   +   0.5*m3*(Zdp(p,i)-Ground_level)^2   -   O_h;
%[m3/m]

    %Determenation of the amount of needed revetment
    Rev(p,i) = sqrt(bg^2+(m1(p)*bg)^2) + b + sqrt(zb^2+(m2(i)*zb)^2); %[m2/m]

    %Width of the dike
    B_d(p,i) = m1(p)*bg + b + m2(i)*(zb-bg) + B_crest + m3*zb;

    %First aproximation of the costs [VND/m]
    Cost(p,i) = P1*O(p,i) + P2*Rev(p,i) + P3*(B_d(p,i)-B_hd);

    end
end
Rup(1,1)=0;

%% Figures

figure(1)
surf(m1,m2,Zdp)
    title('Effect of the inclination of the slope')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Crest level [m]')
    axis([1 8 1 8 0 10])

hold off

figure(2)
surf(m1,m2,Rup)
hold on
surf(m1,m2,Zdp)
hold on
surf(m1,m2,Ztk_m)
colormap jet
    title('Effect of the inclination of the slope')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('[m]')
    axis([1 8 1 8 0 10])
    legend('Wave Run-up','Crest level','Design W.L.')

hold off

figure(3)
subplot(1,2,1)
surf(m1,m2,O)
    title('Amount of soil')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Amount of soil [m3/m]')
```

```
    axis([1 8 1 8 0 500])
subplot(1,2,2)
surf(m1,m2,Rev)
    title('Amount of revetment')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Amount of revetment[m2/m]')
    axis([1 8 1 8 0 100])

hold off

figure(4)
surf(m1,m2,Cost)
    title('Cost [VND/m]')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Cost[VND/m]')
    axis([1 8 1 8 0 100000000])
```

15.3 Optimazation berm with overtopping

```
clear all; close all; clc;

%% Effect of the outer berm (with overtopping).
    %What is the effect of a berm for different width and levels
    %Design in combination with calculation of the inclination, doc.:
    %inclination.
    %Itteration: bermwithd & bermlevel --> inclinations --> crestlevel -->
    %overtopping --> bermwithd & bermlevel --> etc.

%% Parameters

%Iteratiff parameters
%Results out of Inclination.m
m1 = 8          ;% [-]        Inclination of the outer slope, beneath the berm (1:x)
m2 = 3.5        ;% [-]        Inclination of the outer slope, above the berm (1:x)

%Parameters
q = 1           ;% [l/s/m]    Allowable overtopping discharge
B_crest = 4     ;% [m]        Width of the crest (see "Technical standards in sea dike design", table 5.4, dike grade IV)
Ztk = 3.17      ;% [m +M.S.L.] Design water level
a = 0.3         ;% [m]        Safety height increment (see "Technical standards in sea dike design", table 5.2, dike grade IV)
m3 = 3          ;% [-]        Inclination of the inner slope (1:x)
Hm0 = 1.70      ;% [m]        Design Wave height at the toe
Tp = 11.16      ;% [s]        Peak period
alfa = 1.15     ;% [-]        1.1 ~ 1.2
beta = 0        ;% [degree]   Angle of incident waves
gamma_f = 0.55  ;% [-]        Reduction factor roughness elements (see "Technical Standards in Sea dike design", Appendix A, Table 1)
g = 9.81        ;% [m/s2]

%Current situation
Ground_level = 0  ;% [m +M.S.L.] Ground level
%Current dike
B_h_crest = 7.5      ;% [m]        Width of the crest (current dike)
Crest_level_h = 4.80 ;% [m +M.S.L.] Crest level (current dike)
```

```
B_H = 5.0               ;% [M]        WIDTH OF THE INNER BERM (CURRENT DIKE)
BERM_LEVEL_H = 3.00     ;% [M +M.S.L.] BERM LEVEL (CURRENT DIKE)
MH1 = 3.5               ;% [-]        INCLINATION OF THE OUTER SLOPE (CURRENT
DIKE)(1:X)
MH2 = 2                 ;% [-]        INCLINATION OF THE INNER SLOPE, BENEATH THE BERM
(CURRENT DIKE)(1:X)
MH3 = 2                 ;% [-]        INCLINATION OF THE INNER SLOPE, ABOVE THE BERM
(CURRENT DIKE)(1:X)

%COSTS
P1 = 130000          ;% [VND/M^3]   PRICE OF SAND
P2 = 540000          ;% [VND/M^2]   PRICE OF BLOCK REVETMENT
P3 = 140000          ;% [VND/M^2]   PRICE TO BUY AGRICULTURE LAND

%% CALCULATIONS: EFFECT OF THE OUTER BERM

%REDUCTION FACTOR FOR OBLIQUE INCIDENT WAVES (GAMMA_BETA) (FOR WAVE RUN-UP)
    GAMMA_BETA = 1 - 0.0022*ABS(BETA);
%REDUCTION FACTOR FOR OBLIQUE INCIDENT WAVES (GAMMA_BETA) (FOR OVERTOPPING)
    GAMMA_BETA2 = 1 - 0.0033*ABS(BETA);

%WAVE STEEPNESS
    TM = TP / ALFA;
    S0 = 2*PI*HM0 / (G*TM^2);

%DETERMENATION OF THE SURFACE AREA OF THE CROSS-SECTION OF THE CURRENT DIKE
    H_HC = CREST_LEVEL_H - GROUND_LEVEL;
    H_HB = BERM_LEVEL_H - GROUND_LEVEL;
    O_H = 0.5*MH1*H_HC^2 + H_HC*B_H_CREST + 0.5*MH3*(H_HC-H_HB)^2 + H_HB*(B_H +
MH3*(H_HC-H_HB)) + 0.5*MH2*H_HB^2;

%WIDTH CURRENT DIKE
    B_HD = H_HC*MH1 + B_H_CREST + MH3*(H_HC-H_HB) + B_H + MH2*H_HB;

Q=Q/1000;

%BERM LEVEL
    P=1;
FOR K = (ZTK-2):0.1:(ZTK+2);
    P=P+1;
    BERM_LEVEL(P) = K;

    %BERM WIDTH
    I=1;
    FOR J = 0:0.5:20
    I = I+1;
    B(I) = J;

    %COMPUTATION WAVE RUN-UP
    N=2;
    RUP(N-1) = 999;
    RUP(N) = 0;
    WHILE (RUP(N-1)-RUP(N)<-0.02) || (RUP(N-1)-RUP(N)>0.02 )

        %BREAKER PARAMETER (XI0)
        LSLOPE = B(I) + 1.5*HM0*M1 + RUP(N)*M2;
        XI0 = ((1.5*HM0+RUP(N))/(LSLOPE - B(I)))/SQRT(S0);
```

```
        %REDUCTION FACTOR FOR A BERM (GAMMA_B)
        LB = B(I) + HM0*M1 + HM0*M2;
        DH = ZTK-BERM_LEVEL(P);
        IF DH<0
            X=RUP(N);
        ELSE
            X=2*HM0;
        END
        GAMMA_B = 1 - B(I)/LB * (0.5 + 0.5*COS(PI*ABS(DH)/X));
        IF GAMMA_B > 0.6
            GAMMA_B = GAMMA_B;
        ELSE
            GAMMA_B = 0.6;
        END

        N = N+1;

        %WAVE RUN-UP (WITH OVERTOPPING)
        IF GAMMA_B * XI0 <= 2
        RUP(N)    =    -    HM0*XI0*GAMMA_B*GAMMA_F*GAMMA_BETA2    /    4.3    *
LOG(Q/(0.067*GAMMA_B*XI0)*SQRT((M1+M2)/(2*G*HM0^3)));
        ELSEIF GAMMA_B * XI0 > 7
        RUP(N)    =    -    HM0*GAMMA_F*GAMMA_BETA2*(0.33+0.022*XI0)    *
LOG(Q/(0.21*SQRT(G*HM0^3)));
        ELSE
        RUP(N) = - HM0*GAMMA_F*GAMMA_BETA2/2.3 * LOG(Q/(0.2*SQRT(G*HM0^3)));
        END
    END

    RUP(P,I) = RUP(N);

    %DETERMINATION OF DIKE CREST LEVEL
    ZDP(P,I) = ZTK + RUP(P,I) + A; %[M]

    ZTK_M(P,I)=ZTK; %[M +M.S.L.]

    IF BERM_LEVEL(P)-GROUND_LEVEL<0;
        BG = 0;
    ELSE
        BG = BERM_LEVEL(P)-GROUND_LEVEL;
    END

    IF GROUND_LEVEL > BERM_LEVEL;
        ZB = ZDP(P,I)-GROUND_LEVEL;
    ELSE
        ZB = ZDP(P,I)-BERM_LEVEL(P);
    END

    %DETERMENATION OF THE AMOUNT OF NEEDED SOIL
    O(P,I) = 0.5*M1*BG^2 + BG*(B(I)+M2*ZB) + 0.5*M2*ZB^2 + B_CREST*(ZDP(P,I)-
GROUND_LEVEL) + 0.5*M3*(ZDP(P,I)-GROUND_LEVEL)^2 - O_H; %[M3/M]

    %DETERMENATION OF THE AMOUNT OF NEEDED REVETMENT
    REV(P,I) = SQRT(BG^2+(M1*BG)^2) + B(I) + SQRT(ZB^2+(M2*ZB)^2); %[M2/M]

    %WIDTH OF THE DIKE
    B_D(P,I) = M1*BG + B(I) + M2*(ZB-BG) + B_CREST + M3*ZB; %[M]
```

```
    %First aproximation of the costs [VND/m]
    Cost(p,i) = P1*O(p,i) + P2*Rev(p,i) + P3*(B_d(p,i)-B_hd);
    end
end
Rup(1,1)=0;

%% Figures

figure(1)
surf(B,Berm_level,Zdp)
    title('Effect of outer berm')
    xlabel('Berm width [m]')
    ylabel('Berm level [m +M.S.L.]')
    zlabel('Crest level [m]')
    axis([1 20 Ztk-1.9 Ztk+2 0 10])

hold off

figure(2)
subplot(1,2,1)
surf(B,Berm_level,O)
    title('Amount of soil')
    xlabel('Berm width [m]')
    ylabel('Berm level [m +M.S.L.]')
    zlabel('Amount of soil [m3/m]')
    axis([1 20 Ztk-1.9 Ztk+2 0 500])
subplot(1,2,2)
surf(B,Berm_level,Rev)
    title('Amount of revetment')
    xlabel('Berm width [m]')
    ylabel('Berm level [m +M.S.L.]')
    zlabel('Amount of revetment[m2/m]')
    axis([1 20 Ztk-1.9 Ztk+2 0 100])

hold off

figure(3)
surf(B,Berm_level,Cost)
    title('Cost [VND/m]')
    xlabel('Berm width [m]')
    ylabel('Berm level [m +M.S.L.]')
    zlabel('Cost [VND/m]')
    axis([1 20 Ztk-1.9 Ztk+2 0 100000000])
```

15.4 Optimazation slope with overtopping

```
clear all; close all; clc;

%% Effect inclination of the outer slope (with overtopping).
    %What is the effect of the inclination of the outer slope
    %Design in combination with calculation of the outer berm, doc.:
    %Berm.m
    %Itteration: bermwithd & bermlevel --> inclinations --> crestlevel -->
    %overtopping --> bermwithd & bermlevel --> etc.

%% Parameters

%Iteratiff parameters
%Results out of Berm.m
```

```
B = 0           ;% [m]        Berm width
Berm_level = 1.2  ;% [m +M.S.L]  Berm level (refference: the middel of the berm)

%Parameters
q = 1             ;% [l/s/m]     Allowable overtopping discharge
B_crest = 4        ;% [m]          Width of the crest (see "Technical standards in sea
dike design", table 5.4, dike grade IV)
Ztk = 3.17         ;% [m +M.S.L.] Design water level
a = 0.3            ;% [m]          Safety height increment (see "Technical standards in
sea dike design", table 5.2, dike grade IV)
m3 = 3            ;% [-]          Inclination of the inner slope (1:x)
Hm0 = 1.70         ;% [m]          Design Wave height at the toe
Tp = 11.16         ;% [s]          Peak period
alfa = 1.15        ;% [-]          1.1 ~ 1.2
beta = 0          ;% [degree]    Angle of incident waves
gamma_f = 0.55      ;% [-]          Reduction factor roughness elements (see "Technical
Standards in Sea dike design", Appendix A, Table 1)
g = 9.81           ;% [m/s2]

%Current situation
Ground_level = 0  ;% [m +M.S.L.] Ground level
%Current dike
B_H_crest = 7.5       ;% [m]          Width of the crest (current dike)
Crest_level_H = 4.80  ;% [m +M.S.L.] Crest level (current dike)
B_H = 5.0             ;% [m]          Width of the inner berm (current dike)
Berm_level_H = 3.00   ;% [m +M.S.L.] Berm level (current dike)
mH1 = 3.5                ;% [-]           Inclination of the outer slope (current
dike)(1:x)
mH2 = 2               ;% [-]          Inclination of the inner slope, beneath the berm
(current dike)(1:x)
mH3 = 2                ;% [-]           Inclination of the inner slope, above the berm
(current dike)(1:x)

%Costs
P1 = 130000          ;% [VND/m^3]   Price of sand
P2 = 540000          ;% [VND/m^2]   Price of block revetment
P3 = 140000          ;% [VND/m^2]   Price to buy agriculture land

%% Calculations: Effect of the inclination of the outer slope

%Reduction factor for oblique incident waves (gamma_beta) (for wave run-up)
    gamma_beta = 1 - 0.0022*abs(beta);
%Reduction factor for oblique incident waves (gamma_beta) (for overtopping)
    gamma_beta2 = 1 - 0.0033*abs(beta);

%Wave steepness
    Tm = Tp / alfa;
    s0 = 2*pi*Hm0 / (g*Tm^2);

%Determenation of the surface area of the cross-section of the current dike
    H_HC = Crest_level_H - Ground_level;
    H_HB = Berm_level_H - Ground_level;
    O_H = 0.5*mH1*H_HC^2 + H_HC*B_H_crest + 0.5*mH3*(H_HC-H_HB)^2 + H_HB*(B_H +
mH3*(H_HC-H_HB)) + 0.5*mH2*H_HB^2;

%Width current dike
    B_HD = H_HC*mH1 + B_H_crest + mH3*(H_HC-H_HB) + B_H + mH2*H_HB;
```

```
Q=Q/1000;

%INCLINATION OF THE SLOPE, BENEATH THE OUTER BERM (1:X)
    P=1;
FOR K = 1:0.5:8;
    P=P+1;
    M1(P) = K;

    %INCLINATION OF THE SLOPE, ABOVE THE OUTER BERM (1:X)
    I=1;
    FOR J = 1:0.5:8;
    I = I+1;
    M2(I) = J;

    %COMPUTATION WAVE RUN-UP
    N=2;
    RUP(N-1) = 999;
    RUP(N) = 0;
    WHILE (RUP(N-1)-RUP(N)<-0.02) || (RUP(N-1)-RUP(N)>0.02)

        %BREAKER PARAMETER (XI0)
        LSLOPE = B + 1.5*HM0*M1(P) + RUP(N)*M2(I);
        XI0 = ((1.5*HM0+RUP(N))/(LSLOPE - B))/SQRT(S0);

        %REDUCTION FACTOR FOR A BERM (GAMMA_B)
        LB = B + HM0*M1(P) + HM0*M2(I);
        DH = ZTK-BERM_LEVEL;
        IF DH<0
            X=RUP(N);
        ELSE
            X=2*HM0;
        END
        GAMMA_B = 1 - B/LB * (0.5 + 0.5*COS(PI*ABS(DH)/X));
        IF GAMMA_B > 0.6
            GAMMA_B = GAMMA_B;
        ELSE
            GAMMA_B = 0.6;
        END

        N = N+1;

        WAVE RUN-UP (WITH OVERTOPPING)
        IF GAMMA_B * XI0 <= 2
        RUP(N)    =    -    HM0*XI0*GAMMA_B*GAMMA_F*GAMMA_BETA2    /    4.3    *
LOG(Q/(0.067*GAMMA_B*XI0)*SQRT((M1(P)+M2(I))/(2*G*HM0^3)));
        ELSEIF GAMMA_B * XI0 > 7
        RUP(N)    =    -    HM0*GAMMA_F*GAMMA_BETA2*(0.33+0.022*XI0)    *
LOG(Q/(0.21*SQRT(G*HM0^3)));
        ELSE
        RUP(N) = - HM0*GAMMA_F*GAMMA_BETA2/2.3 * LOG(Q/(0.2*SQRT(G*HM0^3)));
        END
    END

    RUP(P,I) = RUP(N);

    %DETERMINATION OF DIKE CREST LEVEL
    ZDP(P,I) = ZTK + RUP(P,I) + A; %[M]
```

```
    Ztk_m(p,i)=Ztk; %[m +M.S.L.]

    if Berm_level-Ground_level<0;
        BG = 0;
    else
        BG = Berm_level-Ground_level;
    end

    if Ground_level > Berm_level;
        ZB = Zdp(p,i)-Ground_level;
    else
        ZB = Zdp(p,i)-Berm_level;
    end

    %Determenation of the amount of needed soil
    O(p,i) = 0.5*m1(p)*BG^2 + BG*(B+m2(i)*ZB) + 0.5*m2(i)*ZB^2 +
B_crest*(Zdp(p,i)-Ground_level) + 0.5*m3*(Zdp(p,i)-Ground_level)^2 - O_h;
%[m3/m]

    %Determenation of the amount of needed revetment
    Rev(p,i) = sqrt(BG^2+(m1(p)*BG)^2) + B + sqrt(ZB^2+(m2(i)*ZB)^2); %[m2/m]

    %Width of the dike
    B_d(p,i) = m1(p)*BG + B + m2(i)*(ZB-BG) + B_crest + m3*ZB;

    %First aproximation of the costs [VND/m]
    Cost(p,i) = P1*O(p,i) + P2*Rev(p,i) + P3*(B_d(p,i)-B_hd);

    end
end
Rup(1,1)=0;

%% Figures

figure(1)
surf(m1,m2,Zdp)
    title('Effect of the inclination of the slope')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Crest level [m]')
    axis([1 8 1 8 0 10])

hold off

figure(2)
subplot(1,2,1)
surf(m1,m2,O)
    title('Amount of soil')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Amount of soil [m3/m]')
    axis([1 8 1 8 0 500])
subplot(1,2,2)
surf(m1,m2,Rev)
    title('Amount of revetment')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Amount of revetment[m2/m]')
```

```
    axis([1 8 1 8 0 100])

hold off

figure(3)
surf(m1,m2,Cost)
    title('Cost [VND/m]')
    xlabel('Inclination of the slope, beneath the berm [1:x]')
    ylabel('Inclination of the slope, above the berm [1:x]')
    zlabel('Cost[VND/m]')
    axis([1 8 1 8 0 100000000])
```

16 APPENDIX Geotechnical

16.1 Basalton® Dike

	Settlement as a result of:		
Meter	the new soil	building time	Total
1	0,001	0	0,001
2	0,001	0	0,001
3	0,001	0	0,001
4	0,001	0	0,001
5	0,001	0	0,001
6	0,001	0	0,001
7	0,002	0	0,002
8	0,002	0	0,002
9	0,003	0	0,003
10	0,036	0	0,036
11	0,062	0	0,062
12	0,06	0	0,06
13	0,058	0	0,058
14	0,056	0	0,056
15	0,054	0	0,054
16	0,052	0	0,052
17	0,05	0	0,05
18	0,048	0,001	0,049
19	0,046	0,001	0,047
20	0,044	0,003	0,047
21	0,042	0,008	0,05
22	0,044	0,03	0,074
23	0,055	0,045	0,1
24	0,067	0,046	0,113
25	0,078	0,046	0,124
26	0,087	0,046	0,133
27	0,097	0,046	0,143
28	0,108	0,041	0,149
29	0,122	0,037	0,159
30	0,136	0,034	0,17
31	0,148	0,032	0,18
32	0,16	0,03	0,19
33	0,031	0,03	0,061
34	0,033	0,028	0,061
35	0,035	0,027	0,062
36	0,035	0,027	0,062
37	0,051	0,025	0,076
38	0,047	0,024	0,071
39	0,06	0,026	0,086
40	0,06	0,026	0,086
41	0,046	0,025	0,071
42	0,037	0,024	0,061
43	0,03	0,024	0,054
44	0,02	0,021	0,041
45	0,012	0,018	0,03
46	0,01	0,016	0,026
47	0,01	0,014	0,024
48	0,009	0,013	0,022
49	0,009	0,012	0,021
50	0,009	0,011	0,02
51	0,009	0,01	0,019
52	0,01	0,01	0,02
53	0,011	0,01	0,021
54	0,012	0,01	0,022
55	0,013	0,009	0,022
56	0,014	0,009	0,023
57	0,015	0,009	0,024
58	0,017	0,009	0,026
59	0,019	0,009	0,028
60	0,023	0,016	0,039
61	0,027	0,016	0,043
62	0,032	0,016	0,048
63	0,034	0,015	0,049
64	0,033	0,006	0,039
65	0,032	0,002	0,034
66	0,031	0,001	0,032
67	0,03	0,001	0,031
68	0,033	0,001	0,034
69	0,039	0	0,039
70	0,051	0	0,051

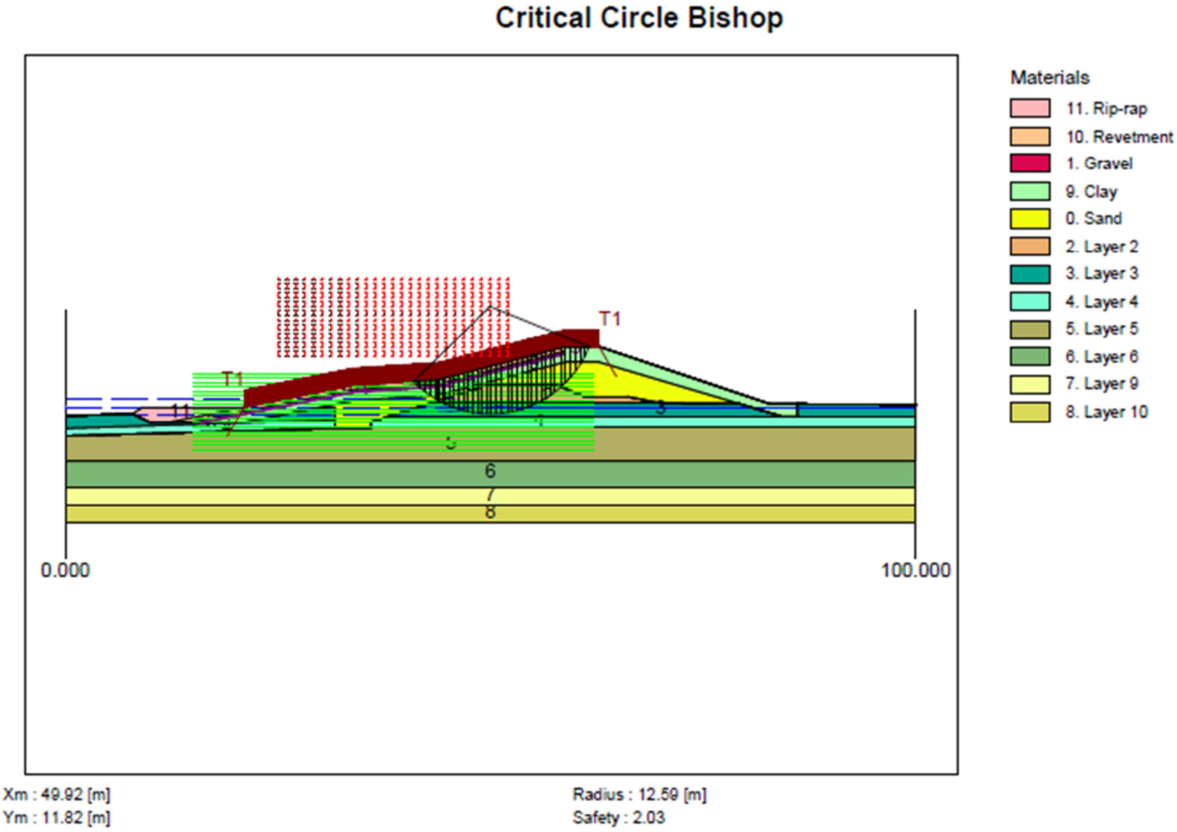
Meter	Settlement as a result of: the new soil	building time	Total
71	0,128	0	0,128
72	0,164	0	0,164
73	0,189	0	0,189
74	0,172	0	0,172
75	0,158	0	0,158
76	0,144	0	0,144
77	0,13	0	0,13
78	0,115	0	0,115
79	0,1	0	0,1
80	0,085	0	0,085
81	0,069	0	0,069
82	0,051	0	0,051
83	0,031	0	0,031
84	0,011	0	0,011
85	0,009	0	0,009
86	0,008	0	0,008
87	0,006	0	0,006
88	0,005	0	0,005
89	0,004	0	0,004
90	0,004	0	0,004
91	0,003	0	0,003
92	0,003	0	0,003
93	0,002	0	0,002
94	0,002	0	0,002
95	0,002	0	0,002
96	0,001	0	0,001
97	0,001	0	0,001
98	0,001	0	0,001
99	0,001	0	0,001
100	0,001	0	0,001

16.2 Rip-rap dike

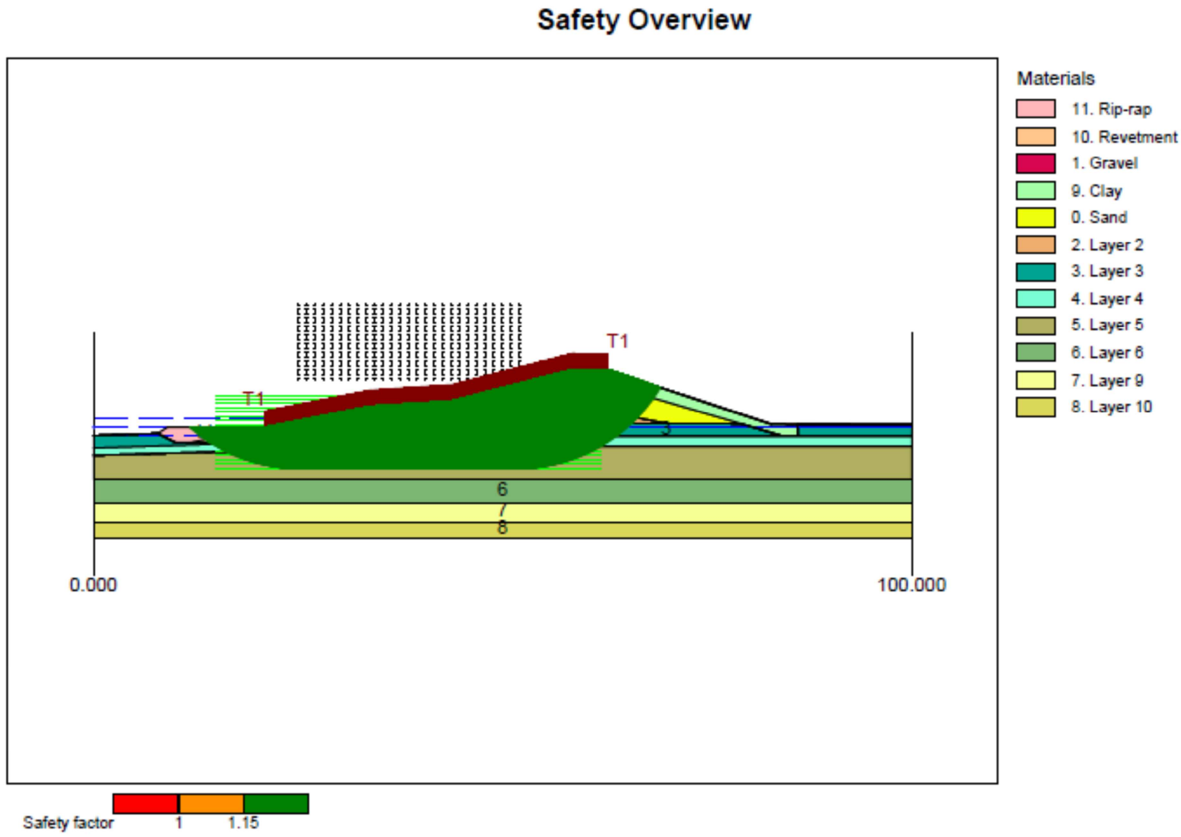
Meter	Settlement as a result of: the new soil	Settlement as a result of: building time	Total	Meter	Settlement as a result of: the new soil	Settlement as a result of: building time	Total
1	0,002	0	0,002	**40**	0,093	0,014	0,107
2	0,002	0	0,002	**41**	0,078	0,014	0,092
3	0,003	0	0,003	**42**	0,072	0,013	0,085
4	0,003	0	0,003	**43**	0,067	0,013	0,08
5	0,003	0	0,003	**44**	0,049	0,011	0,06
6	0,046	0	0,046	**45**	0,025	0,009	0,034
7	0,072	0	0,072	**46**	0,021	0,008	0,029
8	0,069	0	0,069	**47**	0,019	0,008	0,027
9	0,068	0	0,068	**48**	0,017	0,007	0,024
10	0,065	0	0,065	**49**	0,016	0,007	0,023
11	0,063	0	0,063	**50**	0,014	0,008	0,022
12	0,062	0	0,062	**51**	0,013	0,008	0,021
13	0,06	0,001	0,061	**52**	0,012	0,009	0,021
14	0,058	0,001	0,059	**53**	0,012	0,017	0,029
15	0,056	0,002	0,058	**54**	0,011	0,017	0,028
16	0,055	0,004	0,059	**55**	0,011	0,017	0,028
17	0,058	0,007	0,065	**56**	0,01	0,016	0,026
18	0,064	0,008	0,072	**57**	0,009	0,009	0,018
19	0,07	0,009	0,079	**58**	0,009	0,002	0,011
20	0,076	0,01	0,086	**59**	0,008	0,001	0,009
21	0,082	0,011	0,093	**60**	0,008	0,001	0,009
22	0,087	0,012	0,099	**61**	0,008	0,001	0,009
23	0,092	0,013	0,105	**62**	0,008	0	0,008
24	0,097	0,014	0,111	**63**	0,008	0	0,008
25	0,102	0,014	0,116	**64**	0,006	0	0,006
26	0,109	0,015	0,124	**65**	0,005	0	0,005
27	0,118	0,015	0,133	**66**	0,005	0	0,005
28	0,127	0,017	0,144	**67**	0,004	0	0,004
29	0,135	0,017	0,152	**68**	0,004	0	0,004
30	0,144	0,017	0,161	**69**	0,003	0	0,003
31	0,152	0,017	0,169	**70**	0,003	0	0,003
32	0,16	0,017	0,177	**71**	0,003	0	0,003
33	0,035	0,015	0,05	**72**	0,003	0	0,003
34	0,038	0,015	0,053	**73**	0,003	0	0,003
35	0,041	0,015	0,056	**74**	0,002	0	0,002
36	0,044	0,015	0,059	**75**	0,002	0	0,002
37	0,064	0,013	0,077	**76**	0,002	0	0,002
38	0,062	0,012	0,074	**77**	0,001	0	0,001
39	0,082	0,013	0,095	**78**	0,001	0	0,001

Meter	*Settlement as a result of:* the new soil	building time	Total
79	0,001	0	0,001
80	0,001	0	0,001
81	0,001	0	0,001
82	0,001	0	0,001
83	0,001	0	0,001
84	0	0	0
85	0	0	0
86	0	0	0
87	0	0	0
88	0	0	0
89	0	0	0
90	0	0	0
91	0	0	0
92	0	0	0
93	0	0	0
94	0	0	0
95	0	0	0
96	0	0	0
97	0	0	0
98	0	0	0
99	0	0	0
100	0	0	0

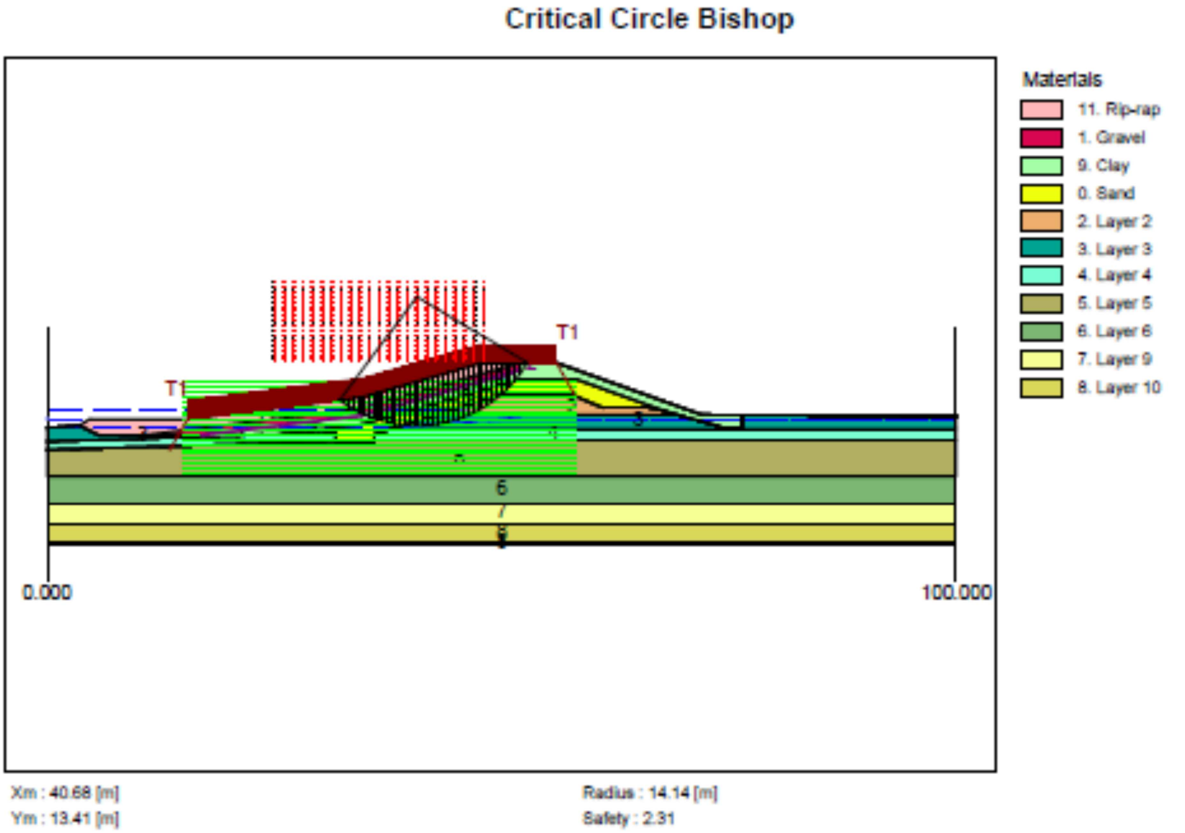
16.3 Stability of the geometry, Blocks, Bishop Circle



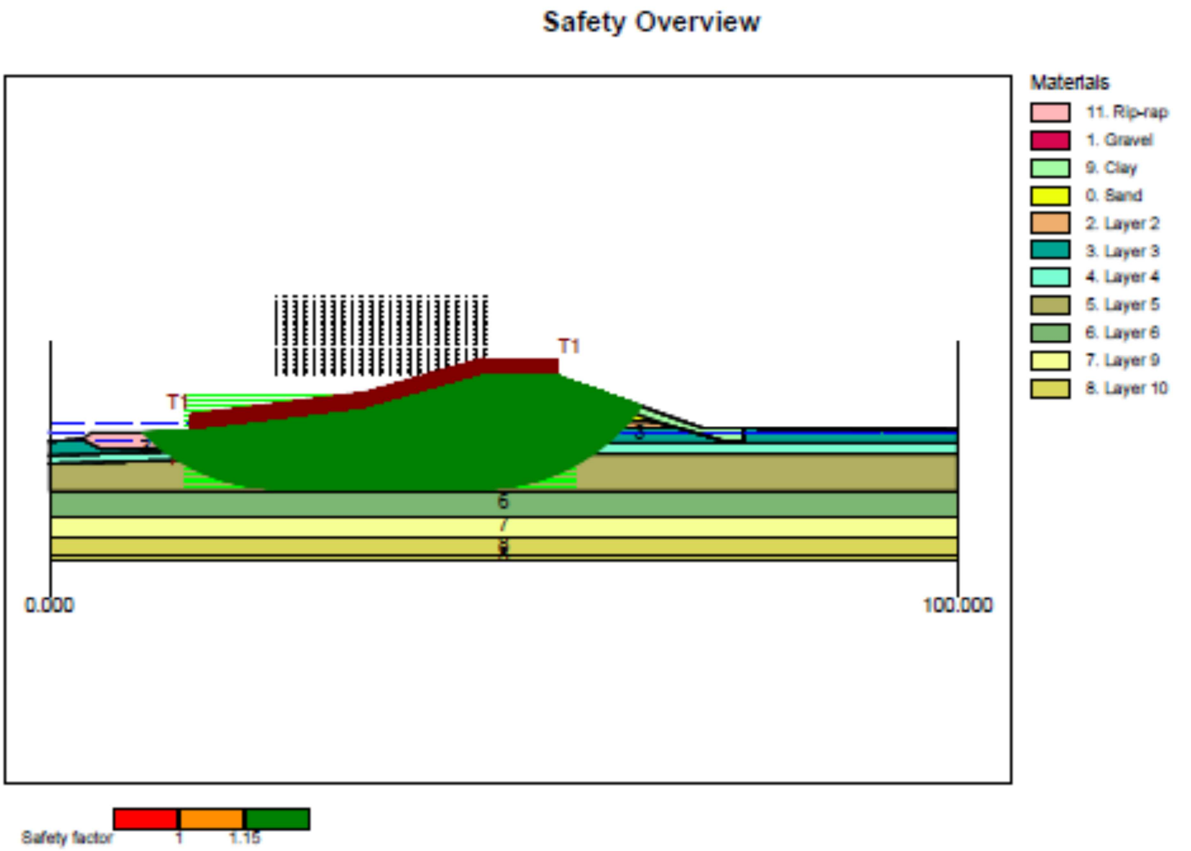
16.4 Stability of the geometry, Blocks, Safety Overview



16.5 Stability of the geometry, Rip rap, Bishop Circle



16.6 Stability of the geometry, Rip rap, Safety Overview



17 APPENDIX Logistics & Finance

17.1 Unit prices Hai Phong Area

	Unit	Price [vnd]	Price [euro]			Unit	Price [vnd]	Price [euro]
Material					**Labor**			
Anti-erosion admixture	kg	VND 10.500,00	€ 0,42		Manpower 2,7/7	man day	VND 74.224,00	€ 2,97
Burlap	units	VND 300,00	€ 0,01		Manpower 3,0/7	man day	VND 77.534,74	€ 3,10
Ligature wire	kg	VND 15.000,00	€ 0,60		Manpower 3,5/7	man day	VND 84.055,26	€ 3,36
Black sand	m3	VND 80.000,00	€ 3,20		Manpower 4,0/7	man day	VND 90.576,39	€ 3,62
Stone 1x2	m3	VND 270.000,00	€ 10,80		Manpower 4,5/7	man day	VND 98.268,00	€ 3,93
Stone 2x4	m3	VND 270.000,00	€ 10,80					
Stone 4x6	m3	VND 210.000,00	€ 8,40		**Construction machines**			
Quarry stone	m3	VND 185.000,00	€ 7,40		Water pumping truck 5m3	Shift	VND 714.417,00	€ 28,58
Stone powder	kg	VND 200,00	€ 0,01		Automatic pouring truck 7 tons	Shift	VND 1.042.213,00	€ 41,69
Fine sand ML=0,7-1,4	m3	VND 270.000,00	€ 10,80		Place vibrator 1 kw	Shift	VND 112.695,00	€ 4,51
Yellow sand	m3	VND 270.000,00	€ 10,80		Penetrating vibrator 1,5 kw	Shift	VND 115.221,00	€ 4,61
Pile	units	VND 15.000,00	€ 0,60		Mobile crane 10T	Shift	VND 1.678.298,00	€ 67,13
Macadam aggregate 0,07-50 mm (lower layer)	m3	VND 210.000,00	€ 8,40		Excavator 1,25 m3	Shift	VND 2.577.453,00	€ 103,10
Fire wood	kg	VND 700,00	€ 0,03		Self-propelled air tyre compactor 9T	Shift	VND 1.058.178,00	€ 42,33
Pointed wooden sheet pile D <= 8 cm L<= 4m	m3	VND 5.000,00	€ 0,20		Steel bending machine 5KW	Shift	VND 119.225,00	€ 4,77
Steel wire	kg	VND 18.000,00	€ 0,72		Welding machine 23 kw	Shift	VND 184.179,00	€ 7,37
Prop wood	m3	VND 2.428.571,00	€ 97,14		Compactor 10T	Shift	VND 766.146,00	€ 30,65
Chocking wood	m3	VND 2.428.571,00	€ 97,14		Rubber tyre compactor 16T	Shift	VND 944.129,00	€ 37,77
Wooden Elastic Joint	m3	VND 2.428.571,00	€ 97,14		Vibratory compactor 25T	Shift	VND 1.866.971,00	€ 74,68
Wooden plank	m3	VND 2.428.571,00	€ 97,14		Bulldozer 108CV	Shift	VND 1.380.497,00	€ 55,22
Oil paper	m2	VND 4.000,00	€ 0,16		Spreader 50-60 m3/h	Shift	VND 1.799.041,00	€ 71,96
Asphalt	kg	VND 10.000,00	€ 0,40		Grader 108CV	Shift	VND 2.488.373,00	€ 99,53
Bitume plastic no. 4	kg	VND 10.000,00	€ 0,40		Concrete mixer 250l	Shift	VND 162.026,00	€ 6,48
Water	litre	VND 8,00	€ 0,00		Concrete mixer 500l	Shift	VND 244.810,00	€ 9,79
Plasticizing admixture	kg	VND 9.308,00	€ 0,37		Mortar mixer80l	Shift	VND 124.166,00	€ 4,97
Soldering stick	kg	VND 19.695,00	€ 0,79		Lift machine 0,8T	Shift	VND 228.654,00	€ 9,15
Lining steel	kg	VND 15.000,00	€ 0,60		Lift 0,8T	Shift	VND 228.654,00	€ 9,15
Form steel	kg	VND 15.000,00	€ 0,60					
Plated steel	kg	VND 15.000,00	€ 0,60		**Cost of purchasing land**			
Round steel D<=10mm	kg	VND 15.000,00	€ 0,60		Agriculture land	m2	VND 100.000,00	€ 4,00
Round steel D<=18mm	kg	VND 15.000,00	€ 0,60		Urban land	m2	VND 500.000,00	€ 20,00
Canvas	m2	VND 30.000,00	€ 1,20					
Nilon	m2	VND 2.000,00	€ 0,08					
Cement PC30	kg	VND 910,00	€ 0,04					
Padded macadam	m3	VND 270.000,00	€ 10,80					
Geotextile	m2	VND 20.000,00	€ 0,80					
Bamboo pile (lenth 0,5m)	units	VND 15.000,00	€ 0,60					
Mountainous soil	m3	VND 130.000,00	€ 5,20					

17.2 Cost and production speed of making Basalton® on site

For making one m3 "Basalton® concrete element, with stone of 1x2 cm, M 300" on site the following materials, manpower and machines are used.

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**973.002**
Cement PC 30	*Kg*	*445,585*	*VND*	*910*	*VND*	*405.482*
Yellow sand	*m3*	*0,45066*	*VND*	*270.000*	*VND*	*121.678*
Stone 1x2 cm	*m3*	*0,877975*	*VND*	*270.000*	*VND*	*237.053*
Water	*Litre*	*176,61*	*VND*	*8*	*VND*	*1.413*
Plasticizing admixture	*Kg*	*22,27925*	*VND*	*9.308*	*VND*	*207.375*
Other material	*%*	*0,5*	*Unknown*			
Manpower					**VND**	**148.867**
Manpower 3,0/7	*Man day*	*1,92*	*VND*	*77.535*	*VND*	*148.867*
Construction machines					**VND**	**27.725**
Concrete mixer 250 L	*Shift*	*0,1*	*VND*	*162.026*	*VND*	*16.203*
Penetrating vibrator 1,5 kW	*Shift*	*0,1*	*VND*	*115.221*	*VND*	*11.522*
TOTAL per m3					**VND**	**1.149.593**

By adding the cost for material, manpower and construction machines, the total cost of producing 1 m3 of Basalton® on site can be found. This is 1.149.593,34 VND. In the report this number is rounded to: 1.150.000 VND.

Depending on the total volume of Basalton®, the total cost can be determined.

17.3 Removal of old revetment

For the removal of revetment, only manpower is used. The Vietnamese guideline indicates that to remove one m3 per day, 1,66 manpower is necessary. This means one worker can remove 0,60 m3 per day.

	Unit	Amount of Unit	Cost per Unit	Total Cost	
Material				**VND**	**-**
Manpower				**VND**	**139.532**
Manpower 3,5/7	*Man*	*1,66*	*VND 84.055,26*	*VND*	*139.532*
Construction machines				**VND**	**-**
TOTAL				**VND**	**139.532**

By dividing the total volume of the revetment that has to be removed, which is 1.850 m3, by the amount one worker can remove per day, results in the total amount of man days: 3045.

By multiplying this amount with the cost of the manpower per day results in the cost of removing the old revetment: 255.983.197,45 VND. In the report this is rounded to 256.000.000 VND.

17.4 Placing of new mountainous soil

The Vietnamese norm uses the following values to determine the time it takes to place the new soil. This results in the following costs:

	Unit	Amount unit	Cost per unit		Total cost	
Material					**VND**	**7.449.676.000**
Mountainous Soil	*m3*	*57305,2*	*VND*	*130.000*	*VND*	*7.449.676.000*
Manpower					**VND**	**77.310.702**
Manpower 3,0/7	*Man day*	*997,11048*	*VND*	*77.535*	*VND*	*77.310.702*
Construction machines					**VND**	**420.814.507**
Bulldozer 108CV	*Shift*	*120,34092*	*VND*	*1.380.497*	*VND*	*166.130.279*
Self-propelled air tyre compactor 9 t	*Shift*	*240,68184*	*VND*	*1.058.178*	*VND*	*254.684.228*
TOTAL					**VND**	**7.947.801.209**

The cost of the soil is calculated by multiplying the volume of soil necessary with the cost of the soil. 57.305 m^3 * 130.000 VND/m^3 = 7.450.000.000 VND.

17.5 Placing of geotextile

For the placement of 100 m^2 of geotextile, the following norm is used:

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**2.086.600**
Geotextile	*m2*	*104,00*	*VND*	*20.000*	*VND*	*2.080.000*
Bamboo Pile	*Units*	*0,44*	*VND*	*15.000*	*VND*	*6.600*
Manpower					**VND**	**119.358**
Manpower 3,5/7	*Man day*	*1,42*	*VND*	*84.055*	*VND*	*119.358*
Construction machines					**VND**	**-**
TOTAL					**VND**	**2.205.958**

For the total area of the dike, being 14.884 m^2 (rounded to 15.000 m^2 in the report), gives the total cost.

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**310.569.544**
Geotextile	*m2*	*15.479,36*	*VND*	*20.000*	*VND*	*309.587.200*
Bamboo Pile	*Units*	*65,49*	*VND*	*15.000*	*VND*	*982.344*
Manpower					**VND**	**17.765.331**
Manpower 3,5/7	*Man day*	*211,35*	*VND*	*84.055*	*VND*	*17.765.331*
Construction machines					**VND**	**-**
TOTAL					**VND**	**328.334.875**

In the report the total cost is rounded to 328.000.000 VND.

17.6 Cost and production speed of placing Basalton® by hand according to norm

The Vietnamese guideline uses the following norm for the placement of blocks by hand. In the guideline it is assumed that a block can have a maximum weight of 100 kg and the distance the block has to be moved over is 30 meters.

	Unit	Amount of Unit	Cost per unit		Total Cost	
Material					**VND**	**-**
Other material	*%*	*10*	*Unknown*			
Manpower					**VND**	**22.644**
Manpower 4,0/7	*Man day*	*0,25*	*VND*	*90.576*	*VND*	*22.644*
Construction machines					**VND**	**-**
TOTAL					**VND**	**22.644**

Table 17-1

By adding the cost for material, manpower and construction machines, the total cost of placing one block can be found: 22.644,10 VND.

The 0.25 manpower means that a worker can place 4 unit/blocks with a maximum weight of 100 kg per piece per day.

The maximum weight of one column of Basalton® corner set, consisting of 1,3 m^2 and being 50 cm high, is 1038 kg. This is about 80 kg per 0,1 m^2. Thus a worker can place 0,4 m^2 per day, costing 22,644.10 VND. This means 2,5 workers are necessary to make 1 m^2 in a day, costing 56.610,25 (= 2,5 * 22.644,10) VND. In the report this number is rounded to: 56.500 VND

17.7 Cost and production speed of placing Basalton® by hand Dutch norm

An experienced stone setter can place around 30 to 40 m^2 per day, depending on the height and thus weight of the blocks. Using the price level of a Vietnamese worker with skill level '4,5/7', this costs 98.268,00 VND per day. This is 3.275,60 VND per m^2 (for 30 m^2). In the report this number is rounded to: 3.300 VND.

17.8 Cost and production speed of placing Basalton® Vietnamese way

With 10 workers and 3 cranes:

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**-**
Manpower					**VND**	**905.764**
Manpower 4,0/7	*Man day*	*10,00*	*VND*	*90.576*	*VND*	*905.764*
Construction machines					**VND**	**4.925.550**
Cranes	*Shift*	*3,00*	*VND*	*1.641.850*	*VND*	*4.925.550*
TOTAL					**VND**	**5.831.314**

With this method, 450 blocks can be placed per day. Per block, an area of 0,16 m² is placed (0,4 m x 0,4 m). This means per day 72 m² is placed. This equals to a cost of 80.990,47 VND per m².

Using 10 workers and 1 crane:

	Unit	Amount of Unit	Cost per Unit	Total Cost
Material				**VND -**
Manpower				**VND 905.764**
Manpower 4,0/7	*Man day*	*10,00*	*VND 90.576*	*VND 905.764*
Construction machines				**VND 1.641.850**
Cranes	*Shift*	*1,00*	*VND 1.641.850*	*VND 1.641.850*
TOTAL				**VND 2.547.614**

This equals to a cost of 35.383,53 VND per m². In the report this number is rounded to: 35.400 VND.

17.9 Cost and production speed of placing Basalton® by machine according to norm

For block (Basalton®) installation by machine, the following guideline prices are used.

Unit: 1 block

Installation	Elements	Unit	Block weight (ton)		
			≤2T	≤ 3T	>3T
Block installation	*Material* Cement mortar M	M^3	0,07	0,10	0,12
	125 Other materials	%	10	10	10
	Manpower 4,0/7	Man day	1,05	1,15	1,20
	Construction machine Crane 10T	Shift	0,05	0,05	0,05

Block installation	Unit	Material	Man power	Machine	Unit price
-Weight ≤ 2tons	1 block	37,157.00	95,105.00	82,093.00	**214,355.00**
-Weight ≤ 3tons	1 block	53,082.00	104,162.00	82,093.00	**239,337.00**
-Weight >3tons	1 block	63,698.00	108,691.00	82,093.00	**254,482.00**

Price for machine is with operator included.

This means that the price for placing blocks up to 2.000 kilos is 214.355 VND. The maximum weight when using the highest size of Basalton® per m² is about 1038 kg.

Blocks are placed per set. This means that only around 1 m² is placed every time. 1 m² x 20 gives 20 m² per day at a cost of 4.287.100 VND. This is VND 214.354,71 per m².

Assuming placing one m² of blocks doesn't take 24 but instead 10 minutes, per day 48 m² can be placed. The cost per m² will then be 89.314,58 VND (= 214.355,00 * (10/24) and also = 4.287.100,00 / 48)

The Vietnamese guidelines indicate that a crane can place 20 blocks weighing less than 2 tons per day. Basalton® blocks are placed with 1,3 m² per time, weighing 1 ton. This means 26 m² can be placed per day. That is one m² per 20 minutes at a cost of 214.355 VND per lift. Per day this is 5.144.520 VND.

Assuming one m² can be placed faster, say one m² per 10 minutes, 48 m² can be placed per day. This is 89.314,58 VND per m². In the report this number is rounded to: 89.300 VND.

17.10 Placing by machine with Dutch standard

	Unit	Amount of Unit	Cost per Unit	Total Cost
Material				**VND -**
Manpower				**VND 294.804**
Manpower 4,5/7	*man day*	*3*	*VND 98.268*	*VND 294.804*
Construction machines				**VND 1.641.850**
Crane 10T	*Shift*	*1*	*VND 1.641.850*	*VND 1.641.850*
Total (without griper)				**VND 1.936.654**

Because placing the blocks by machine is new for Vietnam, it is assumed the placement speed is reduced by factor 2. This means 250 m² can be placed per day. This is about 1 m² per 2 minutes. Using Vietnamese prices, the cost per m² then becomes 7.746,62 VND. In the report this number is rounded to: 7.750.000 VND.

17.11 Calculated speed for machine

	Unit	Amount of Unit	Cost per Unit	Total Cost
Material				**VND 3.871**
"Other material"		0,1042	VND 37.157	VND 3.871
Manpower				**VND 9.511**
Manpower 4,0/7	Man day	0,105	VND 90.576	VND 9.511
Construction machines				**VND 8.209**
Crane 10T	Shift	0,005	VND 1.641.850	VND 8.209
TOTAL				**VND 21.590**

For this calculation the Vietnamese calculation method is used to start with. This method is explained in appendix 17.9. By assuming one block can be placed per 2,5 minutes, this equals 200 blocks per 8 hour workday. This means per placement of block, 0,005 (= 1 / 200) crane shift is necessary. For manpower the value of 0,105 is based on the method of appendix 17.9.

17.12 Cost of buying and placing rip-rap

Cost to buy rip rap per m^3, Per m^3 it is assumed 1,20 m^3 quarry stone and 0,06 'stone 4x6' is needed. Also 1,40 manpower 3,5/7 is necessary. By multiplying these numbers with the total m^3 needed, results in the cost of buying rip-rap. See below.

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**4.635.149.400**
Quarry stone	*m3*	*23688,00*	*VND*	*185.000*	*VND*	*4.382.280.000*
Stone 4x6	*m3*	*1204,14*	*VND*	*210.000*	*VND*	*252.869.400*
Manpower					**VND**	**2.322.951.165**
Manpower 3,5/7	*Man day*	*27636,00*	*VND*	*84.055*	*VND*	*2.322.951.165*
Construction machines					**VND**	**-**
TOTAL			**Cost per unit**		**VND**	**6.958.100.565**

Cost for transport and to place rip-rap per m^3 are as follows:

	Unit	Amount of Unit	Cost per Unit		Total Cost	
Material					**VND**	**-**
Manpower					**VND**	**869.152.200**
Manpower 4,0/7	*Man day*	*9595,83*	*VND*	*90.576*	*VND*	*869.152.200*
Construction machines					**VND**	**8.536.491.412**
Crane 10T	*Shift*	*456,94*	*VND*	*1.641.850*	*VND*	*750.234.236*
Truck 7 tons	*Shift*	*7615,74*	*VND*	*1.022.390*	*VND*	*7.786.257.176*
TOTAL					**VND**	**9.405.643.612**

This is used to calculate the total price as follows: A crane can place one block per 2,5 minutes. This equals 200 blocks per crane per day, this is 43,2 m^3 per day per crane.

A total of 19,740 m^3 is necessary, thus 450 crane shifts are necessary.

One manpower can place 2,05 m^3 per day. Thus 9450 manpower is necessary in total.

By multiplying the needs for transport and placement with the cost per shift, results in the total cost for the placement of rip-rap. This is 8,5 billion VND

18 APPENDIX MONITORING PLAN

Failure mechanisms are the different possibility's covering the different ways of failing of the dike or dike section. Failing can occur through a single mechanism or a combination of different kind of failure mechanism. There are twelve types of failure mechanism, see Figure 18-1.

In Vietnam failing of the sea dikes mostly occur by wave overtopping in combination with stability of the inner and outer slope. When wave overtopping occurs, the water that flows over the dikes erodes the inner slope what can result in sliding of the inner slope.

The different kinds of failure mechanism are explained.

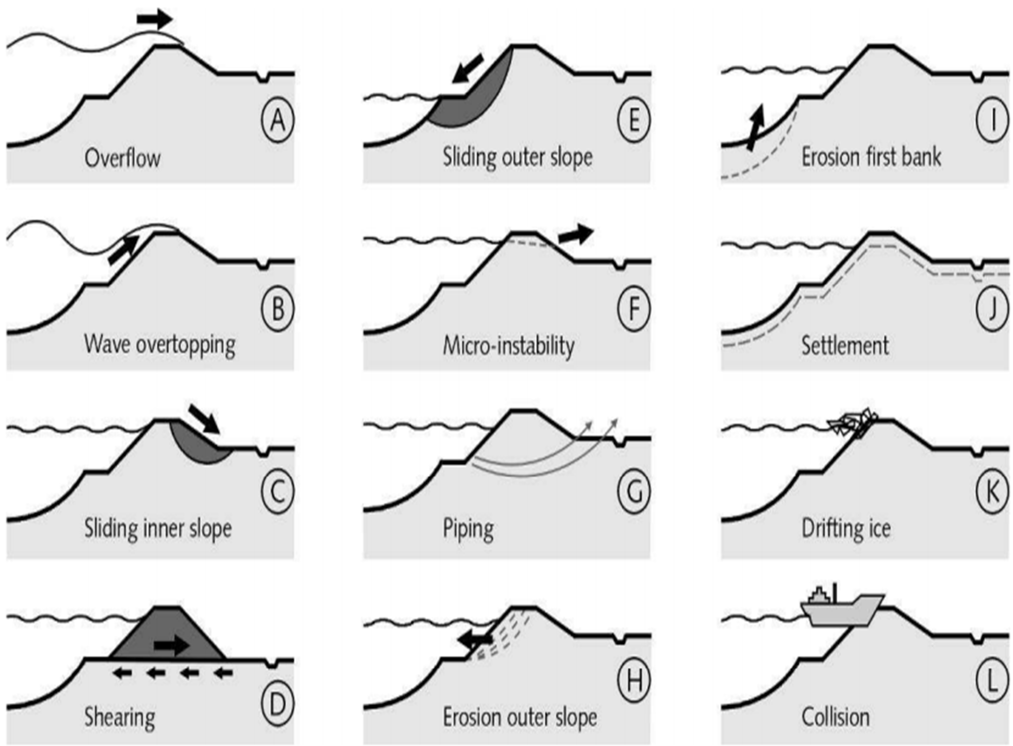


Figure 18-1 Failing mechanism

A: Overflow

Overflowing of a dike is the most obvious failure mechanism. The limit state of that mechanism is a statement that a dike must be able to withstand a water level that will be exceeded with a normative small probability. This means that the resistance to erosion of the inner slope is important. Sometimes, if the inner slope is very resistant to overflowing water another criterion becomes important. "How much water can be absorbed by the hinterland." This is especially important if the hinterland is water itself. As this phenomenon often involves great velocities the stability of the dike itself is the decisive factor to be reckoned with.

B: Wave overtopping

The overtopping of waves is related to the water level, as is the overflow mechanism. The amount of water the inner slope can withstand is defined as a limit state. This amount is connected to the state of the inner slope. The limit state can be different for adjacent sections of a dike, if the inner slope is different.

C: Sliding inner slope

If the sliding of an inner slope will occur during a period of high water the "overall" stability of the construction will be in jeopardy. Also this limit state, however, could occur and leave the water retaining function of the dike intact. If the whole of the top is sliding down far enough however, the overtopping of water or waves will do the rest. If that water is not high enough (any more) is possible that the dike will still retain the water. Nevertheless a significant sliding is considered a limit state. The stability factor (load over resistance) based on a calculation with the Bishop method is defined as a limit state.

D: Shearing

This mechanism is not observed often in prototype. No special limit state is defined. A recent failure indicates however that this mechanism might be more important than expected. The failure occurred in the Netherlands near Wilnis with a regional water defence constructed of peat. For calculations this shearing is treated as a horizontal sliding. Failing by shearing occurs only by dykes with low own weight. For sea dikes build from sand, clay and a heavy revetment, shearing will not occur. For the sea dike design in Cat Hai this failure mechanism can be ignored.

E: Sliding outer slope

The sliding of the outer slope can occur if the water in front of the dike drops very fast. The water inside the soil mass cannot follow and the pressure of the water inside the dike will cause the dike to slide towards the water. This process can cause severe damage to the construction but it will seldom cause direct problems as the hydraulic load is dropping already. A second high water surge will cause problems. The limit state is similar to that of the inner slope.

F: Micro instability

Micro instability is caused by seepage water that reaches the inner slope of a dike. The name "micro" is used because it concerns the smaller particles of the dike that can erode from water pressures from the inside. It could be a minor damage, but, if the build up of the pressure inside the dike body is big enough, the integrity of the whole construction is in jeopardy. Often there will be a substantial residual strength, but this cannot be calculated yet. The connection of the phreatic line inside the dike body with the surface of the inner slope, is considered the limit state of this mechanism. This can be calculated with standard soil water analysis. The unknown materials and the stratification of the dike itself is the real challenge.

G: Piping

Piping is a process very similar to micro instability. In the case of piping the erosion will occur underneath a cohesive layer in the subsoil. The seepage water will, if the exit gradient is high enough, erode the soil particles from underneath the dike body and forms a meandering cavity, "a pipe". Older descriptions of the limit state (Rules of Bligh and Lane) are indirect. They are linked to the gradient over de subsoil. The maximum gradient depends on the type of soil. A modern definition of the limit state gives a maximum length of the pipe that can exist without uncontrollable erosion (Rule of Sellmeijer). It should be noted that a pipe needs a roof over its head. A cohesive layer, that is on top of erodible and permeable subsoil.

H: Erosion outer slope

The erosion of the outer slope can start the collapse of a dike. The dike is often protected against this erosion process by a top layer that provides the resistance. In circumstances, where the load is not so big, (river dikes/wave run up zone) the protection often is provided by a grass cover on top of a clay layer. If the load is higher, a more resistant solution is chosen. These are for example:

- Asphalt;
- Natural stone;
- Concrete blocks.

There will be a residual strength of the layers underneath this top layer, but as the resistance of these layers cannot easy be calculated, the limit state is defined as damage to the top layer.

I: Erosion "first bank"

The erosion of the first bank can start the instability of the outer slope of a dike. This process can start if there is a steep under water slope in front of the dike. If the sand in the subsoil is loose packed, this mechanism can become a problem. If, by some reason, the sand is moving into a more dense state the present soil water pressure can raise quickly. If this overpressure is not drained fast enough, the sandy subsoil will change into a thick fluid. This is called settlement flow. The soil mass involved often is much larger then can be found with a “normal” slide. The limit state of this erosion is a certain distance from the toe of the dike where no erosion gully with a certain depth and steepness of the slope can be permitted. This is also limited by the width of the erosion gully, because if this gully is filled with the debris of the settlement flow the process will stop.

J: Settlement

Settlement of a dike is a strange process in this context. Collapse is often regarded as a phenomenon that happens quickly. Settlement on the other hand is a process that will occur over years. Both views are not complete. The deformations of a constructions based on geotechnical processes can happen very fast like an avalanche (seconds) ore very slow like for instance a consolidation process or creep (decades) and everything in between. Very slow processes will be met in the maintenance programs while the fast deformations are taken into account during the design phase. In general the deformation of a dike during the life cycle of 50 years is taken into account in the design. Sudden deformation is often triggered during the building process or an extreme load.

K: Drifting ice

Because of the climate in Vietnam there will be no ice on the sea, so drifting ice will never be a cause of dike failure.

L: Vessel collision

The collision of a vessel could theoretically damage a dike and start the overall collapse. In practice there is no limit state defined. The chance that a ship will damage a dike is low, destruction of a construction like a sluice is more probable. Dike designing that could resist a collision with a vessel, would result in unrealistic dimensions of the dike.