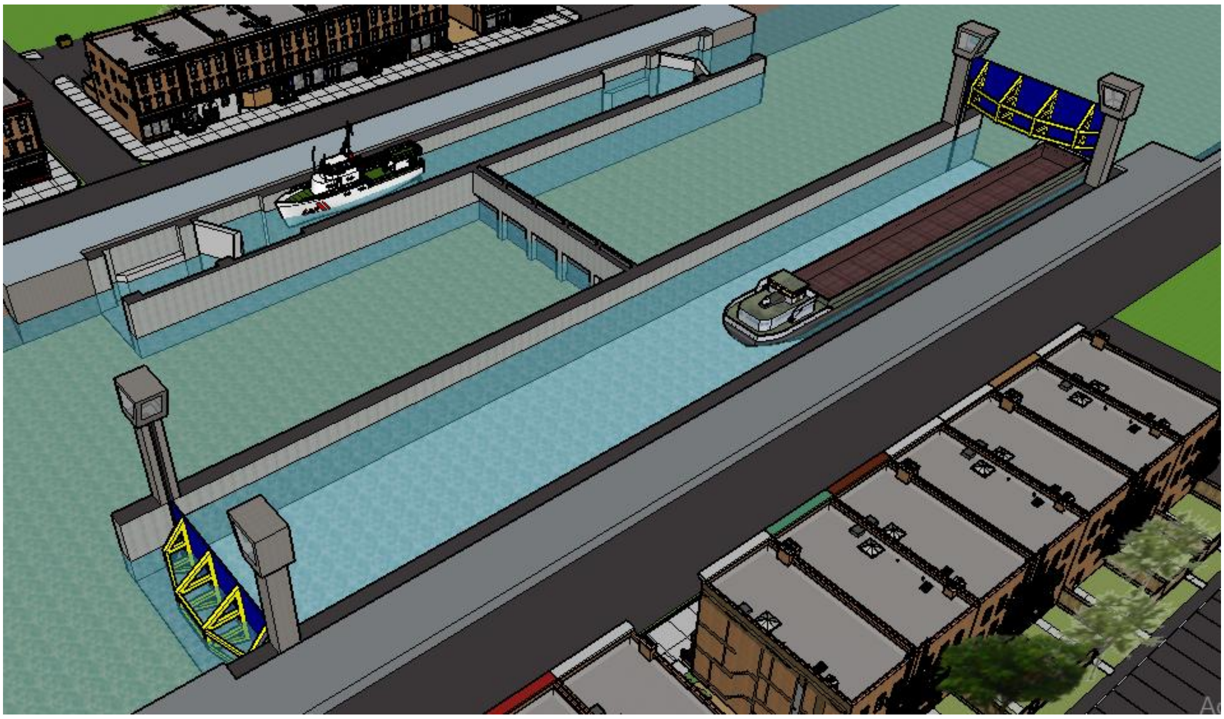

Design of a flood-retaining hydraulic structure at Phú Dinh.

Abstract

This report contains an overview of the process of designing a tidal barrier structure in the river Phu Dinh. First different alternatives are weighted, then two are worked out into more detail. Finally a structure with two navigation locks and a dam with culverts is chosen and some more details and the construction phases of this structure are worked out.

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Preface

This report is the result of our study on the Phu Dinh hydraulic structure in Ho Chi Minh City. The goal of this project was to make a new design for a structure that provides flood protection and keeps navigation possible. We lived in Vietnam to work 8 weeks on this project, in this way we learned a lot about working in another land and culture. Due to the multidisciplinary nature of the project and our group we gained a lot of experience. Before arrival our knowledge about the project was limited, but when we arrived we quickly got insight in the situation and started working on our design. The result can be found in the following report.

We want to thank our supervisors' dr. Trinh Cong Van, ir. Henk Jan Verhagen and ing. Bert Everts for their time and help throughout the project. Also we want to thank ir. Tran Nguyen Cong Danh and ir. Phan Huu Cuong from the IWER who provided us with the information for the design and who were available for technical questions. The IWER also provided us with a place to work in their office at the Thuy Loi University.

During our stay we had the possibility to travel and look at various beautiful places in Vietnam like for instance Da Lat, Vung Tau, Phu Quoc and the Mekong Delta. We had the opportunity to meet Truong Tuan Duy who showed us the office of Royal Haskoning in HCMC and provided us with some technical background information on the project.

In conclusion we had a very eventful, instructive and enjoyable time in Vietnam and we want to thank everyone who contributed to this project.

Yours sincerely,

Sytse Alkema
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Summary

Ho Chi Minh City is flooded on a regular basis due to the lack of flood protection. Plans have been made to protect the city from flooding in the future and therefore a barrier is built at Phu Dinh. This barrier needs to fulfil three requirements. Firstly it should protect HCMC against a flood, as well resulting from tidal waves as from peak rains. Secondly during the dry season it should keep the water level above a minimum. Thirdly, while performing the first two requirements, navigation should be hindered barely.

In order to find the ideal design first 3 variants were selected from 17 options with a MCA. Two of these variants were merged because of additional research. Variant A, two locks and a culvert dam, and variant B, a lock with a tidal barrier, were made into a preliminary design. These designs were compared with a new MCA and variant A was chosen as the best design.

Variant A (Figure 1) consists of a big and a small lock in order to keep the navigation possible during flood protection. When possible the big lock can be opened for free navigation due to its lift gates. The small lock will be built with double mitre gates. Between the locks a dam with culverts is built to close of the river and provide the needed conveyance area for the outflow of rain water. This design meets the requirements stated about flood protection, outflow regulation and navigation. The risks during operation are low because of the double set of doors in both locks. Because there are two locks the chances are low that the navigation will ever be totally obstructed. The estimated costs of this variant are VND 1150 billion (€ 46 million).

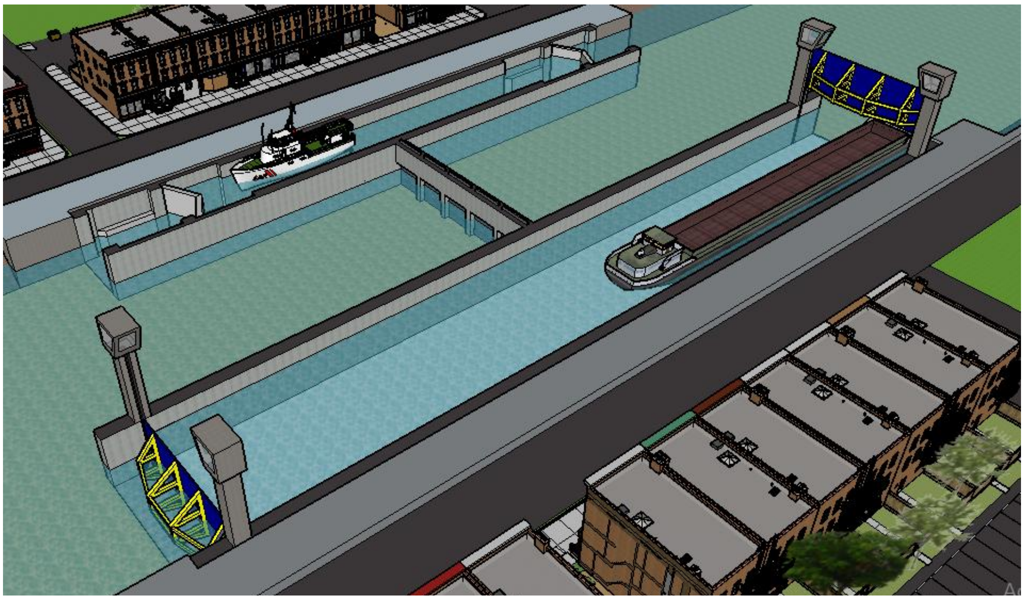


Figure 1: Impression variant A, the Mekong Delta on the right side and HCMC on the left.

It is recommended to look at improving the barrier by building a bridge over it. Surrounding the barrier there are no embankments present, therefore in order for the barrier to function these have to be built. Further the application of pumps in the culvert can lower the risks of a flooding due to rain, when investigating this it is important to do research on the catchment and conveyance area in the city the barrier has to deal with. During the investigation it was hard to make a good estimation about the importance of stakeholders and MCA criteria. Therefore more experience with the local environment could improve the design.

Figures & Tables

Table of content

List of abbreviations

CPT	=	Cone Penetration Test
CtCd	=	Center to Center distance
DWT	=	Dead Weight Tonnage
FOS	=	Factor Of Safety
HCMC	=	Ho Chi Minh City
HLD	=	Head Level Difference
Inside of barrier	=	The Ho Chi Minh City side of the barrier
IWER	=	Institute for Water and Environment Research
MCA	=	Multi Criteria Analysis
MSL	=	Mean Sea Level
Outside of barrier	=	The Mekong Delta side of the barrier
SPT	=	Standard Penetration Test
UWC	=	Under Water Concrete

1. Introduction

In this chapter a broad overview will be given of the project itself, the problem description and the related goal of the project. Also the relation of this project with other projects will be discussed, as are the stakeholders, the navigational routes and finally an outline of the report will be presented.

1.1 Problem description

The main problem of Ho Chi Minh City is frequent floodings of the area as a result of a combination of high tide, rainfall and high river discharges from upstream. These problems are displayed in figure 2. The floodings of HCMC are also related to three other problems, they will be mentioned here below but are outside the scope of this report:

1. **The urban drainage system**: as a result of the continuous expansion of the urban area the capacity of the drainage system became insufficient to discharge the rainfall. Moreover the urban drainage system is designed such that water is discharged by gravity via the urban drainage network and open channels into the rivers surrounding and crossing the city. Due to this open connection, the high water levels in the river directly affect the water level in the open channels which causes flooding
2. **Subsidence of the city**: due to extensive groundwater withdrawal the soil compacts resulting in subsidence of the city which causes extra flood risk
3. **Sea level rise**: sea level rise in the future will cause higher water levels in the river deltas. The probability of floodings will therefore increase.

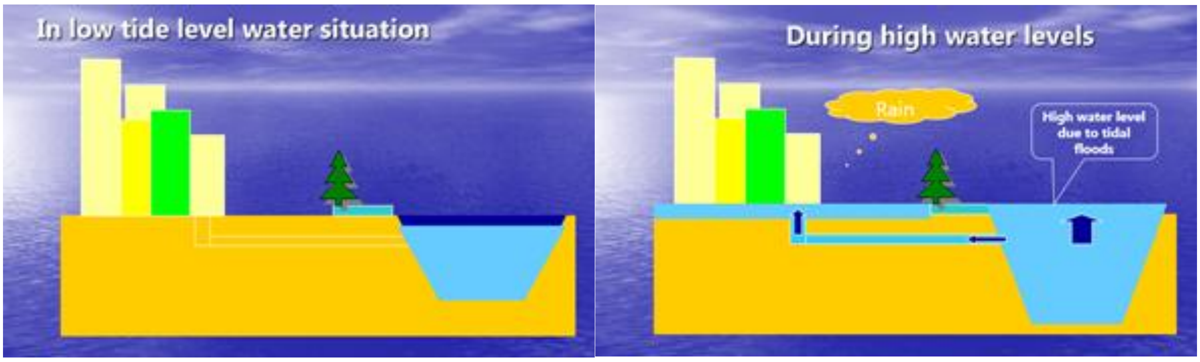


Figure 2: How flooding occurs in Ho Chi Minh City

In order to prevent these floodings, an entire tidal barrier system has been developed to protect the city concerning the high tides. The system also needs to keep the water levels in the canals as low as possible to offer storage capacity. This system as a whole consists of 13 hydraulic structures along with embankments along the rivers and canals. This system initially meant to close of a very large region consisting of the Ho Chi Minh province and the province of Tinh Long An. The entire project has been estimated to cost 750 million USD.

This system would have been a great solution to the problem since it would protect a larger area than Ho Chi Minh City, allowing further growth of the city within the closed-off region. Apart from the benefits concerning the possibility for expansion of the city and further urbanization the larger region would also provide larger reservoir capacities increasing the flood resisting capacities of the system. Due to a lack of available funding the enclosed area was reduced to a size only protecting the city itself and not the surrounding lands. Even this alternative worked out to be more expensive than possible so the municipality also included private investors to fund several hydraulic structures, the total budget came down to approximately 500 million USD. This is for the six remaining hydraulic structures that are under construction since July 2016.

The project that will be treated in this report is one of the six hydraulic structures, part of the total project. The hydraulic structure is located in the south east of Ho Chi Minh in Phu Dinh and has the purpose of functioning as a tidal barrier as well as giving the opportunity to offer constant navigation through the river.
Figure 3 indicates the location of the structure with a green dot. The red southern dotted line indicates the enclosed area by an embankment according to the original plan whereas the solid line above displays the enclosed area by an embankment according to the new plan.

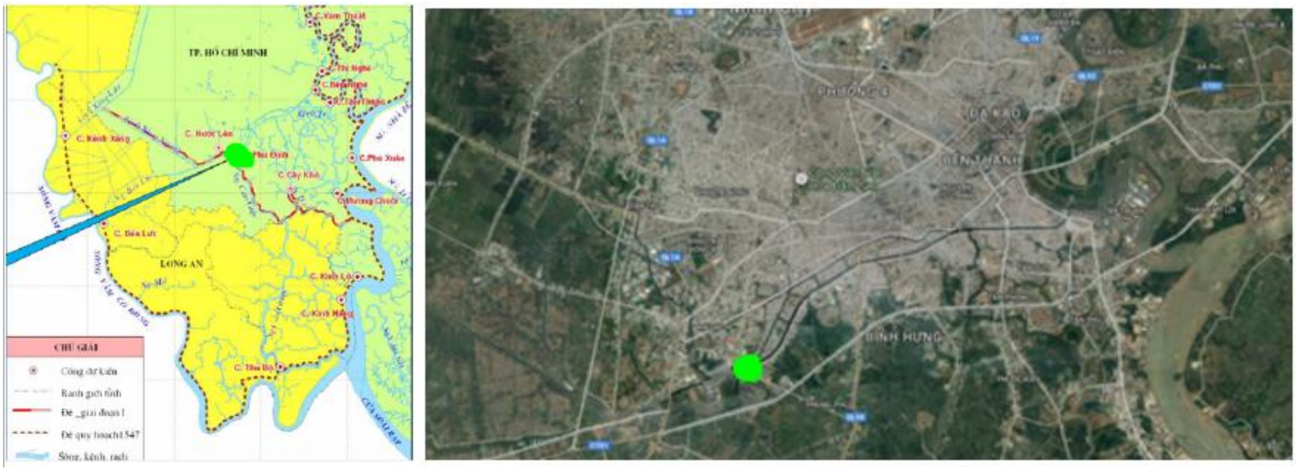


Figure 3: Project area and location of concerning structure

Concerning the given structure a design has already been made and it is in the construction phase. The IWER team chose to build a 40 m wide vertical tidal gate that can be lifted and lowered into the desired position. When high tide occurs during rainy season, the tidal sluice has to be closed which limits the navigation a lot. So a navigation lock will be constructed next to the gate in order to ensure constant navigability of the river. Both the lock and the gate must be able to withstand both positive and negative water level differences. The design can be found in figure 4. In this figure a distinction is made between inside and outside of the barrier. Inside is the Ho Chi Minh City area, outside is the area of the Mekong Delta.

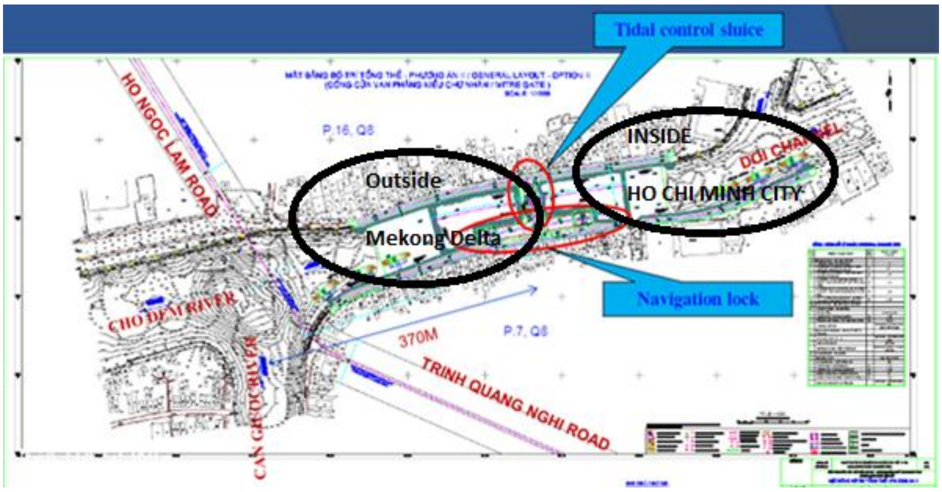


Figure 4: Barrier/lock complex near Phu Dinh

1.2 Project outline

New insights can be achieved by re-developing alternatives for the lock/barrier-complex which is currently under construction at Phu Dinh. The alternatives are developed in the conceptual design phase after which they are assessed and compared with the help of a multi criteria analysis. This analysis should indicate which alternative would serve as the most suitable options and determines the most promising variants. After these variants are chosen, they will be worked out in more detail in the preliminary design phase. This phase gives a more precise idea and cost estimate of the design, bringing them to a higher level of detail, more calculations will be made determining the rough dimensions of the structures.

After this phase the resulting variant will be chosen and final calculations will be performed.

The client wishes to see if their solution somewhat matches with the ideas and best alternative of the project team. They want a new vision of a possible structure at Phu Dinh, so the project team should work with basic knowledge of the constructed structure at Phu Dinh. After choosing a final design, the project team can provide a preliminary design, that gives the client new insights for the structures that will be built in the future..

1.3 Goal

The client is IWER (Institute for Water and Environment Research). It is a research institute linked to the Thuy Loi University and was responsible for the design of the hydraulic structure at Phu Dinh. They would like to know if there are other (better) options to construct such a hydraulic structure in the future. They are interested in how such a structure should look like, because expanding the protected area of the Mekong Delta in the future is likely to occur. This assignment serves as some sort of assessment of their work and hopefully new insights for future hydraulic structures. A sub goal is to compare the design of IWER with the preliminary design of this report. The following goal of the project has been defined:

''Develop a conceptual and preliminary design of a flood-retaining hydraulic structure at Phu Dinh (HCMC) for potential future use and assessment.''

1.4 Stakeholders

Building a barrier and lock will influence the surroundings. This structure will contribute to prevent flooding of Ho Chi Minh City. This construction has some side effects which are positive and negative. The three most important stakeholders are taken into account in this study and shown in figure 5. These stakeholders will be taken into account in the multi criteria analysis. The three main stakeholders are:

- Ho Chi Minh City
- People living next to Phu Dinh Barrier
- Port activities

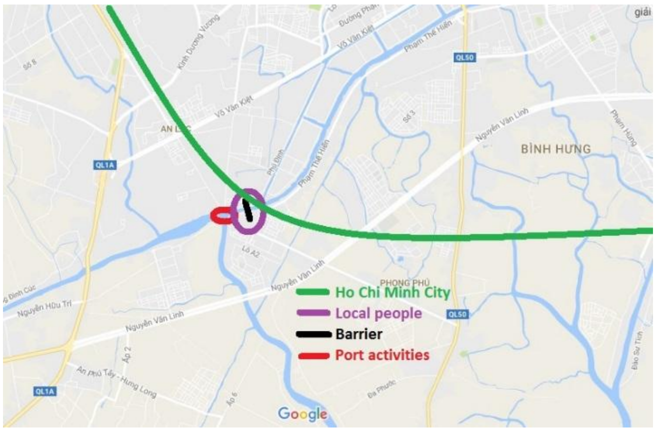


Figure 5: stakeholders location

1.4.1 Ho Chi Minh City

Prevent floodings in Ho Chi Minh City was the reason to develop the original MARD plan. High tide or a combination with heavy rainfall causes the water levels in the open channel system in Ho Chi Minh City to rise and cause floodings. It is assumed that there will be no floodings in Ho Chi Minh City when the water level in the rivers at Ho Chi Minh City does not exceed +1.0 m MSL (Mean Sea Level).

There is a side effect for the Ho Chi Minh City municipality to build the barrier at Phu Dinh. The rivers in Ho Chi Minh City are polluted, so is the river at Phu Dinh. This due to the fact that the river flow from upstream to downstream changes due to the tide in combination with rainfall. That is why waste in the river is not able to flow away from the city. Building a barrier is an opportunity to make a one way water flow stream from the city to the delta at sea.

1.4.2 Local people

The barrier is located at Phu Dinh, on both sides of the river there is a road and people who live adjacent to this road. The construction of the barrier can disturb the living of those people. Also the barrier can influence the dynamic of the surrounding area. A wide support of the local community for the barrier can limit possible complaints and provides opportunities to let them positively influence the design and construction phase.

1.4.3 Port activities

In the area around Phu Dinh port activities are going on. For the competitiveness of the ports in Ho Chi Minh City and ports upstream the river it is important that vessels can still pass, especially with a tidal barrier present. Another aspect to be a competitive inland waterway is to have a low travelling time for the river at Phu Dinh. The stakeholders and their interest are displayed in table 1.

Stakeholders	Interest
Ho Chi Minh City	Prevent flooding, improve water quality of river
Local community	No nuisance due to the project
Port activities	Be a competitive waterway

Table 1: Stakeholders and interest

1.5 Navigational overview

Around the area of the barrier the only option for crossing is by use of the ferry service crossing the river. An opportunity is, to include a bridge in the final design. This would increase accessibility of the area and also reduce the obstruction for other ships travelling along the river.
The barrier will be located in the Phu Dinh channel, from Figure 6 it can be seen that this channel is crucial for shipping to enter and exit the city from the west, therefore it is crucial that the barrier is capable of satisfying the navigational requirements at all times.

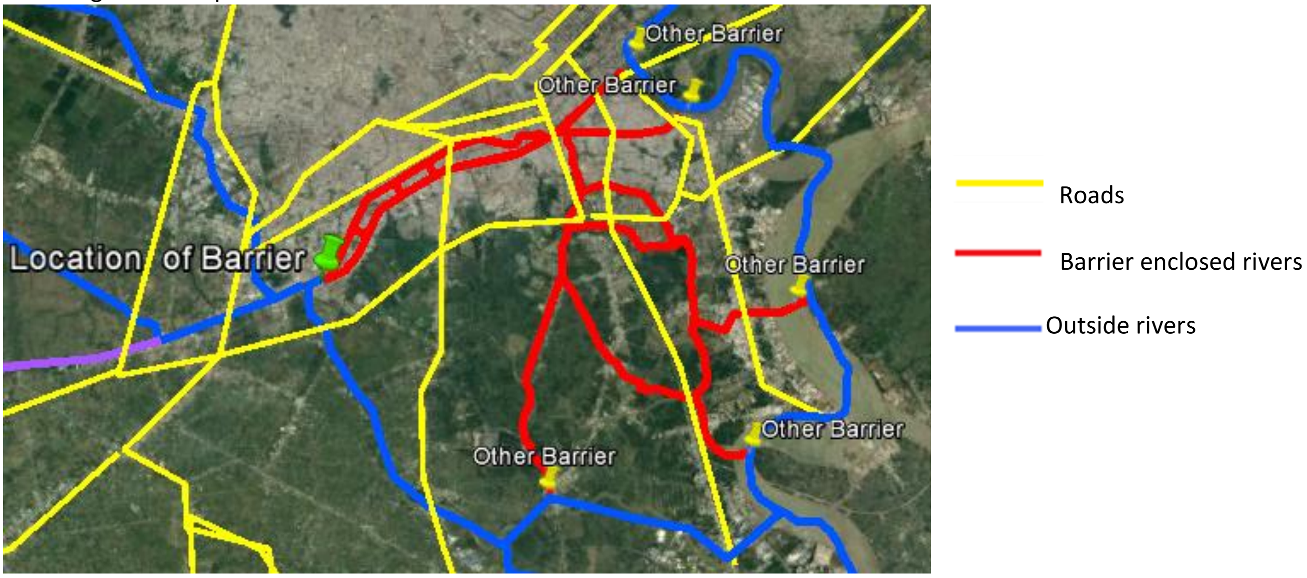


Figure 6: Routes for navigation

In figure 6 the yellow lines show the highways through Ho Chi Minh (around the position of the barrier these are the only road crossings), red are the rivers/canals enclosed by the barriers and blue are the surrounding rivers outside the defence complex. More information about the navigational overview can be found in appendix B. In this appendix the routes are showed more zoomed out in comparison to the figure above and attention is paid to overland traffic.

1.6 Reference projects

Seven projects are studied to gain knowledge about possible designs for our project at Phu Dinh. Information is obtained about costs, span and the closure mechanism. The following projects are investigated:

1. Maeslantkering
2. Thames Barrier
3. Balgstuw Ramspol
4. Hartelkering
5. Hollandse Ijssel
6. T.J.O'Brien lock and dam
7. The Boat conveyer

Relevant data about these projects can be found in appendix A. An overview of the most relevant aspects is displayed in table 2. The costs are transformed to net present values.

	Country	Span (m)	Cost (€ million)	Closure mech. (time)	Characteristic of gate
Maeslantkering	Netherlands	400	450	Long	Vertical radial gate
Thames Barrier	England	525	1540	Long	Circular gate
Balgstuw Ramspol	Netherlands	70	96	Long	Air balloon
Hartelkering	Netherlands	100+50	200	Quick	Vertical lift gate, round shaped
Hollandse Ijssel	Netherlands	80+24	154	Quick	Horizontal radial gate
T.J. O'Brien lock and dam	USA	90	56	Quick	Vertical radial gate
The Boat conveyer	Netherlands	/	cheap	Quick	Conveyer

Table 2: Reference projects

1.7 Report outline

This is a report of the design process for a tidal barrier complex, in the river at Phu Dinh.

- **Chapter 1**: the basic problem of flooding in Ho Chi Minh City is described and the goal of this investigation is shown.
- **Chapter 2**: the required data is summarized and it is explained how some of the data is obtained and can be used.
- **Chapter 3**: a MCA is performed to select the most promising variants out of the 17 different variants that the design team came up with.
- **Chapter 4**: takes the reader through the phase of translating the obtained ship data into dimensions for a lock and also for two locks.
- **Chapter 5 and 6**: two different variants are elaborated into more detail. Variant A in chapter 5 consists of two different locks. A big one with lift gates, a small one with mitre gates and a dam with culverts in between. The second alternative, alternative B, consists of a tidal barrier with a lift gate and a lock with mitre gates.
- **Chapter 7**: describes the choice for the definitive variant, for which a MCA is used.
- **Chapter 8**: a method of construction is described for the chosen variant.
- **Chapter 9**: additional details are calculated for the chosen design. For instance a more detailed look is taken at the culvert dam, the bed protection and the use of pumps.
- **Chapter 10**: final recommendations, a conclusion and a discussion are given.
- **Chapter 11**: references can be found in chapter 11 and the appendices can be found in a separate document.

2. Data

An analysis was carried out to determine the input data for the design. A required data tree was created for all the main elements in the project. This chapter describes the data that was obtained and will be used for the hydraulic structure. Most of the requested data was available at the Thuy Loi University and supplied by IWER. Part of the data is neglected and some data is assumed in collaboration with the supervisors from IWER. Hydraulic data will be used but also structural, geological and transport data is necessary to make a design. In some cases the required data is used in several design fields.

2.1 Design parameters

table 3 below gives an overview of the gathered data.

Design Criteria		Unit	Value	Comment
Hydraulic Parameters				
1	Flow velocity	m/s	1.80[(1]	v = Q/(B*h)
2	Highest tidal level	m	1.97[(2]	With P(%) = 0.2
3	Lowest tidal level	m	-2.33[(2]	With P(%) = 99
4	Sea water level rise	m	0.3	With P(%)=0.2 in the year 2050
		m	0.77	With P(%)=0.2 in the year 2100
5	Sedimentation	m/year	0.05-0.1	Without catchment upstream 0.3 m/year (according to the client)
6	Water elevation in storm conditions	m	0.7[(2]	On top of the given maxima
7	Wind	m/s	25	In storm conditions
8	Maximum discharge	m3/s	185.5	From inside to outside
		m3/s	382.4	From outside to inside
9	Maximum rainfall	m/day	0.18[(2]	Historical data from 1942
10	Tide	m	[(3]	Tidal data 2016
Geological Parameters				
10	Width of river	m	66[(1]	At average water level at Phu Dinh
11	Seismic conditions	g	0.0745[(4]	below 0.08, neglected
12	Bed profile		[(5]	See Figure 8 below
13	Bed level	m	-5.5[(6]	The situation just after dredging
14	Soil layers		[(7]	See Table 4
Transport Parameters				
15	Design vessels			Maximum vessel
	Vessel length	m	66.5[(8]	
	Vessel width	m	9.7	
	Vessel draught	m	2.5	
	Mass	t	1000	Vessel displacement
	DWT	t	600	
16	Ship intensity	Ships/day	182[(9]	Average amount of ships per day
Functional requirements				
17	max water level inside	m	1[(10]	For flooding
	min water level inside	m	-0.6[(10]	For navigation and environmental issues
18	Lifetime	years	100	

Table 3: Data overview

2.2 Explained parameters

The data parameters that are interpreted or calculated by the project team are explained in this paragraph, these parameters have a superscript in Table 3. The other parameters are directly obtained from an excel sheet provided by the Thuy Loi University. The parameters with a superscript in the table above will be explained in paragraph 2.2.1 through 2.2.8.

2.2.1 Flow velocity and width of the river

1) The flow velocity in the river is determined, using the measured flow at the location of the Phu Dinh barrier, the local width and the depth. The maximum flow velocity is present when the water level is at its minimum. This is the case, when water is flowing in the direction of the City of Ho Chi Minh, the discharge is 382 m^3/s. The original width at Phu Dinh is the given 66 m. However, due to the building of the lock, the remaining width for the barrier will be 40 m. The depth at maximum water level will be 5.5 m, due to dredging that is just performed in this situation. The occurring water level at minimum discharge is -0.18 m. This leads to a flow velocity of (382 m^3/s/ ((MSL - 5.5 m - MSL - 0.18 m) * 40 m)) = 1.80 m/s at the average water level.

2.2.2 Water level

2) The lowest probability of a highest tidal level derived from the hydraulic data from the Thuy Loi University is 0.2%, this gives a highest tidal level of MSL + 1.97m. The highest tidal level with a probability of 0.5% is only 0.03 m lower. The probability of 0.2% exceedance is chosen because the probability of reaching this water level is lower than 0.5% and the structure only has to be 0.03 meter higher. Thus the risk of flooding will be reduced, by adding a small extra investment.

For the lowest tidal level a probability of 99% is chosen, a higher probability for the minimum tidal level is not available from the derived hydraulic data. Although a probability of 95% exceedance, provides a 0.01 meter higher minimum tidal level, this will lead to a less reliable design and the uncertainty about how often the barrier has to be closed increases.

From historical data (IWER) it can be found that the maximum intensity of rainfall in one day is 0.18 meter. The maximum amount of rain in one year is irrelevant for the design because this gives a lower intensity than the maximum intensity per day. For the design, the governing situation is when the highest tidal level coincides with the maximum intensity of rainfall.

Under storm conditions an extra 0.7 meter of water level has to be added. For the lowest water level this has a positive effect because the barrier can be closed with a lower frequency. On the highest water level, storm conditions have a significant influence. Figure 7 displays the several layers where the maximum water level consists of in 2100 just before dredging.

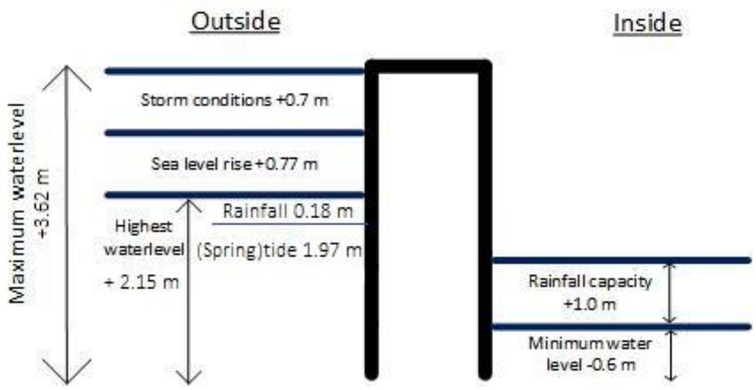


Figure 7: Water level at barrier in 2100, all values are given in MSL

2.2.3 Tide

3) In 2015 the tidal levels of 2016 at Phu Dinh are estimated by a hydrological station, the IWER gave us these values. This data can be used for analysing the correlation between the tide and navigation. These values can also be used to calculate the maximum discharge from inside to outside at Phu Dinh.

2.2.4 Seismic conditions

4) The seismic acceleration at the site of the Phu Dinh barrier has a maximum of 0.0745 g. According to the Eurocode 8 (European Committee for Standardization, 2004) anything below the value of 0.08 g is considered a low seismicity case. The Eurocode is considered to be at least as conservative as the building code in Vietnam, so after discussing this with the client, the Eurocode is used. This means that normal design criteria can be used and no extra loads will have to be taken into account.

2.2.5 Bed profile

5) The bed profile provides information about the altitude of the bed. It is necessary to know the water depth in the river for navigation purposes. Also for the construction it is required to know the water depth in the river to calculate the dimensions of the foundation of the structure. Figure 8 shows a top view of the bed profile at Phu Dinh.

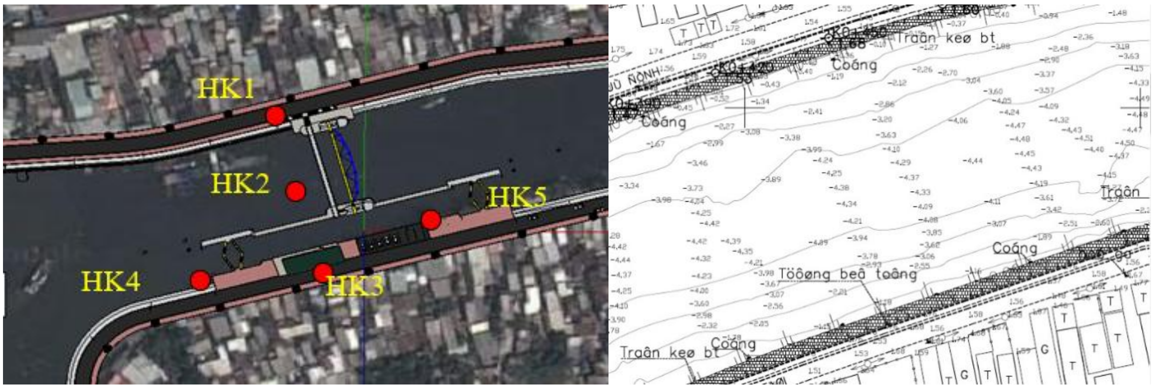


Figure 8: Boring locations and bed profile

2.2.6 Bed level

6) The dredged level just after construction is finished will be brought to a depth of MSL -5.5 m. The minimum required level of the sill, in order to let ships pass is MSL - 3.5 m. The sill will thus be sticking 2 m out of the bottom at the end of construction. This leaves enough room for sediment to be collected around the gate. The sedimentation will not immediately lead to big problems for shipping. After a certain amount of time, maintenance dredging will have to be performed, however this is not a concern for the design of this structure.

2.2.7 Soil layers

7) In order to find the design parameters of the soil at the project location, 5 SPT's and borings have been performed. Figure 8 shows where the soil investigations took place. When looking at the 5 SPT's it can be said that they are very similar, only the precise depths of some layers are different. Therefore the soil layers can be schematised as shown in Table 4.

	Layer number	**Type of soil**	**SPT blows (avg)**	γ_w	**ϕ**	**C**
				[kN/m^3]	[°]	[kN/m^2]
	1	Blue-gray clay, very soft, flowing. High ductility, saturated. Mixed with shells.	0.5	15	5	7.6
	2	Gray-green clay, golden brown. low viscosity, soft plastic state	7	18.5	15	31.3
	3	Yellow-brown clayey sand. Small grains, smooth, saturated. Light till very compacted.	15	19.6	29	4.2
	4a	Yellow-brown dust sand. Small grains, smooth, saturated. Compacted.	50	20.2	32	4.5
	4b	Yellow-brown dust sand. Small grains, medium saturated. Compacted.	30	20.1	32	4.1

Table 4: Soil layers

At the different locations the depth of the layers is different. Table 5 shows the depth of each layer with respect to the local coordinate system.

Layer number	Top layer level (in m above MSL)				
	HK1	HK2	HK3	HK4	HK5
1	1.45	-4.46	1.61	-2.52	-2.87
2	-15.25	-17.96	-16.99	-16.12	-17.37
3	-19.85	-23.06	-21.39	-20.02	-21.47
4a	-29.55	-35.46	-31.59	-29.72	-30.47
4b	-35.55	-39.06	-35.39	-33.52	-34.27

Table 5: layers at different locations

2.2.8 Design vessel

The dimensions of the design vessel are provided by the IWER. Also a table with ship classes according to DWT and the related dimensions of the vessels is provided. This table has a gap between 300 DWT and 1000 DWT, using interpolation the dimensions of the vessels with a DWT between these numbers is obtained.

2.2.9 Ship intensity

9) The number of 182 boats per day is related to the average of boats over a period of 6 months. There are different types of ships, namely cargo vessels and tourist vessels. The majority of the ships are cargo vessels. The design vessel is the transport vessel of 600 DWT, this vessel has the biggest dimensions of all barges, transport vessels, canoes or tourist boats. The distribution of boats arriving at Phu Dinh is important to determine the capacity of the lock. Therefore, three categories are created and subsequently calculated over the 6 days of vessel observation in august 2016. The three categories are:

- The average amount of all boats per hour over the observed six days.
- The maximum amount of all boats per hour over the observed six days.
- The maximum amount of the design vessels per hour over the observed six days.

The three categories and their distribution are displayed in figure 9. This graph visualises the average and maximum arrival pattern of the vessels. The astronomical tide is added in the graph to see whether there is a correlation between the distribution of vessels and the tide. The astronomical tide is interpolated from the measured highest and lowest water levels on these days.

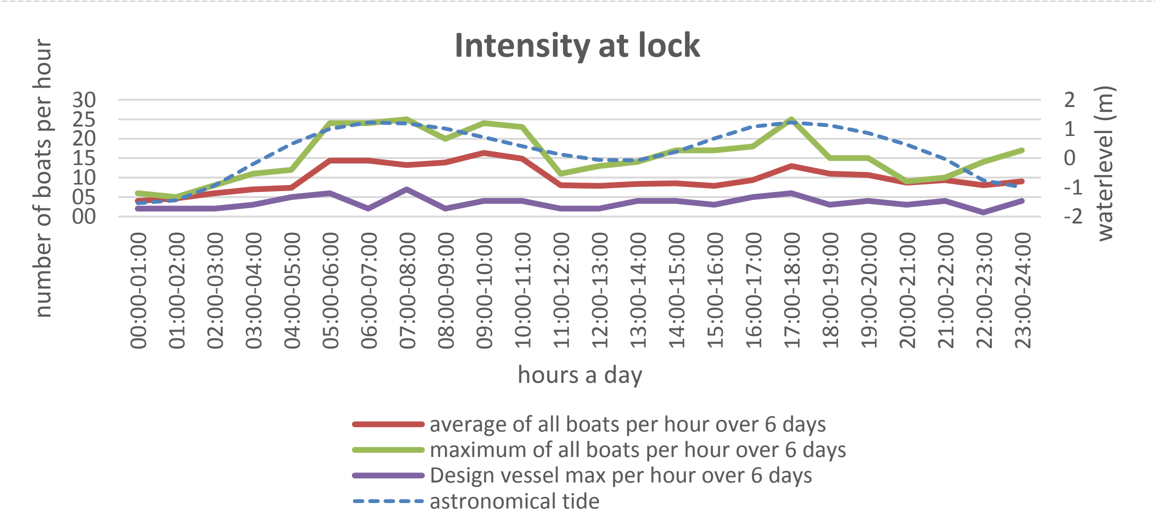


Figure 9: Ship intensity with astronomical tide

Conclusions that can be drawn from figure 9 are:

- The astronomical tide is correlated to the intensity distribution.
- There is a peak in the intensity at the moment of high tide.
- The average arrival pattern of all boats and the design vessel is about constant.
- The maximum arrival pattern of all boats increases during higher tide.
- The maximum arrival pattern of all boats fluctuates around the 15 boats per hour.

To determine the lock capacity in a later design phase the DWT per arrival has to be known. In determining the cycle time tourist boats are neglected because they pass Phu Dinh 0.14 times a day. Canoes can also be neglected because they are very small and pass Phu Dinh with a low frequency. The DWT per vessel passing Phu Dinh is recorded for six month, the result is shown in table 6.

Type of boat	Flow per day	Cargo volume per day (T)
>500T	32	17
301-500T	21	5
201-300T	19	3
101-200T	20	2
51-100T	25	1
5-50T	65	1
Total	182	29

Table 6: Cargo passing the river at Phu Dinh each day

When analysing the table and figure above, it has to be kept in mind that the vessels can pass Phu Dinh loaded and unloaded. Another important aspect is that the largest barge almost has the same dimensions as the design vessel and that smaller vessels still need a significant amount of space in the lock. So besides the purple line in figure 9 the green line is important for determining the lock capacity as well.
Data on ship collisions is not available for the river section at Phu Dinh, according to the IWER. The client specified however, that ship collisions will be prevented by an early warning system, which will be used to monitor shipping at Phu Dinh, so it should not be taken into account in this investigation.

2.2.10 Maximum/minimum water level

10) There is a maximum water level that may occur at the inside (Ho Chi Minh side) of the barrier. This maximum water level is MSL + 1 meter as stated in the MARD-plan and MARD-plan-variant. According to the client, this level cannot be exceeded due to flooding. The minimum water level inside of the barrier is MSL - 0.6 m to ensure the navigability of the river and to avoid environmental issues.
The barrier however will be closed at a lower water level, then this MSL + 1 m, in the wet season, in order to have a sufficient buffer for rainwater in the waters around Ho Chi Minh City.
In the current situation pump stations are built to be able to drain water from the system, however in the coming designs, this option will be avoided. This is because pumping water consumes a lot of energy and is out of the scope of our project. During low tide it will be possible to let water flow from the inside to outside without the help of pumping stations.

2.3 Functional requirements

The main functional requirements for the barrier at Phu Dinh are summed up below:

1. The water level should be maintained at a height of maximum MSL + 1.0 m and minimum MSL – 0.6 m, on the city side of the barrier in the dry season.

When the water level is between these levels, the water in the river has to flow in and out freely for environmental purposes and navigation of class III ships should be hindered barely.

2. In the wet season the barrier closes below MSL + 0.6 m, water may not exceed MSL + 1.0 m.

The barrier has to be closed below MSL + 1.0 m in this case, to prevent floodings in Ho Chi Minh City, which above MSL +1.0 m. If rain is forecasted, the barrier might be closed below MSL + 0.6 m, so for instance around MSL itself, to have a bigger buffer capacity, where excess rain of Ho Chi Minh City can be stored.

3. Navigation must always be possible through the Phu Dinh area.

According to the client, when the barrier is closed, ships must still be able to pass the river section at Phu Dinh. So a navigational structure will be needed. When the water level is below MSL - 0.6 m, navigation is not possible for the design vessel. The water level can get lower in the wet season to prevent flooding. The flooding requirement is more important than the navigation requirement, according to the client, so it is expected that navigation is not always possible, but it should in those cases be hindered to a minimum.

4. The structure must have a lifetime of at least 100 years.

2.3.1 Boundary conditions

There are a few boundary conditions which have to be dealt with:

1. The site of the barrier is the first condition (coordinates: 10°42'38.3"N 106°37'14.2"E). The river section at the location of Phu Dinh has a maximum width of 66 m.

2. The design and construction must be executed by local companies.

This boundary condition will stimulate the economic growth of the area. The expertise and experience will increase among the people involved.

3. Vessels at the hydraulic structure that arrive first are allowed to lock first.

Whenever the capacity of the lock can be increased by a vessel taking over another vessel in the queue it is accepted however.

3. Conceptual design

The design of a hydraulic structure at Phu Dinh consists of several alternatives. These alternatives are presented in paragraph 3.2. After an explanation of the alternatives a multi criteria analysis (MCA) is executed in paragraph 3.3 which leads to the best alternatives. These alternatives will be continued with in the preliminary design phase, with a higher level of detail in chapter 5 and further.

3.1 Introduction

On beforehand it is not clear which type of hydraulic structure will fit best at Phu Dinh to fulfill the functional requirements. That is why the project team came up with several alternatives for the layout of a hydraulic structure. These alternatives are tested using a multi criteria analysis to come up with the best alternatives. In the conceptual design the focus is on the different combinations of main elements of the design.
Several possible barriers and several possible locks are selected. Also some 'out of the box' barriers and locks are listed in the conceptual design. The reference projects are in some cases used as a guideline. For other barrier types or lock types the knowledge, experience and creativity of the project team is used as a guideline.
The six possible chosen barrier types and six lock types are all combined to 36 alternatives. When an alternative does not fulfill the functional requirement or the function of the lock and barrier type is contradictory, they are displayed with a minus in Table 8. This criterion decreases the amount of alternatives from 36 to 17. The 17 remaining alternatives are judged in the multi criteria analysis. With this analysis the best alternatives are selected and worked out into more detail in the preliminary design.
In the preliminary phase, alternatives are created based on more detailed differences, though the basic layout is already selected in the conceptual phase. More detailed differences could be for example: gate type, dimensions of the structure, a pump station or no pump station and the quality of material are different in each alternative.

3.2 Variants

The following possible barrier types and lock types are obtained after investigation:

Barrier	Lock
Dam with culvert	2 locks
Tidal barrier	Falkirk lock
Inflatable barrier	Boat conveyor
Lift barrier	1 lock
Turning barrier	1 big lock over full width
No barrier	Ship lift

Table 7: Possible alternatives for the barrier and lock

All possible barriers and locks are combined to form 36 alternatives. If an alternative does not meet the functional requirement or there is a clear reason why the design is inappropriate, it is displayed with a red minus in table 8 and not taken into account anymore. The four main reasons why an alternative does not meet the functional requirements are:

- Navigation is not possible when the barrier is closed. (A)
- There is no barrier to equalise the water level difference.(B)
- The design of the lock and barrier are contradictory. (C)
- The longitudinal length of Phu Dinh is limited; the slope of the lock will be too high to serve as a suitable lock for navigational vessels. (D)

Table 8 below, shows all possible combinations of locks and barriers. When an alternative meets the functional requirement a green plus is displayed.

	2 locks	Falkrik lock	Conveyer boat	1 lock	1 big lock	Ship lift
Dam with culvert	+	− **	D −	+	C −	+
Tidal barrier	+	A −	D −	+	C −	+
Balgstuw barrier	+	A −	D −	+	C −	+
Lift barrier	+	A −	D −	+	C −	+
Turning barrier	+	A −	D −	+	C −	+
No barrier	+	B −	B/D −	B −	+ *	B −

Table 8: The different alternatives, marked if they fulfill the functional requirements or not

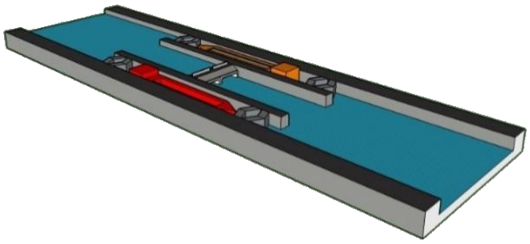
*One big lock can be used as a barrier when there are two barriers that can open in upstream and downstream direction. These barriers are also functioning as a gate of the lock.
** The height difference between the two water levels for a closed barrier, is not big enough to justify the use of such a complex structure at Phu Dinh

The 17 selected alternatives are shown paragraph 3.2.1. Each alternative contains a figure which visualises the alternative.

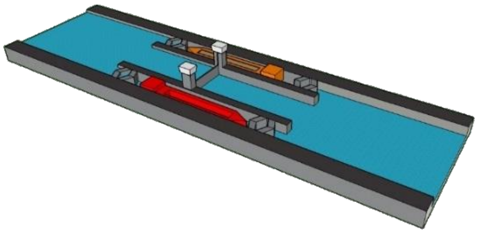
3.2.1 Description of the different alternatives

The 17 remaining alternatives have been visualised by the design team. These visuals of the different combinations of alternatives give a good overview of possible advantages and disadvantages per variant.

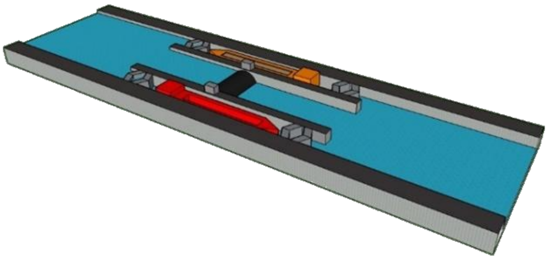
1. Dam with a culvert and 2 locks



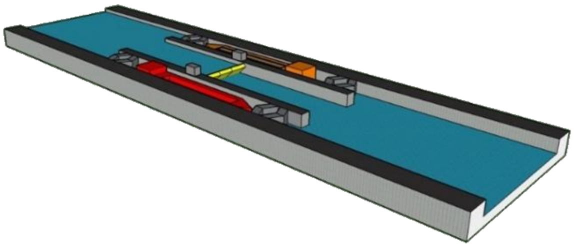
2. Tidal barrier with 2 locks



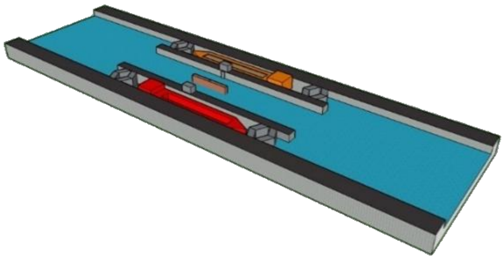
3. Balgstuw barrier and 2 locks



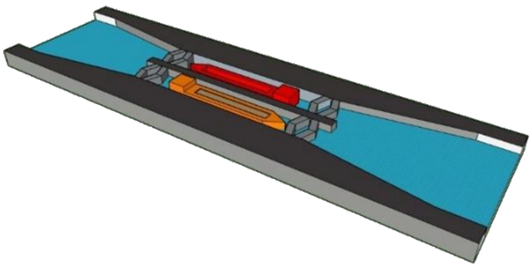
4. Lift barrier and 2 locks



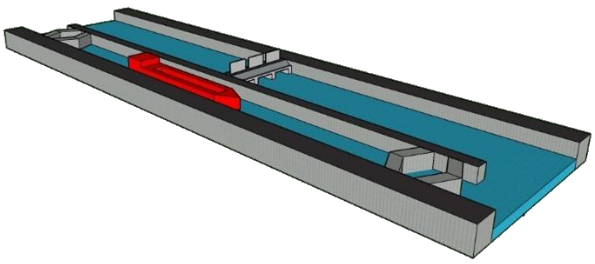
5. *Turning barrier and 2 locks*



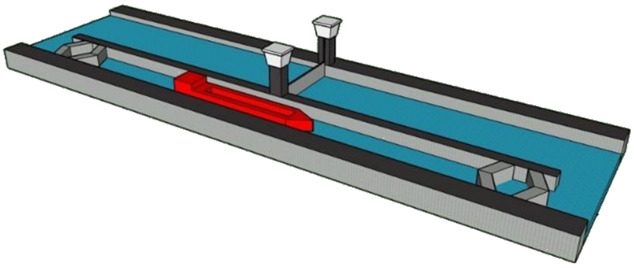
6. *No barrier and 2 locks*



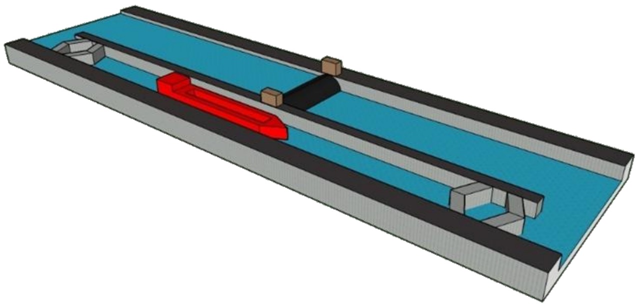
7. *Dam with culvert and 1 lock*



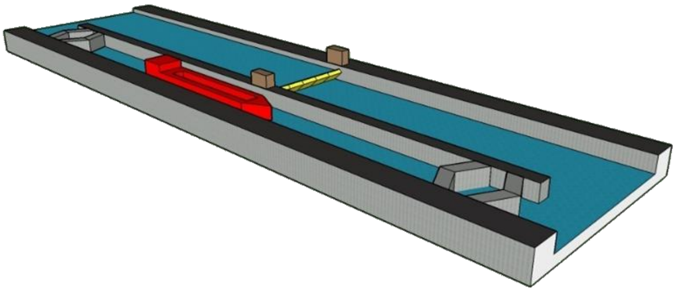
8. *Tidal barrier with 1 lock*



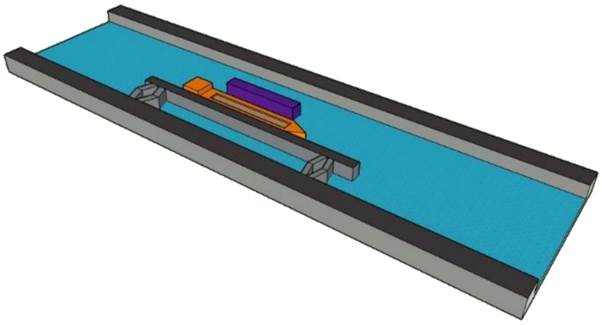
9. Balgstuw barrier and 1 lock



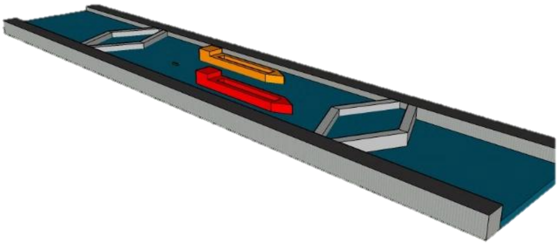
10. Lift barrier and 1 lock



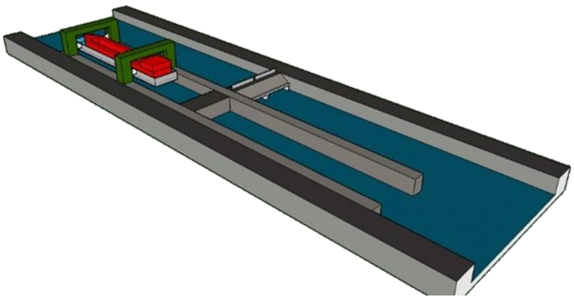
11. Turning barrier and 1 lock



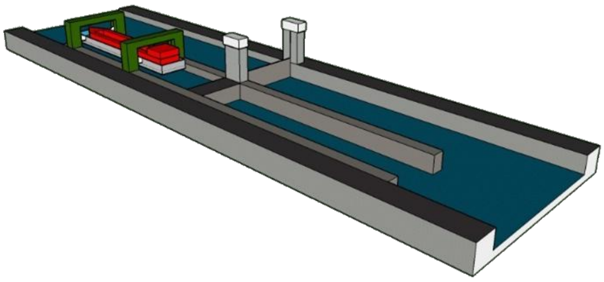
12. No barrier and 1 big lock



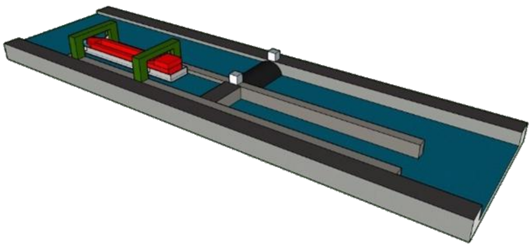
13. Dam with a culvert with a ship lift



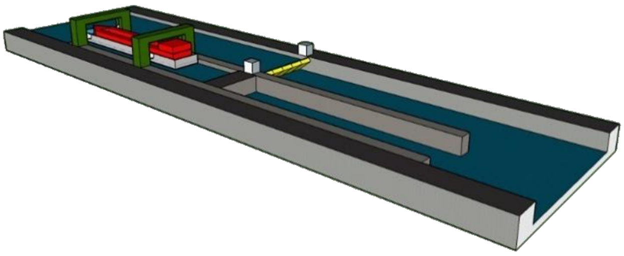
14. Tidal barrier with a ship lift



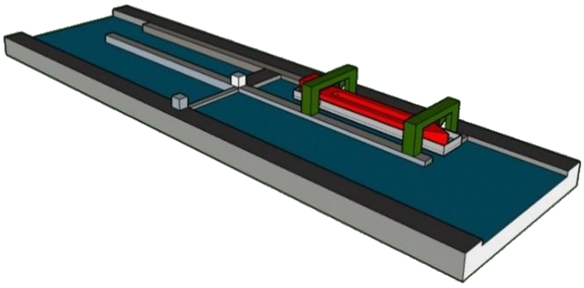
15. Balgstuw barrier with a ship lift



16. Lift barrier with a ship lift



17. Turning barrier with a ship lift



3.3 Multi Criteria Analysis

The different alternatives which are designed in the conceptual design should be evaluated to choose the best alternative for this project. In the first phase, the MCA will bring the amount of alternatives from 17 to a number of alternatives with the highest score. There should be a clear distinction in the final score between the selected alternatives and the other alternatives. Therefore different criteria are defined which are evaluated for each alternative and each alternative is given a certain score. These scores are multiplied by a weight factor because not every criterion shares the same importance. The weight factors are determined by means of a pairwise comparison which will be presented in this chapter. First the criteria for the multi-criteria analysis will be defined.

3.3.1 Criteria

There are five criteria which will be evaluated for every alternative. These criteria form the basis of the multi-criteria analysis. These five criteria are divided into subcriteria and a definition of every criterion and the different subcriteria is given below. The main criteria are:

- **Costs**
- **Durability**
- **Constructability & Technical feasibility**
- **Usability**
- **Environment/stakeholders**

Costs

This project aims at new, innovative and different alternatives than the current lock/barrier-complex so there can be quite a variety in costs for the different designs.
In most projects costs is a very important criterion, for this project this is not the case. The Ho Chi Minh City government is willing to invest in the best alternative. But the money that is invested in this project should not be thrown away to a creative but useless design.
The costs are used as total costs in the MCA, and they consist of:

Labour costs: Includes the costs of all labour during the entire project cycle.

Material costs: This includes all costs that are directly related to the construction of the structure.

Maintenance costs: These costs consist of:

- Frequency of maintenance
- Approachability when replacing parts/doors
- Approachability when doing inspections
- Anti-corrosion/erosion measures of the structure

Durability
The durability is subdivided into the durability of the structure and the lifetime of the structure.

The lifetime of the structure: is simply the timespan which the structure should survive. Parts can be replaced during this lifetime, but the main structure should remain during this period.

The durability of the structure: Consists of: two material characteristics and an energy consumption level:

- **Amount of material needed:** raw material/building equipment) gives an indication of the arising pollution with respect to the environment and the greenhouse effect. Lots of materials needed and lots of heavy equipment means a larger impact on the environment and the greenhouse effect.
- **Type of material needed:** mainly the distinction between concrete and steel (maybe also plastics). Concrete is more durable and lasts longer than (unprotected) steel.
- **The amount of energy needed to open and close the hydraulic structure:** Energy consumption does not only cost money, it also has a negative influence on the environment. So the more energy is needed to transfer a vessel in the lock or the more energy is needed to open and close the barrier, the higher the costs and harm to the environment.

Constructability and technical feasibility
Constructability and technical feasibility is important in countries in which there is few experience with large and special projects which are quite new in the construction business. Constructability and technical feasibility is divided into the experience and expertise needed, temporary structures and the complexity of the structure.

Experience/expertise needed: innovative solutions such as the Maeslantkering or the Venice Lagoon barrier need a lot of experience and expertise of the consultants and the contractor. The experience and expertise needed should (partly) be available in Vietnam to avoid high execution costs.
An innovative structure leads to little experience in the construction process. This increases the risk of failure.

Temporary structures: another aspect of innovative solutions often is the necessity of temporary structures which serve the construction of the main structure. There could be thought of construction platforms in the river, temporary extra construction space on the quays, the use of pontoons or pre-casting yards.

Complexity of the structure: complexity of the structure, placement and installation of the structure elements. For example construction and installation of very large quantities of elements or a single very large element can be considered more complex to construct than standard constructions such as sheet piles. Also working on water and in-situ concrete works are examples of work which are considered more complex and these complex structures can cause additional cost.

Usability
The usability of the structure describes the behavior and functioning of the structure after the construction phase. The usability will be tested for the navigation function of the hydraulic structure and the retaining/regulating function of the structure. Also the operability of the structure will be mentioned.

Navigation function: the navigation function describes the extent to which the structure is able to process the vessels which sail through Phu Dinh.

Retaining/regulating function: the retaining/regulating function describes the extent to which the structure is able to retain high tidal water levels from the sea and to regulate the water levels in times of high river discharges and rainy periods.

Adjustability of navigation function: when ships become larger in the future at Phu Dinh, or when the intensity is growing due to more port activities in the region, it is important that the hydraulic structure at Phu Dinh allows this growth of navigation.

Adjustability of retaining/regulating function: due to sealevel rise and heavier periods of rain, the hydraulic structure at Phu Dinh likely has to cope with more extreme water levels in the future. The structure should be able to cope with these water levels without extremely large adaptions of the design.

Operability
The operability of the structure describes the risk of failure and the user friendliness of the structure. Standard hydraulic structures which are familiar in Vietnam are easy to operate for the personnel but new, innovative solutions can be hard to operate and need a lot of training of the personnel.
When during operation the structure fails (operational and structural) the consequences can be extremely disastrous for Ho Chi Minh City. Lives could be lost and a lot of damage can occur. So the risk of failure of the hydraulic structure is an important aspect during the design and operability.

Environment/stakeholders
The hydraulic structure at Phu Dinh will be constructed in a densely populated area without much free space. Any change in the environment will affect stakeholders and the aim is to satisfy the stakeholders as much as possible. For the three main stakeholders, the extent to which their interests are satisfied will be investigated.

Satisfies stakeholder 'Municipality Ho Chi Minh City': the main interest of this stakeholder is to prevent flooding and to improve the water quality.

Satisfies stakeholder 'local people': the main interest of this stakeholder is to be involved into the project.

Satisfies stakeholder 'port activities': the main interest of this stakeholder is to be a competitive waterway

Aesthetics/landscapes: hydraulic structures are often very large structures which have a big influence on the view of the surroundings. Also hydraulic structures can be pollutive for the environment, these two aspects can decrease or possibly increase the value of the landscape.

3.3.2 Weight factors
Weight factors are given to each of the five main criteria described above. This is done because not every criterion has the same importance and the same influence on the outcome of the MCA. The determination of the weight factors is done by the project group in collaboration with the client. The client explains his wishes and most important evaluation criteria. The project group advices on the different evaluation criteria and the weight factors.

The weight factors are determined using a pairwise comparison method. The pairwise comparison helps with the determination of the relative importance of each criterion in a pairwise comparison matrix. This matrix is used by members of the project team (4 outcomes) and by the client (averaged outcomes of mr Van and mr Cuong). The outcomes of the 6 pairwise comparisons are averaged to obtain the final weight factors for the MCA. The final weight factors can be found in Table 9.

	Louwrens	Marc	Martijn	Michel	Mr. Van & Cuong (2x)	Average
Costs	14%	19%	19%	17%	19%	18%
Durability	24%	19%	16%	17%	19%	19%
Constructability	11%	11%	13%	25%	11%	14%
Usability	32%	33%	33%	29%	29%	31%
Stakeholders	19%	19%	19%	13%	22%	19%

Table 9: Weight factors for the MCA

An explanation of the exact method for determining the weight factors using the pairwise comparison can be found in Appendix D.

Sub-weight factors
The five main criteria are divided into subcriteria to specify the main criteria. Every subcriterion has got its own weight factor, just like the main criteria. The determination of the sub-weight factors is based on expert judgement, reference projects and comparison of the sub-criteria.

3.3.3 MCA table

The MCA table, Table 10 is filled out by every project member individually. A score is given for every sub criterion for each alternative between 1 and 5. The score of 1 means that the alternative scores badly on the concerning criterion and a score of 5 means that the alternative scores well for the concerning criterion. All the scores are then multiplied by the weight factors and all the scores are summed up to achieve the total score.

								Alternatives												
Main criteria			Sub-criteria	Weight	Importance	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1. Costs	18%	1.1.	Total costs	1,00	18	2,75	1,75	2,75	2,00	2,25	3,75	4,75	3,25	4,50	3,25	3,00	2,00	2,75	2,00	2,75	2,00	2,00
2. Durability	19%	2.1.	Durability of structure	0,33	6,33	3,50	3,25	3,50	3,00	2,75	3,50	4,00	4,00	3,50	3,50	3,50	2,75	2,25	2,25	3,00	2,50	2,50
		2.2.	Lifetime of structure	0,67	12,67	4,25	3,50	1,75	2,75	3,25	4,00	4,50	4,00	2,00	3,00	3,50	3,50	2,75	2,75	1,75	2,00	2,50
3. Constructability	14%	3.1.	Expertise/experience	0,50	7	4,25	3,75	2,00	2,00	1,75	4,00	4,50	3,75	2,25	2,00	1,75	1,75	2,25	2,00	1,75	1,50	1,50
		3.2.	Temporary structures	0,33	2,33	3,25	2,75	2,25	2,00	2,50	2,25	3,25	2,75	2,25	2,50	3,00	1,75	3,25	2,75	2,50	2,25	3,00
		3.3.	Complexity of structure	0,17	4,67	4,00	3,50	2,25	1,75	2,50	4,00	4,75	3,25	2,50	2,25	2,50	2,25	2,00	2,25	2,25	1,75	2,00
4. Usability	31%	4.1.	Navigation function	0,35	9,3	3,50	4,75	4,75	4,75	4,00	3,50	1,75	3,75	3,50	3,75	3,50	3,75	2,00	3,25	3,25	3,25	2,75
		4.2.	Regulating function	0,35	10,85	2,00	3,75	3,25	3,50	3,75	2,75	2,50	4,25	4,25	4,50	4,00	3,50	2,25	3,25	3,50	3,50	2,75
		4.3.	Adjustability navigation function	0,05	1,55	2,25	2,25	2,25	2,25	2,25	1,50	3,00	2,75	3,00	2,75	3,00	1,50	1,75	1,75	2,00	2,00	1,75
		4.4.	Adjustability regulating function	0,10	1,55	1,75	2,25	3,50	3,25	3,50	1,25	2,50	2,50	3,50	3,50	3,25	1,50	2,50	2,00	2,75	2,25	1,75
		4.5.	Operability	0,15	7,75	3,75	3,50	2,75	3,00	3,00	4,00	4,25	4,00	2,75	2,75	3,50	3,25	2,25	2,50	2,75	2,50	3,00
5. Stakeholders	19%	4.1.	Satisfaction Ho Chi Minh City	0,43	6,33	2,50	3,50	3,25	3,25	3,50	3,00	2,75	4,00	3,25	3,50	3,75	2,75	2,25	2,75	2,75	2,75	2,75
		4.2.	Satisfaction local people	0,14	2,11	2,75	2,75	3,50	3,50	3,25	3,25	3,25	2,75	3,50	3,75	3,25	2,75	2,25	2,25	2,50	2,75	2,25
		4.3.	Satisfaction port activities	0,29	4,22	3,50	4,25	4,00	4,00	4,00	3,25	2,00	3,00	3,00	3,50	3,25	3,75	2,00	2,75	3,00	3,00	2,75
		4.4.	Aesthetics/landscape	0,14	6,33	3,25	2,50	3,00	3,75	3,25	3,50	3,75	2,50	3,25	4,00	3,50	3,50	1,75	1,50	2,00	2,25	1,75
				Total		327,99	326,10	297,07	299,17	305,35	351,63	370,40	362,07	332,40	333,86	330,64	288,47	236,79	249,93	264,12	245,36	239,26

Table 10: MCA completed

3.4 Selected alternatives

It can be deduced from the MCA, which alternatives are most useful to be continued with in the preliminary design. There are 3 alternatives that score significantly higher than other alternatives. This are numbers 6, 7 and 8 (marked green in the table above). These alternatives are: 2 locks without a barrier, 1 lock with a dam and culverts and 1 lock with a tidal barrier. These alternatives are chosen and will be further developed in the preliminary design phase.

4. Main dimensions of hydraulic structures

The main criteria to which the variants are designed are: the required navigability allowance and the water conveyance/regulating capacities. The navigability and the regulating function of the structure are directly influenced by the lock dimensions and the conveyance width of the regulating structure.

4.1 Lock dimensions

This sub-chapter determines the dimensions of the lock. This is obtained by computing the gathered ship intensity data into usable data for the cycle time of a lock. The cycle time will be analyzed and from this, the capacity of the lock is determined. In paragraph 2.2.9 it is concluded that there is a correlation between the peak intensity and high tide. So during high water, the peak intensity should be able to pass the lock. On the other hand the lock should not be over dimensioned to remain cost efficient.

4.1.1 Lock data

During six months the frequency is measured of the passing loaded and unloaded vessels at Phu Dinh. In Table 11, the second column shows the sum of the loaded and unloaded vessels that occur per class. The fourth column shows the amount of tonnage that a loaded vessel transports on average for this class. This is derived by the amount of annual tonnage per class divided by the occurrence of loaded vessels in that class.

Ship classes	occurrence in 6 months	% occurrence	Tonnage average	Tonnage x occurrence
>500 DWT	5760	18	839.2	148
301-500 DWT	3869	12	400.1	47
201-300 DWT	3361	10	250.3	26
101-200 DWT	3537	11	149.9	16
51-100 DWT	4464	14	75.0	10
5-50 DWT	11622	36	25.1	9
TOTAL	32613	100		256

Table 11: Ship occurrence and tonnage, over 6 months at Phu Dinh

As stated in paragraph 2.2.9 in august 2016 there were 6 days of almost continuously counting at Phu Dinh. The transport is these 6 days is counted for eight different categories:

- Canoes
- Medium boats
- Big boats
- Barges 100 DWT-200 DWT
- Barges 300 DWT-400 DWT
- Barges 500DWT-700 DWT
- Barges 700 DWT-1000 DWT
- Barges > 1000 DWT

To calculate the lock dimensions the canoes are neglected due to their small dimensions and low frequency of passing Phu Dinh, they can easily fit in free space left when locking other ships. The medium boats coincide with the ship classes from 5 DWT - 500 DWT, this can be interpreted from the occurrence of medium boats during the six days and the occurrence of boats between 5 DWT and 500 DWT during the six months. The average DWT for the medium boats is 200 DWT. The same reasoning holds for the big boats who coincide with the >500 DWT boats in the table above. A few assumptions are done to calculate the cycle time of the lock:

- Barges larger than 700 DWT are not allowed to pass through the lock, due to their dimensions and low rate of occurrence.
- The calculations in this chapter are conducted with mitre gates, if in a later stadium of this project another type of gate is chosen this can be changed easily. It will affect the cycle time very little due to a small difference in operating time of different lock doors.
- The initial design vessel of 600 DWT with dimensions 122m x 9.5m x 2.5m has not been measured during the six days in august 2016. This is a special type of boat due to its high length and relative small width and draught. In consultation with the client it has been decided that this vessel does not have to go through the lock. The new dimensions for the design vessel are 66.5 m x 9.7 m by 2.5 m.

The dimensions of the five categories that have to pass through the lock are interpolated from a provided table by the IWER. The table from the IWER can be found in appendix G, the interpolated measurements are displayed in Table 12.

DWT (T)	Vessels		Barges		
	840	200	100 - 200	300 -400	500 - 700
Length (m)	66.5	34	27	41	40
Width (m)	9.7	6.6	6.4	11.2	11
Draught (m)	2.5	1.7	1.0	1.3	2.2

Table 12: Interpolated ship dimensions at Phu Dinh

4.1.2 Variables needed for cycle time

According to the lecture notes of Ports and Waterways of the TU Delft (Groenveld, Verheij, & Stolker, 2006) the cycle time can be determined by adding up the time used for entering the lock, the operating time and the time for leaving the lock. To calculate the cycle time a lot of variables are needed. These variables are listed in Table 13.

Variable	Unit	Value	Comment
Length	m	160	Initial assumption
Width	m	22	Initial assumption
Height	m	6.42[1,2]	This is 7.12 m - storm condition (0.7 m)
Hmax	m	2.92[1,2]	See Figure 7; 3.62 m - storm condition
Ok	m^2	3520	Length x width
As	m^2	5	Assumed according to Groenveld et al. (2006)
m	-	0.9	Assumed according to Groenveld et al. (2006)
g	m/s^2	9.81	
laden	%	59.2	Vessels passing Phu Dinh
Ak	m^2	144.3	Width x height
unladen	%	40.8	Vessels passing Phu Dinh
Loop distance	m	300	Assumed according Groenveld et al. (2006)
Loop time, (T_l)	min	1.1	Figure 4.22 Groenveld et al. (2006)
Empty/filling time (T)	min	10.1	Depends on: Ok, Hmax, m, As, g

Table 13: Variables needed to calculate the cycle time of a lock

1) The height of the lock has to be at least 7.12 meter as can be seen in Figure 10 on the right. The lock has to be able to operate at minimum water level, this is MSL -0.6 m. The threshold depth, which is the draught of the vessel (2.5 m) + 0.4 m (keel clearance) = 2.9 m. This has to be added to the minimum lock depth needed. On the other hand the lock has to be as high as the barrier in case of high water levels to prevent flooding over the lock.

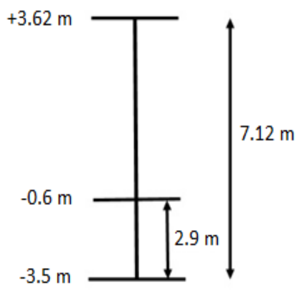


Figure 10: The different water levels for the lock

2) Two assumptions are done in the lock design.

- During storm conditions (0.7 meter) there is no lock operation due to a high possibility of collision. So the maximum water level difference is 3.52 meter instead of 4.22 meter (MSL + 3.62 m+ MSL - 0.6 m).
- When there is a highest water level on the outside of the barrier of 3.62meter, the water level inside is higher than MSL – 0.6 meter due to rain on the inside, it is assumed that the water level will rise until 0 meter due to bad weather. So the water level difference during locking decreases further from 3.52 meter to 2.92 meter. So in practice the variables: maximum lock height and Hmax is 1.3 meter lower than in Figure 10. Though this does not mean that the height of the lock itself is 1.3 meter lower, it is just the maximum height of a possible locking operation.

4.1.3 Cycle time

To determine the cycle time, a length, width and height are assumed keeping in mind the ship dimensions. With the method, discussed in this paragraph the cycle time can be determined. When the cycle time is known the capacity of the lock can be determined. The dimensions and intensities of the vessels and barges are known, so it can be verified whether the capacity is high enough to deal with the (peak) intensity. If the capacity is to low, the dimensions of the lock have to increase and an iteration is conducted. The aim is to achieve a cycle time lower than 60 minutes, to make the lock useful relative to the barrier. Another important aspect is that the queue will result in too much waiting time.

4.1.3.1 Cycle time of first assumption

The length, width and height of the lock are assumed to be 160m x 22m x 6.42m. With all the variables known from Table 13 the last variables that are depending on the dimensions can be determined

- The maximum number of ships (N) that the lock chamber can accommodate is determined to be 17.
- The operating time (T_b) for the mitre gate is 4.1 minute when the chamber width is between 16 and 24 meter.
- The time for laden and unladen vessels for entering (T_i) and leaving (T_u) the lock is determined from figure 4-20 and 4-21 of the Capacities of Inland Waterways book (Groenveld, Verheij, & Stolker, 2006). This is done for the direction from low to high and for the direction high to low water. The average of T_{ia} and T_{ua} is used in the formula.
- The last ship sailing out plus the average time for entering of the first ship is the loop time (T_l) used in the formula, when the direction is to lower reach, the time for entering the high reach is taken into account.

The locking duration in the direction of the lower reach (T_d) and upper (T_u) reach is calculated with the same formula, although the values are different for both directions. An example of this calculation is provided with the first assumption of lock dimensions. This formula is:

Lock duration to lower reach (T_d) = $T_l + (N-1)*T_{ia\text{-}high}+T+T_b+N*T_{ua\text{-}low}$

T_d= 2.58+ (17-1)*1.48+10.1+4.1+17*0.96= 56.7 minutes

Lock duration to upper reach (T_u) =2.80+(17-1)*1.70+10.1+4.1+17*0.88= 59 minutes

The total cycle time is the sum of the locking duration upstream and downstream = 56.7+59= 115.7 minutes.

4.1.4 Capacity 1 lock

4.1.4.1 Dimensions: 160m x 22m

Maximum number of ships in lock

The maximum number of ships that can be locked, with the given cycle time of 115.7 minutes is 17.64 ships per hour. This is the maximum, which means that the average should be below this, to have a sufficient lock capacity. The maximum amount of ships that is measured in 1 hour, following the given data, is 12.36 ships per hour. This means that maximum lock capacity is well above the needed capacity, according to the calculation.

Maximum ship intensity

A problem that arises however, is that the ship distribution is not constant in time. For this reason it is decided to test the lock analytically, to see if the lock can handle the maximum measured ship intensity, especially in the future when a higher water head difference will be present.

Using the lock dimensions that the design team came up with, the lock surface became 22 m by 160 m. Using the data of the measured ships at Phu Dinh, for each ship a design area is matched with its area of water that it needs. This provides a very rough method to determine if the lock area is big enough to lock most of the ships. The peaks of area needed, occur at the moments that most ships are present in the canal at Phu Dinh. These moments, as shown in chapter 2.2.9, occur around high water, which means that most of this traffic will have to go around a closed barrier, so through the lock.

The maximum required area that is needed, occurs in one of the hours measured and it has a maximum of 25 ships. The required area is at least 8730 m^2, while only 3520 m^2 is available in one direction. The maximum locking cycle will take 116 minutes. This means that ships can be moved in the upstream and in the downstream direction. This means that the total locking area that is available is 7040 m^2 per 2 hours. This is however too small to accommodate all the ships in the maximum hour, especially if it is taken into account that there also has to be room in between the different ships. This leads to the conclusion that the lock chamber will have to be bigger or the cycle time much lower.

4.1.4.2 Dimensions: 220 m x 22 m

To make the capacity of the lock bigger, the full design process is executed again, but now for a lock of 220 m in length. Almost all the parameters are the same as for the first calculation, however the N_{max} becomes bigger and will be 24 ships.
The lock cycle for this lock becomes even bigger and is 160 minutes. The maximum amount of ships per hour through the lock is 18. The lock cycle thus becomes much longer, while the amount of ships per hour, does not increase significantly.

The cycle time is much longer than one hour, in which the different ships arrive. This means that a waiting queue will form, this will result in long waiting times for the ships in the queue. To make sure that locking is useful, it has to ensured that the locking ships, does not take longer than waiting for the tidal gate to open. This means that choices will have to made for the maximum waiting time and the ships that are allowed through the lock. This has to be done, in cooperation with the client.

4.1.4.3 Dimensions 111m x 21m

The final lock design is a compromise between a cycle time of about 1 hour and optimal lock dimensions to fit the most ships optimal in 1 cycle of the lock. After a discussion with the client it is decided to focus on a lock with a waiting time of about 1 hour. Using the dimensions of 111 m in length and 21 m in width, the cycle time is determined to be 68 minutes. This leads to 15.2 ships per hour.
Using the lock dimensions, the ship sizes and the margins of 3% ship length in the longitudinal direction and 1% of the ship width in the transverse direction (Capacities of Inland Waterways, 2006, p. 45) the different combinations that fit in a lock are determined. This is shown in Figure 11.

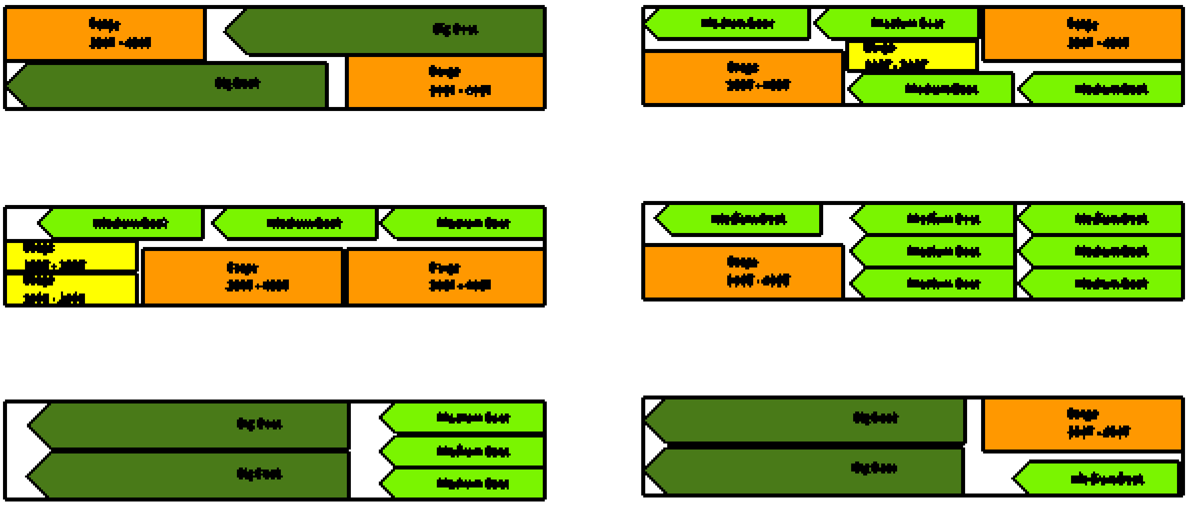


Figure 11: The 6 possible combinations for the lock, the 300T-400T barges are interchangeable with 500T-700T barges

Canoes are neglected and Barges bigger than 700T will have to pass the barrier when the tidal barrier is open. All the other boats are spread into different categories:

1. Medium boats, L = 34 m B = 6.6 m, D = 1.7 m & weight of 200 T
2. Big boats, L = 66 m B = 9.7 m, D = 2.7 m & weight of 840 T
3. Barge 100T-200T, L = 27 m B = 6.4 m, D = 1.0 m
4. Barge 300T-400T, L = 41 m B = 11.2 m, D = 1.3 m

4.1.4.4 Waiting time

Using an analytical approach, the ship intensity data is used to predict the maximum waiting time, for ships that want to pass the barrier when it is closed. The maximum waiting time is 5 hours. This is right after the peak moment, but assuming that the ships arrive in the most convenient order.
This assumption can be gratified, because in practice the lock cycle will be shorter than 1 hour. This is due to the fact that a higher maximum water level difference is taken into account as a parameter, which in practice will not occur for the locking procedure. An average value for the water level difference will occur more frequently and this is lower than the maximum water level difference which leads to a lower cycle time.
So in the worst case scenario, a waiting time of 5 hours will occur. In this case, the lock will operate at full capacity, till the barrier is opened again, then the waiting queue will disappear. There is no criterion for the maximum waiting time and according to the client, in the future, the ship intensity will go down. That is why the design team agrees with this maximum waiting time. The analytical approach conducted in excel could be shown by the design team on request.
However, if a distinction is made between the ships that are and are not allowed through the lock, the waiting time can be reduced. By prohibiting the barges of 500T – 700T from passing the lock, the maximum waiting time becomes 4 hours for one big ship. So overall the maximum waiting time will be around 3 hours. This makes the lock more efficient, relative to the closed time of the barrier, which is around 5 hours. So this could be a good solution to prevent the waiting time from becoming too big of a factor.

4.1.5 Capacity of 2 locks

For a design with 2 locks, the previous designed lock is used first. For the second lock, a smaller and faster lock is used. The design ship for this lock is 1 big boat. The surface area for the second lock becomes 70 m by 10 m. Big barges cannot pass this lock, only 1 big boat, 2 medium boats or 2 small barges or any combination of one medium boat and one small barge.
The cycle time is calculated, using the method shown before and is 29 minutes. This is equal to 10.5 ships per hour.
This cycle time means that the lock can lock up and lock down twice per hour. By using the same analytical method as before, different combinations are tried for each hour, to lock the maximum number of ships. Using the given ship intensity of the maximum day, the results are that ships will never have to wait more than one hour, when a combination of 2 locks is operating. Small barges will also never have to wait more than one hour, the medium barges of 300T – 400 T can be given priority over the biggest barges, which means that they also will never have to wait more than one hour. In the case of maximum ship intensity in the canal at Phu Dinh, there will be one big barge of 500T – 700T that will have to wait a maximum of 3 hours, all other waiting times will be shorter.
Considering that the intensity will go down in the future, according to the client, these waiting times are acceptable according to the design team.
So the proposed lock dimensions for the 2 lock design are:

1. Big lock, with an area of 21 m by 111 m and a depth of at least 6.42 m
2. Smaller lock, with an area of 10 m by 70 m and a depth of also at least 6.42 m

With this combination, large barges will have the longest waiting time, using the given arrival time, they are however not forbidden to pass, but these barges can only use the big lock.

4.2 Water flow and conveyance area

In this chapter the conveyance area for different combinations of locks and barriers is investigated, to see if the outflow can be high enough to get rid of all the water in the small amount of time that this is possible.

4.2.1 Matlab Script

To design the barrier it is important to have a general idea about the water flow through the barrier. A Matlab script is set up in order to calculate those flows. This has two goals, firstly to look at the flooding probability in different rain situation and secondly to look at the limits of navigation.
For this script the following input is used:

- Conveyance width and depth of the gate;
- The tide outside of the barrier;
- The normative rains and the area over which it should be discharged with this barrier;
- The relative storage area for this barrier;
-

The following requirements apply:

- The barrier closes at MSL + 1.0 meter and MSL - 0.6 meter in the dry season
- The barrier closes at a maximum of MSL + 0.6 in the rain season;
- There is no navigation possible when the flow speed of the water is too high.

The script uses the following equation which is elaborated in appendix F.A.

$$\frac{dh_{in}}{dt} = \frac{\left((q_{rain}(t) * A_{rain}) - (v(t) * B_{width} * \ h_{in}(t))\right)}{A_{storage}}$$

This differential equation is used to calculate the water levels over time, using the Runga Kutta method. Using the calculations of the script, various data about the flow speed, water levels and navigation availability over time are found.

4.2.2 Flooding

In appendix F.A an analysis of the flooding probability is conducted. This analysis is hard to perform accurately because a lot of different factors influence the probability. Therefore the uncertainty level is quite high. Still this analysis is useful to get a general understanding and feeling about the flooding probabilities.

Based on peak rain intensities the flooding probability in the current situation – with a unlimited conveyance width – is discussed, leading to an estimated risk of flooding once every 500 years when the barrier is used correctly. When looking at the future, a sea level rise of 0.7 meters is taken into account. In this situation the probability of a flood becomes a lot higher. Every 12.5 years there will be a peak rain situation which will lead to a flood if it occurs at the worst possible moment. In this case the probability that a peak rain which does not occur on the worst moment, still leads to a flooding also becomes higher. Therefore the estimated probability of a flood becomes once every 20 years if the sea level rise is taken into account.

Following it is discussed that the conveyance width of the barrier influences the flooding probability. When this width is lower than 10 meter it will delay the water level on the inside of the barrier in such a way that it becomes unacceptable. Between 10 and 30 meters the delay is still serious and has to be dealt with during the design. Above 30 meters the width has almost no effect on the flooding probability.

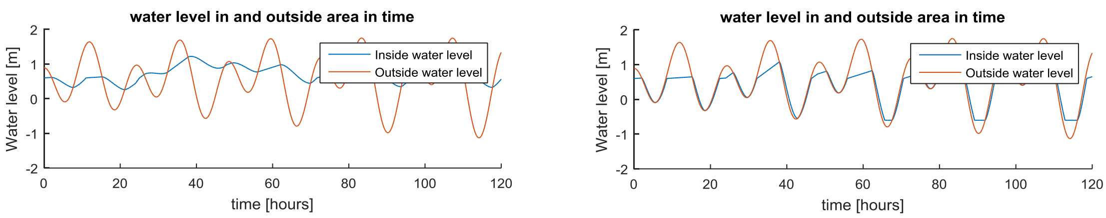


Figure 12: Effect of varying the conveyance width as shown in the situation at 5 meter (left) and 60 meter (right).

4.2.3 Navigation

The flow speed of the water limits the navigation in certain situations. When the barrier is closed and when the water flow speed becomes too high, navigation is not possible. Therefore an analysis is done regarding the time the barrier is not usable and the locks have to be used. This analysis focuses on the wet season because this will be normative for the design. In a later phase the results of a specific design can be calculated for both the dry and wet season. An elaboration of the analysis can be found in appendix F.A.
The analysis considers maximum flow speeds during navigation of 1.5 and 2.0 m/s. The width of the barrier is varied between 5 and 60 meters. Resulting in the graphs shown in Figure 13.
It can be concluded that the ideal width of the barrier is reached at 40-45 meters if the 2.0 m/s speed is normative. In case of 1.5 m/s as a limit for navigation, the width is ideal when it reaches 60 meters. When built large enough the barrier will be available for navigation half of the time. The other half of the time, the locks must be used to make navigation possible.

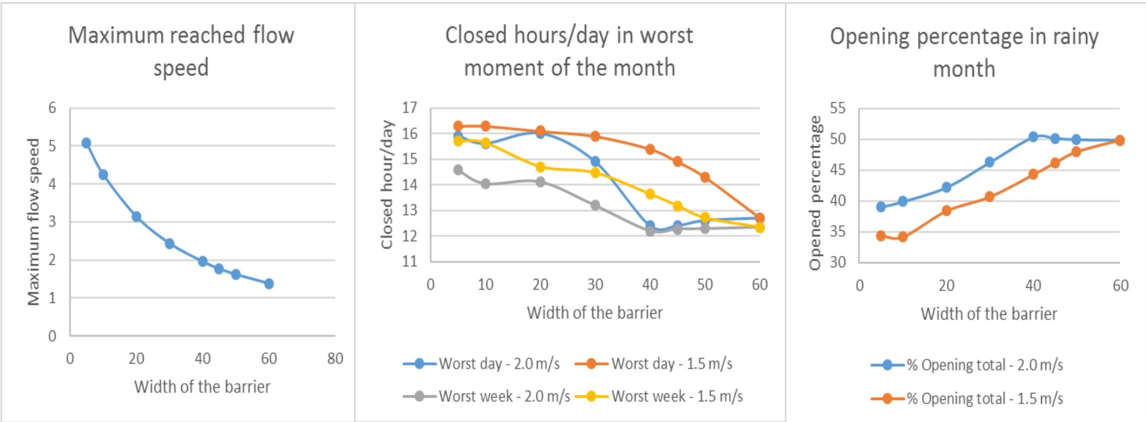


Figure 13: Results analysis of the navigation availability of the barrier.

4.3 Conclusion

From the above investigations it can be concluded that variant 8 (navigation lock with tidal barrier) is very suitable for the project concerning the navigability and the water conveyance capacity. From now on, this variant will be variant B. For variant 6 and 7 however this is not the case. variant 6 (Two locks, no barrier) seems unfit for this project as it does not meet the conveyance and regulating requirements. If the locks would be opened the water velocities would exceed the allowed maximum, making shipping impossible and if closed they would not convey enough water. Variant 7 (One lock with dam with culverts) however could meet the conveyance requirements but has a problem concerning its navigability. In order to solve these problems of Variant 6 and Variant 7 the decision has been made to combine them into a single variant, namely the variant with two navigation locks and a dam with culverts. From this point on, this variant will be called variant A. This leads to two variants that will be investigated in the preliminary design instead of three.

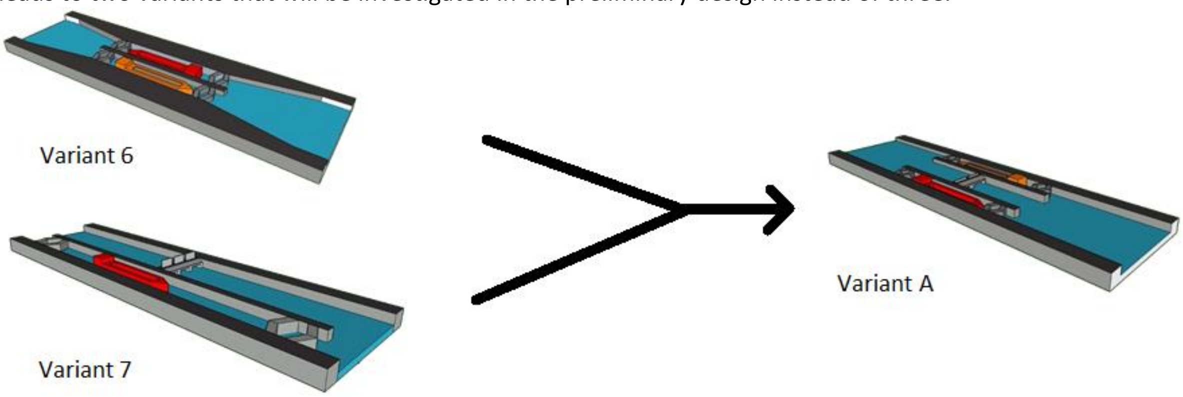


Figure 14: Two variants combined into one.

5. Alternative A

Alternative A consists of a dam with culverts in combination with 2 navigation locks. 1 lock is approximately two times larger than the other lock and the two locks are located next to the embankments. Between these two locks in the middle of the river, a dam with culverts is located.

5.1 Gate selection

A lot of different gate types can be chosen for the design of locks and barriers, depending on the layout of the structure and the environment. In section 3.2 and appendix C, ten types of gates are described which can be used for the hydraulic structure. Not all ten gate types are suitable for the design of this project because every gate type has its restrictions which may cause unsuitability for the situation at Phu Dinh. A preselection of these ten gates is therefore made based on expert judgement and restrictions coming from the descriptions, advantages and disadvantages as presented before. Only the gates which are really suitable will be evaluated in the MCA.

5.1.1 Preselection gate type for the locks

Four main restrictions can be found which may cause unsuitability for the type of lock gates used in the locks in alternative A:

1. The span which is needed for the lock gates at Phu Dinh is too large for the gate type
2. Additional horizontal space in the quays is required to accommodate the structure
3. The operating time is too high which increases the lock cycle time
4. A high level of experience and/or expertise is needed for the concerning gate type

The gates will be tested on suitability based on these four restrictions. In Table 14 a red X indicates that the gate type is definitely unsuitable at Phu Dinh. A green X indicates that the gate type may be unsuitable at Phu Dinh but this depends on the situation.

	Res. 1	Res. 2	Res. 3	Res. 4
Double mitre gates[1]				
Single leaf gate	X			
Lift gate				
Submersible gate				X
Rolling/sliding/caisson		X		X
Radial/sector gate		X		
Radial/tainter gate				
Inflatable weir		X	X	X
Flip-up barrier			X	X
Rotatable weir				X

Table 14: Preselection gate type for alternative A: dam with culverts + 2 locks

After this preselection three lock doors remain for the big lock and the small lock in alternative A: double mitre gates, lift gates and tainter gates.

5.1.2 Scia investigation

The goal of this investigation is to create insight into the amount of steel needed for the different gate types and into the way the forces caused by the water pressures are transferred from the water through the gates to the supports, for the different doors of the preselection.

Gate types for the different structures

As previously mentioned, there are three possible doors for the small navigation lock and large navigation lock of variant A. For the small navigation lock the only gate type that is assessed is the double mitre gates, for the large lock a mitre gate, vertical lifting gate and a radial gate are investigated. For the mitre gates in the large

[1] Single mitre gates are able to retain water in one direction. Double mitre gates are able to retain water in two directions.

lock various alternatives have been made altering the complexity of the structure and the amount of steel required.
In this section all the gate types have been developed and a rough estimate using the program SCIA Engineer has been made. Concerning the mitre gates two different gates have to be dimensioned for each structure, one in order to retain the water from the outside and one to retain the water from the inside:

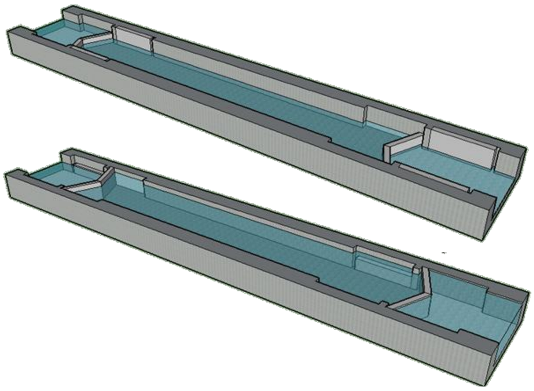


Figure 15: Different water levels that have to be retained by the mitre gates

The illustration above displays the governing water levels the navigation locks will have to cope with. The situation for higher inner water results in much lower water levels and thus forces, therefore the gates retaining the inner water can be dimensioned much smaller than the gates retaining high outer water. This holds for both the dimensions of the profiles used in the structures as the height of the gates, during high outer water the top of the smaller gates can be underwater. The gates other than the mitre gates have to be designed to be able to resist water pressures from both sides. The SCIA designs are found in Appendix H

Findings and concluding remarks

For the large navigation lock it is found that concerning the mitre gates a lot of steel can be saved if the water is regulated appropriately in the lock (maximum 2,5m water height difference per gate) and if the cross beams are optimized. The difference between the least and most optimized design comes down to a 30% reduction of steel used. This optimization however does lead to a more complex structure concerning the amount of profiles used and the amount of welding needed. Furthermore, the radial gate and the vertical lifting gate both have very similar amounts of steel needed. The following tables give an overview of the gates for the various structures:

Small navigation lock:

Mitre gate: High outside water gate		**Mitre gate: High inner water gate**	
Water difference	3.62m	Water difference	3.5m
Amount of steel used	131KN	Amount of steel used	57KN
Amount of profiles	2	Amount of profiles	2

Table 15: Summary of Scia results for the small lock

Large navigation lock:

Mitre gate: High outside water gate Alternative 1		**Mitre gate: High outside water gate Alternative 2**	
Water difference	3.62m	**Water difference**	3.62m
Amount of steel used	421KN	**Amount of steel used**	404KN
Amount of profiles	2	**Amount of profiles**	3
Mitre gate: High outside water gate Alternative 3		**Mitre gate: Truss variant**	
Water difference	2.5m	**Water difference**	2.5m
Amount of steel used	362KN	**Amount of steel used**	291KN
Amount of profiles	2	**Amount of profiles**	4
Mitre gate: High inner water gate		**Vertical lift gate**	
Water difference	3.5m	**Water difference**	3.62m
Amount of steel used	111KN	**Amount of steel used**	294KN
Amount of profiles	2	**Amount of profiles**	8
Radial gate			
Water difference	3.62m		
Amount of steel used	301KN		
Amount of profiles	7		

Table 16: Results for different gates for the big lock

5.1.3 MCA: Final selection gate type

After the preselection and the Scia investigation, the best gate option for the two locks in alternative A has to be determined. A MCA is used to select the best option. This MCA consists of 7/8 criteria with different weight factors. First, the eight criteria will be discussed briefly to clarify what they stand for. After this discussion, the criteria will receive scores which are multiplied by their weight factor to obtain the best gate type. The weight factors are determined in the same way as in chapter 3.

5.2 Criteria

There are eight criteria which will be evaluated for every alternative. These criteria form the basis of the multi-criteria analysis. Some of these eight criteria are divided into subcriteria and a definition of every criterion and the different subcriteria is given below. The main criteria are:

- **Air draught limitation**
- **Regulation possibilities**
- **Operating time**
- **Total costs**
- **Risk of failure**
- **Load transfer to wall**
- **Aesthetics**
- **Additional space required**

Air draught limitation

If a superstructure for the gate is needed, this can limit the height of ships that can pass. Height limitations lead to a lower score compared with gate types that do not have an air draught limitation.

Regulation possibilities

A possible function of the lock, but not a necessary one, is to help regulate the water level. It may be beneficial if a gate type can help regulating the water level which leads to a higher score in the MCA.

Operating time

Different gate types have different operating times. The faster the gate is able to open and close, the higher it scores on the MCA.

Total costs

The total costs consist of three components:

Material costs

The amount of steel needed is calculated for each gate type and higher amounts of material leads to higher costs which has a negative influence on the MCA score.

Maintenance costs

When looking at maintenance costs, the accessibility of the gate will be evaluated. A high accessibility of the gate leads to a higher MCA score than a low accessibility. Also anti-corrosion measures are included in the maintenance.

Constructability

The amount of work that is needed to construct the door, including welding at the construction location and transport to the project site.

Risk of failure

Risk of failure consists of risks that can occur in the field of operating, for example the closing process or the possibility that the gate fails if a ship sails underneath it. Also the possibility to operate the gate manually in case of power outages will be evaluated. Besides operating failures, the probability of a ship collision is in the MCA. A higher probability means a lower score.

Load transfer to wall

The way in which the gate transfers the hydrostatic loads to the wall is different for the gates. It is beneficial if the gate transfer the loads in the longitudinal direction of the lock. When the gate transfers the loads in lateral direction, this leads to thicker walls which cause a lower score.

Aesthetics

Large superstructures have a negative influence on the view of the surroundings. It is benefical when as much structure as possible lies below the water level because this looks better.

Additional space required

Different gates lead to different extra amounts of space needed in the length direction of the lock. The less extra space needed, the higher the score. The result of the MCA for the big lock in alternative A can be found in Table 17.

Criteria	Weight	Vertical lift gate	Radial tainter gate	Mitre gates
Operating time	22%	3	3	5
Risk of failure	19%	4	4	3
Aesthetics	15%	2	2	4
Total costs	12%	4	4	3
Load transfer to the wall	12%	5	3	3
Regulating possibilities	11%	5	5	2
Additional space required	9%	5	4	3
	Total:	3.8	3.47	3.48

Table 17: MCA Big lock gate alternative A

The big lock in alternative A will be constructed with a vertical lift gate. The radial tainter gate and the mitre gates score significantly lower. Especially the regulating possibility of the lift gate and the fact that no additional space is required for this lock, lead to a higher score.

The MCA for the small lock in alternative A is almost the same but there are some small differences. The criterion 'air draught limitation' is included. Because the vertical lift gate will be the gate type in the big lock, it is necessary to check air draught limitation for the gates of the second lock. If for example also lift gates are used for the second small lock, both locks have got an air draught limitation for ships, which is disadvantageous for the navigability of the river.

Another difference is the exclusion of 'Regulating possibilities'. Because the lift gate of the big lock is able to help regulate the water level, it is not necessary that the second lock has regulation possibilities. The results of the MCA for the small lock in alternative A can be found in Table 18.

Criteria	Weight	Vertical lift gate	Radial tainter gate	Mitre gates
Operating time	21%	3	3	5
Risk of failure	21%	4	4	3
Air draught limitation	16%	1	1	5
Aesthetics	12%	2	2	4
Total costs	11%	4	4	3
Load transfer to the wall	10%	5	3	3
Additional space required	8%	5	4	3
	Total:	3.21	2.93	3.83

Table 18: MCA small lock gate alternative A

The small lock in alternative A will be constructed with double mitre gates. The vertical lift gate and the radial tainter gate score significantly lower, this is mainly because of the air draught limitation of both other gate types and their longer operating time.

5.3 Dimensions

Dimensions of the walls, the floor and the gate are determined for the lock. The wall and floor dimensions are determined using hand calculations for concrete from the Manual Hydraulic Structures (2016). The gate dimensions are determined using SCIA Engineer. In this section only the most important conclusions are presented, while the whole calculation and method can be found in appendix I. For alternative A, the wall thicknesses are calculated separately for the small lock and the big lock.

5.3.1 Gate dimensions

The maximum water level in the river ever, will be 7.12 m. For the lock doors, however, it is good not to only take the highest water level into account, but also to be prepared for waves that could occur during this highest water level. For this reason a height higher than 7.12 m is assumed. For inland waterways, if no wave data is available, 0.5 m has to be taken into account (CIE3330 Hydraulic Structures - locks, 2011). This seems high in relation to the wind setup, so it is decided to round the water height to 7.5 m.
For the small lock, the mitre gates of Table 15 are used. They close of an area of 10 m wide and are 7.5 m high. For the big lock the lift gate of Table 16 is used. This door is 21 m wide and also 7.5 m high.

5.3.2 Wall dimensions

The dimensions of the wall are determined based on the maximum bending moment in the walls. This moment is caused by hydrostatic pressure differences and by forces exerted by the gate (in case of the mitre gates) on the wall. On one wall of the small lock and one wall of the big lock, also a resultant force is exerted due to a water level on the one side and a soil layer on the other side. However, this resultant force does not lead to a governing bending moment.

The maximum bending moment in the small lock with mitre gates (Figure 17) is caused by the forces exerted by the doors and transferred to the walls. In addition to this moment, a moment due to the hydrostatic pressure differences on the walls is added.

The maximum bending moment in the big lock (Figure 16) with a lift gate is caused by hydrostatic pressure differences on the walls only, because the lift gate transfers its loads in longitudinal direction of the lock and therefore does not cause a bending moment in the lock walls (this is the reason that the maximum bending moment is much smaller, however the wall has to be thick due to buckling considerations). The maximum bending moments are schematized in figures 15 & 16.

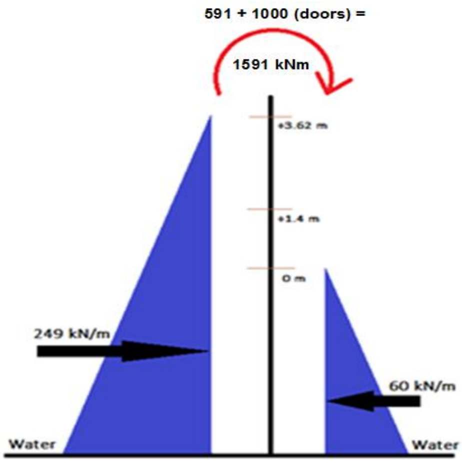


Figure 17: Governing moment small lock, mitre gates

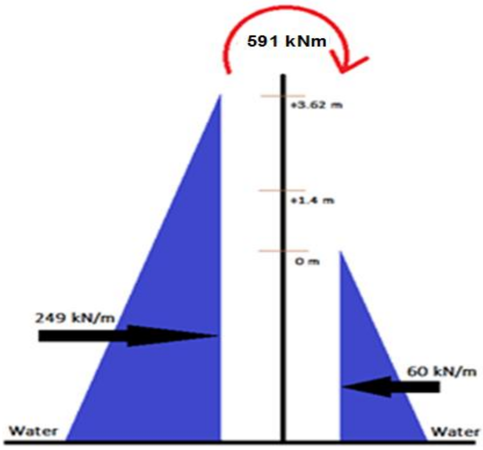


Figure 16: Governing moment big lock, lift gate

For the calculation of the wall thicknesses, use is made of the following parameter:

$$\frac{M_u(breaking\ moment)}{b\ (width) \cdot \mathrm{d\ (height)}^2\ \cdot \mathrm{f}_{cd}(design\ strength)}$$

For a complete description, calculation and schematization of the wall thicknesses, reference is made to appendix I. The maximum bending moments, wall thicknesses and material costs can be found in Table 19.

	Max bending moment (kNm)	Wall thickness (m)	Material cost (₫)
Small lock	1591	0.8	10,548,305
Big lock	591	1.0	13,354,680

Table 19: Resulting moment and wall thickness for the locks of variant A

It is chosen to dimension both walls of each lock equally large because this eases the construction and makes the construction safer, because it is not possible to switch the walls and reinforcement. Also, material costs are not that high so that the total costs of the wall will not grow significantly.

5.3.3 Floor dimensions

The hydraulic structure contains a big concrete floor as a foundation of the lock(s).The stability and strength of the structure have to be checked for ULS conditions, using load factors to obtain the required safety. It is chosen to use compact underwater concrete without reinforcement as floor material to prevent leakages. The concrete class is C35/45, this is the same as for the walls of the structure. For the design of the lock chamber floor, the following functions have to be considered:

- Retaining water, this implies resisting water pressure. A concrete floor is impermeable and will be loaded by hydrostatic water loads, it has to withstand the water pressure as a load. It also has to withstand water pressure acting as a buoyancy force on the concrete floor.
- Load transfer function, a concrete floor has due to its stiffness a considerable influence on the stability. It has to be designed to resist these forces by transferring it to the piles, or by using expansion joints to separate different segments as could be determined in detail design.

The dimensions of the floor are calculated in the next paragraph for a big lock (111x21m), a more detailed explanation of these calculations can be found in appendix K. The floor has to be designed in such a way that it provides overall stability by making sure that ∑V,∑H,∑M=0 and because the floor is constructed without reinforcement, the tensile stress has to be lower than the tensile strength. The forces in horizontal direction cannot be resisted by the floor, these forces are transferred to the piles. This is discussed in and calculated in paragraph 5.4.

5.3.4 Big Lock

The length of the floor is 140 m and the width is 23 m. The maximum water level on the inside(HCMC) can be MSL +1.0 m, while on the outside the maximum water level can be MSL +3.62 m. The lock is divided into two equal surface area zones with a different water level on the outside of these two zones, the blue dotted line indicates the border as can be seen in Figure 18.

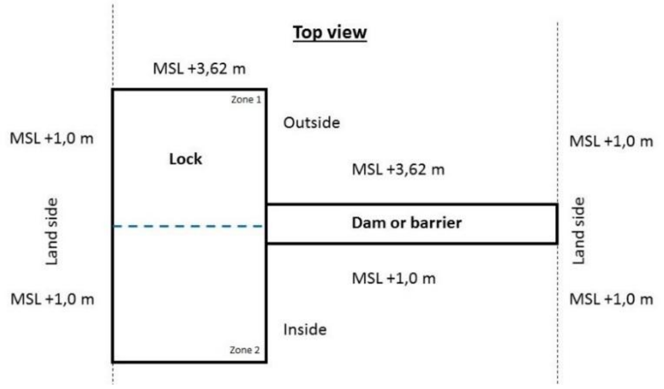


Figure 18: Floor of the big lock, divided into pressure zones

The upward pressure of the two zones is:
P1=ρ*g*h1=1000*9.81*7.12=69847Pa= 70KN/m
p2=ρ*g*h2=1000*9.81*4.5 = 44145 Pa = 44KN/m

5.3.5 Thickness of the floor

The buoyancy force of the lock is for half the surface area 44KN/m and for the other half 70KN/m. This buoyancy force has to be lower than the vertical downward forces of the construction so the construction is not lifted up by the water pressure. The vertical forces downward of the construction consist of: weight of the floor, water, walls and gate. This force is in total 50.4KN/m+23KN/m* floor thickness. This downward force is displayed in Figure 19 as Q3. The two buoyancy forces are displayed as Q1 and Q2.

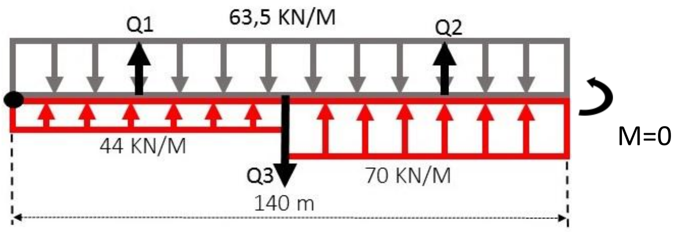


Figure 19: Forces on the concrete floor

For the piles it is easier to resist a compressive force than a tensile force and it is difficult for the piles to resist a moment. Therefore two aspects are important to create in the floor of the lock:

- ∑V=0 or ∑V = downward.
- ∑M=0

When ∑V=0 than ∑M≠0, so the thickness of the floor is calculated in such a way that ∑M=0 and that no buoyancy occurs. It is calculated that the value of Q3 has to be equal to 63.5 KN/m, so 63.5=50.4+23*thickness of floor, this provides a thickness of the floor of 0.57 meter.

A floor thickness of 0.57 meter does result in a ∑V downward compressive force of 6.5KN/m with a maximum water level difference for inside and outside the lock.

Bending stress in concrete floor

Because the floor is constructed without reinforcement, the tensile stresses have to be lower than the tensile strength of the concrete. According to the structures manual, the design value for concrete tensile strength is computed as follows:

$$f_{ctd} = \frac{\alpha_{ct} * f_{ctk,0.005}}{\gamma_c} \frac{1 * 2.2}{1.5} = 1.47\ MPa$$

The tensile stresses can be computed as follows:

$$\sigma = \frac{M}{W} = \frac{\frac{1}{10} * q * l^2}{\frac{1}{6} * b * h^2}$$

The concrete floor can be constructed in several segments according to the lock manual. The maximum length of a segment can be determined by equalising the tensile strength and the tensile stress, the only unknown parameter is the segment length (l). The maximum length is determined to be 59 meter.

The tensile stress should be lower than the tensile strength to prevent bending for safety reasons. As is stated in the lock manual a typical length for the segments is 25 meter. No bending stresses in the floor will occur with a segment length smaller than 59 meter, because in practice the length will be about 25 meter, a floor thickness of 0.57 meter is safe.

A side view of the big lock is provided in Figure 20.

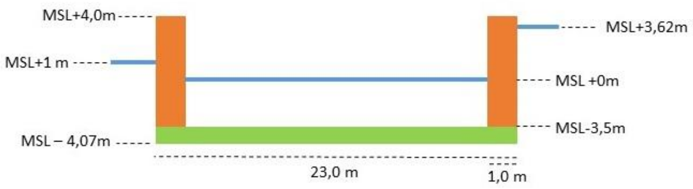


Figure 20: Side view of the big lock

5.3.6 Small lock

This alternative contains two locks, a big lock and a small lock. The big lock in this alternative has the same floor dimensions as the big lock calculated in the other alternative.
The dimensions of the floor of the small lock are 85m x 11.6m. The upward water pressure is the same as for the big lock, again it is aimed to have ∑M=0. To achieve that ∑M=0, the value of Q3 is again 63.5 KN/m. The vertical force downward is for the small lock: 65.3KN/m+23KN/m*thickness of floor.

The total weight of the water and walls makes clear that the floor is not needed to achieve a higher vertical force downward than upward. Of course a floor is constructed so locking can take place and the floor can transfer the forces to the piles. A floor thickness of 0.40 meter is assumed.

The higher vertical load downward than upward can be resisted by the piles and is 11 KN/m for a maximum water level difference for inside and outside the lock. This vertical downward force creates a moment in the concrete floor which turns clockwise. There are only two possible pivot points on the floor, as is shown in Figure 21. The black lines show there can only occur a compressive force and no tensile force due to the created moment. The compressive forces can be resisted by the piles so it is safe to assume a floor of 0.4m.

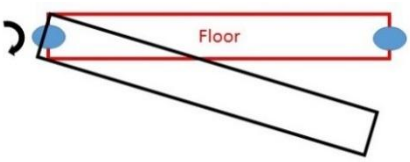


Figure 21: Pivot points of the floor

The tensile strength is the same for the small lock and the big lock. When the tensile stress equals the tensile strength, the length of the segments has to be 74 meter. As is stated before, the distance between the piles will be a lot shorter, so the tensile stress will be a lot smaller than the tensile

5.3.7 Dimensions of the culvert

The culvert dam is assumed to be 31.4 meters wide (66 m – 23 m – 11.6 m) and it is 8.5 meters high. From MSL + 1 to MSL - 3.5 there are 5 culverts. The 5 culverts are parted by 4 slabs of concrete with a width of 0.6 meters. The dam is assumed to be 2 meters wide.

5.4 Foundation

5.4.1 Method

To make sure the designed lock and barrier stays on the same place a foundation is needed. Because the soil in the area is too weak to make a shallow foundation, the locks are founded on a pile foundation. In order to make this pile plan, soil investigation is used to describe the capacities of some pile types. To obtain these results the Eurocode 7, Shariatmadari et al. (2008), Baguelin (1972), Briaud (1997), R. Rajapakse (2008) and Vesic (1977) are used. The steps taken are listed below, followed by the resulting pile plans. A total calculation is presented in appendix J.

1. Concrete piles are chosen as useable piles, based on their high load capacity, corrosion resistance and the fact that they can be easily driven through soft layers.
2. The bearing, tension and lateral capacities of 4 selected piles are calculated.
3. The normative situations are selected, and it is argued that the locks can be considered infinitely stiff.
4. The normative forces on the locks are calculated. Two parts for each of the variants are distinguished. For variant A both of the locks are assumed to take the forces of the culvert dam. For variant B a pillar is designed at the other side of the lock.
5. With the normative pile capacities and normative forces (see Table 20) the pile plan is designed.

Part	Bearing (ULS)		Tension (ULS)		Lateral (ULS)	Settlement (SLS)		
	$F_{max;down}$	M_{max}	$F_{max;up}$	M_{max}	$F_{max;hor}$	$F_{v'SLS}$	$F_{h;SLS}$	M_{SLS}
	[kN]	[kNm]	[kN]	[kN]	[kN]	[kN]	[kN]	[kNm]
A – big lock	208624	25157	-93477	1423853	9508	135640	2553	6133
A – small lock	82938	18451	-23666	363902	6706	59855	1802	4319

Table 20: Normative forces for the locks

5.4.2 Result

The pile plan for this variant is presented in Table 21.

Part	Length	Width	Pile type	Prestress	No of piles	CtCd width	CtCd length	Lowest FOS	Displacements	
									Vertical	Lateral
	[m]	[m]	[-]	[N/mm^2]	[-]	[m]	[m]	[-]	[mm]	[mm]
Big lock	120	23	400*400	3.1	120	5.5	5	1.05	10	0.018
Small lock	85	11.6	400*400	3.1	45	5.15	5.66	1.37	12	0.034

Table 21: Pile plan for the big and small lock

The total number of piles is 165. This pile plan meets the ULS and SLS requirements for tension, stress and moments and is checked on the 'clod criterion'. The capacities of the 400*400 pile with a reinforcement percentage of 0.4 % are found as presented in Table 22. Due to prestressing to 3.1 N/mm^2 the internal tension capacity is extended to the soil tension capacity.

Diameter	Internal capacity		Soil capacitiy			Movement	
	Pressure	Tension	Lateral	Bearing	Tension	Vertical	Lateral
	$R_{p;p}$	$R_{p;t}$	$H_{max;ULS}$	$R_{c;d;uls}$	$R_{s;k;uls}$	S_{total}/F_v	$y_{0;2}/F_h$
[mm]	[kN]	[kN]	[kN]	[kN]	[kN]	[mm/kN]	[mm/kN]
400	8614	742	1159	3692	1235	0.0093	0.00085

Table 22: Properties of the concrete pile

5.4.3 Comments

Some comments have to be made about this design:

- The pile capacities are found by a literature study, but without experience with the local soil. Therefore the values taken in the calculations are always taken for the worst case scenario. This may lead to over-dimensioning of the foundation. On the other hand the uncertainty about the applicability of the literature may cause unexpected risks.
- The floor thickness is designed based on the worst case situation in navigation circumstances, but not with the empty lock chamber, due to maintenance, in mind. Therefore there a very high tension forces in a maintenance situation. If the floor is designed thicker these forces would be lower and a more balanced design could be found.
- In the future it is recommended to use CPT instead of SPT, this is also recommended by Shariatmadari N. et al. (2008).
- The piles chosen are driven prestressed concrete piles. It is assumed that the piles can be driven in this area, when this is not possible, screwed piles can be used. It must be taken into account that for most of the soil capacities of the pile the capacities of a screwed pile should be taken 0.9 for that of the driven piles. Also it is not possible to prestress screwed piles and therefore the reinforcement percentage should be increased.

5.5 Flow and navigation

The width of the opening in this variant becomes the 29 meters of the culvert dam and the 21 meters of the big lock. Because the small locks have mitre gate doors, they can only be opened separately. Therefore the total conveyance width becomes 29 + 21 = 50 meters. This situation is described in chapter 4.2 where a minimum conveyance width of 30 meters is assumed enough for the outflow during peak rains. Therefore this design meets the requirements for flood protection.

Following chapter 4.2 the maximum flow speed during dewatering becomes 1.62 m/s. Because the stated maximum flow speed during navigation is assumed 1.5 or 2.0 m/s, in this case it can be said that the situation with maximum flow speed is still good to navigate. Therefore free navigation will be possible 11.6 hours on the 'worst day' and is possible 50% of the time. When the barrier is closed due to flood functions, the locks can be used to continue navigation. In this design an additional benefit is that the navigation is never blocked by a high water flow, and the additional risks coming with this are not present.

5.6 Cost estimation

In order to estimate the costs of the variant a comparison has been made between the variant's dimensions and the existing variant designed by the IWER for which the complete cost estimation is available. For example the known costs of dredging are for instance divided by the area that has to be dredged creating the dredging cost per unit, these costs per unit are then multiplied by the surface area of the variant that requires dredging. The costs taken into account for comparison are the costs of: foundation, construction methods, embankment, improvement of the river bed, dredging, the control center, lifting gates and the mitre gates.

	Given existing project				**Variant A**	
Activity/Item	Costs	Unit		Unit Costs	Dimensions	Costs
Site preperation	VND 22,572,811,058	6500	m^2	3,472,740	8192	VND 28,448,687,413
Foundation	VND 286,616,955,673	6500	m^2	44,094,916	8192	VND 361,225,553,980
Construction method	VND 332,180,335,845	1		332,180,335,845	1	VND 332,180,335,845
Embankment	VND 240,465,564,412	228	m	1,054,673,528	320	VND 337,495,528,999
River bed improvement	VND 13,213,343,151	1		13,213,343,151	1	VND 13,213,343,151
Dredging	VND 39,890,988,584	1	m^2	39,890,988,584	1	VND 39,890,988,584
Control Centre	VND 3,677,120,947	1		3,677,120,947	1	VND 3,677,120,947
Lifting Gates	VND 23,397,840,228	360	m^2	64,994,001	315	VND 20,473,110,200
Mitre Gates	VND 24,407,348,111	450	m^2	54,238,551	318	VND 17,247,859,332
Total costs	VND 986,422,308,009					VND 1,153,852,528,451
Total costs expressed in euros	**€ 39,456,892.32**					**€ 46,154,101.14**

Table 23 - Estimated costs of variant A

The table above shows how the costs of the IWER project and Variant A relate to each other using estimates for the dimensions. Concerning the construction method, river bed improvement, dredging and the control center no variations are made in the costs as these are not dependent on the dimensions of the individual structures.

5.7 Conclusion

After all the calculations in this chapter, the final design of variant A can be found in Figure 22.

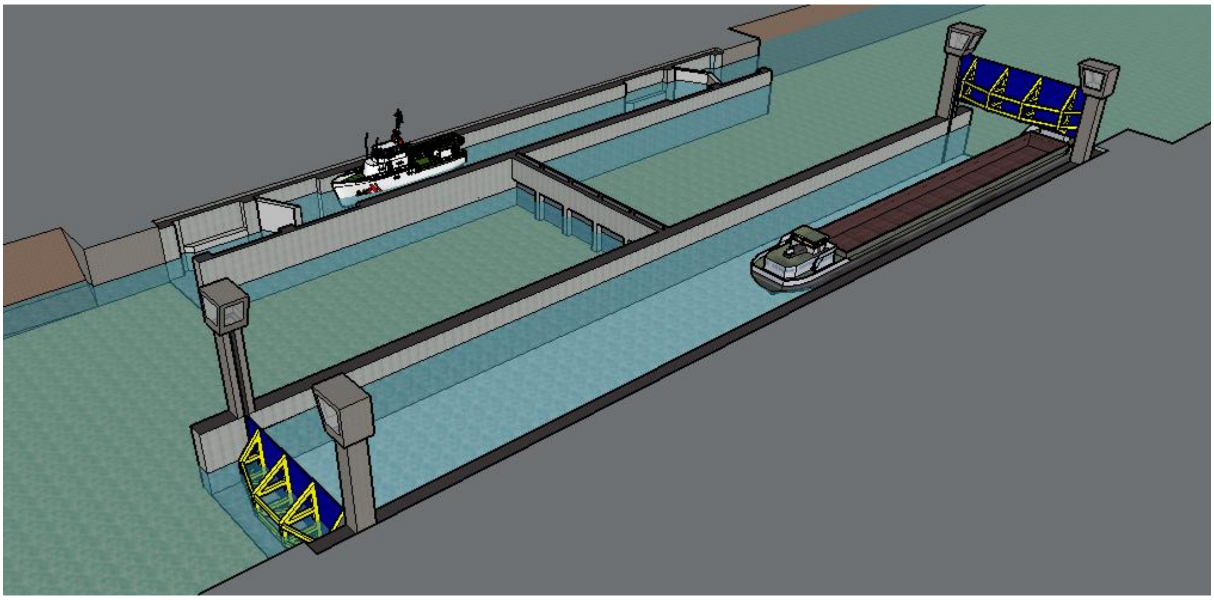


Figure 22: Visualization of the final design of variant A, with the Mekong delta on the right.

The specifications of variant A are:

- The small lock has dimensions of the water area of 10 m by 70 m and a cycle time of 29 minutes, which leads to 10.5 ships passing per hour.
- The big lock has dimensions of the water area of 21 m by 111 m and a cycle time of 68 minutes, which leads to 15.2 ships passing per hour.
- The width of the walls of the small lock is 0.8 m.
- The width of the walls of the big lock is 1.0 m.
- The area of the big lock is 120 m by 23 m.
- The area of the small lock is 85 m by 11.6 m.
- The dam with culverts is 2 m wide.
- The small lock has mitre gates.
- The big lock has lift gates.
- The thickness of the floor of the big lock is 0.57 m.
- The thickness of the floor of the small lock is 0.4 m.
- A total of 165 piles form the foundation of this variant.

The main advantages of this variant is that the ship waiting time for the locks is a maximum of 3 hours for 1 ship. The risk of flooding is low, due to a double set of doors in each lock and the big lock can be used for releasing water and the passing of ships in the open situation.

6. Alternative B

Alternative B consists of a tidal lift barrier in combination with a navigation lock. The big lock has got the same size as the big lock in alternative A. The same procedure with respect to the gate selection will be followed as in chapter 5.

6.1 Gate selection

In the same way as for alternative A, a preselection of the gate types is made before ranking gate types in the MCA.

6.1.1 Preselection barrier gate type

The preselection is the same as in chapter 5.1. Four main restrictions can be found which may cause unsuitability for each type of gates used in the barrier in alternative B:

1. The span which is needed at Phu Dinh is too large for the concerning gate type
2. Additional horizontal space in the quays is required to accommodate the structure
3. Significant current or wave forces prevent the gate from operating
4. A high level of experience and/or expertise is needed for the concerning gate type

The difference between these restrictions and the lock restrictions for alternative A is that the criterion 'the operating time is too high which increases the lock cycle time' is replaced for the criterion 'significant current or wave forces prevent the gate from operating'. This is because the operating time of a gate is not as important for a barrier as it is for a lock. Also, the barrier should operate in situations where there is significant flow. This is not important for the lock because the lock only operates when the barrier is closed and in that situation there is no flow.
In the same way as for alternative A, the gates will be tested on the four restrictions. Again, in Table 24 a red X indicates that the gate type is definitely unsuitable at Phu Dinh. A green X indicates that the gate type may be unsuitable at Phu Dinh but this depends on the situation.

	Res. 1	Res. 2	Res. 3	Res. 4
Double mitre gates	X		X	
Single leaf gate	X			
Lift gate				
Submersible gate				X
Rolling/sliding/caisson		X	X	X
Radial/sector gate		X		X
Radial/tainter gate				
Inflatable weir				X
Flip-up barrier[2]				X
Rotatable weir			X	X

Table 24: Preselection gate type for the tidal barrier of alternative B

After this preselection three lock doors remain for the tidal barrier in alternative B: the vertical lift gate, the radial tainter gate and the flip-up barrier.

6.1.2 Scia investigation

The doors that have not been investigated before are investigated. In this case, those doors are the doors for the tidal barrier. The results for the big lock can be found in chapter 5.1.2. The SCIA designs are found in Appendix H

[2] The flip-up barrier is a quite complex gate type but it is interesting to investigate whether this gate type can be useful and what the scores will be in the MCA because it seems a good option, however it is a bit out of the box.

Findings and concluding remarks

For the tidal barrier the vertical lifting gate and the radial/tainter gate contain an equal amount of steel, however the vertical lifting gate can cover the entire 40m span while the radial gate would consist of two smaller gates since the 40m span would be too large for a single radial gate. The following tables give an overview of the gates for the tidal barrier:

Vertical lift gate		Radial gate	
Water difference	3.62m	Water difference	3.62m
Amount of steel used	525KN	Amount of steel used	526KN
Amount of profiles	7	Amount of profiles	7

Table 25: Overview of the results of the Scia investigation for the tidal barrier.

6.1.3 MCA: Final selection gate type

After the preselection, the best gate option for the lock and the barrier in alternative B has to be determined with an MCA. The results of the MCA for the barrier gate selection can be found in Table 26.

Criteria	Weight	Vertical lift gate	Radial tainter gate	Flip-up gate
Operating time	19%	5	4	2
Risk of failure	19%	5	4	3
Total costs	12%	5	4	4
Additional space required	12%	3	2	4
Aesthetics	11%	2	3	5
Air draught limitation	10%	2	2	5
Load transfer to the wall	9%	4	4	5
Regulating possibilities	9%	5	5	4
	Total:	3.94	3.54	3.46

Table 26: MCA tidal barrier gate selection alternative B

The barrier in alternative B will be constructed with a vertical lift gate. The radial tainter gate scores lower for its risk, total costs and additional space required, while the flip up gate scores much lower on operating time and the involved risk.

The MCA for the lock gate selection differs from the MCA for the barrier gate selection with respect to one criterion. 'Regulating possibilities' is kept out of the MCA for the lock gate because the regulating function of the whole system at Phu Dinh is executed by the barrier. Therefore the lock does not have to fulfill a regulating function.

The results of the MCA for the lock gate selection can be found in Table 27.

Criteria	Weight	Vertical lift gate	Radial tainter gate	Mitre gates
Operating time	21%	3	3	5
Risk of failure	21%	4	4	3
Air draught limitation	16%	1	1	5
Aesthetics	12%	2	2	4
Total costs	11%	4	4	3
Load transfer to the wall	10%	5	3	3
Additional space required	8%	5	4	3
	Total:	3.21	2.93	3.83

Table 27: MCA lock gate selection alternative B

The navigation lock in alternative B will be constructed with mitre gates. These score the highest, due to their low operating time and due to the fact that no air draught limitation will be present.

6.2 Dimensions

For alternative B, the lock walls are dimensioned in the same way as for alternative A. The only difference is the fact that in alternative A a lift gate is used as gate type and in alternative B mitre gates are used as gate type.

6.2.1 Gate dimensions

For the tidal barrier, the lift gate is chosen. It will be 40 m wide and 7.5 m, the same height as is explained in section 5.2.1. For the lock, mitre gates are selected in this case. They close an area of 21 m wide and are 7.5 m high.

6.2.2 Wall dimensions

The lock in alternative B is a lock with mitre gates. One wall of the lock, the wall adjacent to the barrier, is subjected to hydrostatic pressure differences. The other wall, the wall adjacent to the quay, is subjected to a hydrostatic pressure at one side and a soil layer on the other side. The largest moment however comes from the load that the mitre gates have to resist. The maximum bending moment is schematized in Figure 23.

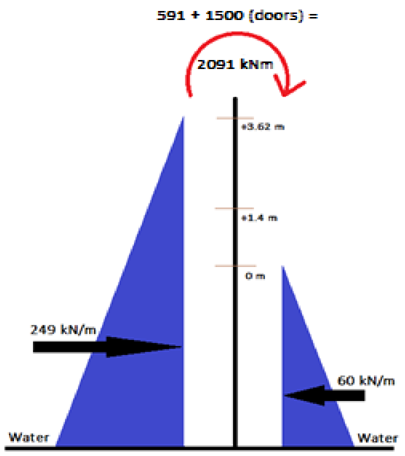


Figure 23: Forces on the wall of the lock

After the wall calculation, thicknesses of 0.6 – 1.4 m were satisfying the reinforcement ratio. A thickness of 0.6 m was the most economic one but a thickness of 0.9 m was chosen because it was only slightly more expensive and it made the construction easier because less reinforcement has to be used. After the shear reinforcement check, the walls were too slim. By increasing the wall to 1.0 m, the shear reinforcement check was satisfied and no shear reinforcement is needed.

The bending moment, thickness and material cost of the walls can be found in Table 28. Both walls are dimensioned with the same thickness of the wall, this is, as explained before, easier for construction.

	Max bending moment (kNm)	Wall thickness (m)	Material cost (₫)
Lock	2091	1.0	13,354,680

Table 28: Bending moment, wall thickness and material costs

For a complete description, calculation and schematization of the walls, reference is made to appendix I.

6.2.3 Floor dimensions

This alternative contains one big lock with mitre gates. The only difference in comparison with the big lock in alternative A is the weight of the gate, and this weight difference between the gates is negligible as can be seen in appendix K. The big lock in this alternative has the same floor dimensions as the big lock calculated in the previous alternative, because the reasoning is the same. So the floor thickness for the lock in this alternative is 0.57 meter.

6.2.4 Sill dimensions

For the dimensions of the sill, for the tidal barrier, no clear rules could be found by the design team. For now the width and thickness of the sill will thus be assumed. The thickness is assumed to be (height dredged (MSL - 5.5 m) – draught just before dredging (MSL – 3.5 m) =) 2 m, this gives enough room for sediment to deposit. The width of the sill is also assumed to be 0.5 m, this gives room for the gate to close on, while it also gives some room to transfer moments if that is needed.

6.3 Foundation

6.3.1 Method

The method used is the same as presented for the foundation of variant A. The resulting forces for both sides of the construction are presented in Table 29. The total calculation is worked out in Appendix J.

Part	Bearing (ULS)		Tension (ULS)		Lateral (ULS)	Settlement (SLS)		
	$F_{max;down}$	M_{max}	$F_{max;up}$	M_{max}	$F_{max;hor}$	$F_{v'SLS}$	$F_{h;SLS}$	M_{SLS}
	[kN]	[kNm]	[kN]	[kN]	[kN]	[kN]	[kN]	[kNm]
B – big lock	213264	29060	-109947	1936031	10435	154389	2837	6818
B – pillar	1452	13515	-	-	4914	1117	1320	3170

Table 29: Governing forces on the structure

6.3.2 Result

The pile plan for this variant is presented in Table 30.

Part	Length	Width	Pile type	Prestress	No of piles	CtCd width	CtCd length	Lowest FOS	Displacements	
									Vertical	Lateral
	[m]	[m]	[-]	[N/mm^2]	[-]	[m]	[m]	[-]	[mm]	[mm]
Big lock	140	23	400*400	3.1	140	5.5	5	1.05	10	0.017
Pillar	25	2	400*400	3.1	6	-	5	1.24	18	0.18

Table 30: Pile plan for variant B

The total number of piles is 146. This pile plan meets the ULS and SLS requirements for tension, stress and moments and is checked on the 'clod criterion'. The capacities of the 400*400 pile with a reinforcement percentage of 0.4 % are found as presented in **Error! Reference source not found.**. Due to prestressing to 3.1 /mm^2 the internal tension capacity is extended to the soil tension capacity.

6.3.3 Comments

The comments made for variant A (paragraph 5.3) also apply to variant B. Therefore it is recommended to read them when using this pile plan.

6.4 Flow and navigation

The width of the opening in this variant becomes the 40 meters of the lift gate. Because the big locks have mitre gates, they can only be opened separately. Therefore the total conveyance width becomes 40 meters. This situation is described in chapter 4.2 where a minimum conveyance width of 30 meters is assumed enough for the outflow during peak rains. Therefore this design meets the requirements for flood protection.

Following the same procedure as in chapter 4.2 the maximum flow speed during dewatering becomes 1.96 m/s. Because the stated maximum flow speed during navigation is assumed 1.5 or 2.0 m/s, in the first case the flow speed becomes too high for navigation at certain moments. Free navigation will be possible 8.6 hours on the 'worst day' and is possible 40% of the time. In case of a limit of 2.0 m/s the navigation will be possible 50% of the time. Because the waiting time in this variant is high when using the locks, the navigation can continue during flood protection, but a big delay will be present for most of the traffic.

6.5 Cost estimation

For the cost estimation the same method has been applied as in chapter 5.6, this method reveals the following estimations.

	Given existing project				Variant B	
	Costs	Unit		Unit Costs	Dimensions	Costs
Site preperation	VND 22,572,811,058	6500	m^2	3,472,740	7476	VND 25,962,205,457
Foundation	VND 286,616,955,673	6500	m^2	44,094,916	7476	VND 329,653,593,940
Construction method	VND 332,180,335,845	1		332,180,335,845	1	VND 332,180,335,845
Embankment	VND 240,465,564,412	228	m	1,054,673,528	228	VND 240,465,564,412
River bed improvement	VND 13,213,343,151	1		13,213,343,151	1	VND 13,213,343,151
Dredging	VND 39,890,988,584	1	m^2	39,890,988,584	1	VND 39,890,988,584
Control Centre	VND 3,677,120,947	1		3,677,120,947	1	VND 3,677,120,947
Lifting Gates	VND 23,397,840,228	360	m^2	64,994,001	300	VND 19,498,200,190
Mitre Gates	VND 24,407,348,111	450	m^2	54,238,551	483	VND 26,197,220,306
Total costs	VND 986,422,308,009					VND 1,030,738,572,832
Total costs expressed in euros	**€ 39,456,892.32**					**€ 41,229,542.91**

Table 31: Estimation costs variant B

6.6 Conclusion

After all the calculations of before, the final design of variant B can be found in Figure 24

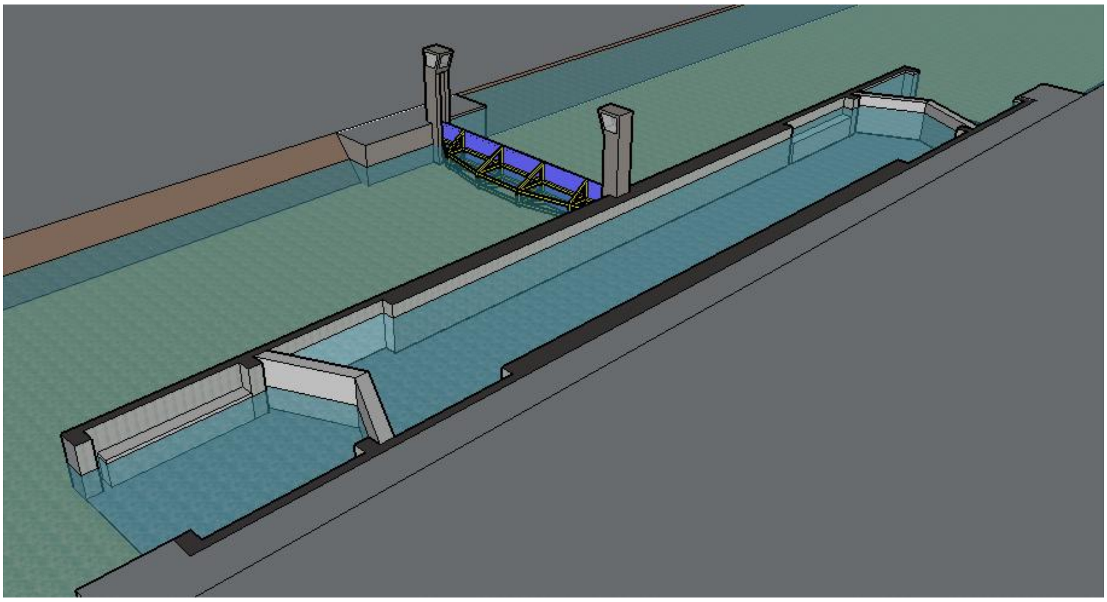


Figure 24: Visualization of the final design of variant B.

The specifications of variant B are:

- The lock has dimensions of the water area of 21 m by 111 m and a cycle time of 68 minutes, which leads to 15.2 ships passing per hour.
- The width of the walls of the big lock is 1.0 m.
- The area of the lock is 140 m by 23 m.
- The width of the tidal barrier is 40 m.
- The tidal barrier is a lift gate.
- The lock has mitre gates.
- The thickness of the floor of the lock is 0.57 m.
- The sill underneath the tidal barrier has a width of 0.5 m and a height of 2 m.
- A total of 146 piles form the foundation of this variant.

The main advantage of this variant is that a wide area of 40 m is available for ships to sail through when the tidal barrier is open. The main disadvantage of this variant is that when the tidal gate is closed, all ships will have to go through the lock, leading to a waiting time of 5 hours.

7. Choice of definitive variant

The choice between the two different variants is made, based on a MCA.

7.1 MCA

To select the definitive variant, as before in chapter 3.3, a multi criteria analysis is performed to make a choice between variant A and variant B. The designs are now more detailed and the MCA is performed again. During the process more knowledge about the performance and requirements of both variants is obtained, therefor the criteria are specified again.

7.1.1 Criteria

There are 4 criteria that are evaluated for both variants:

- **Performance**
- **Costs**
- **Risks**
- **Environmental suitability**

In the following paragraphs these criteria are elaborated. It is important to notice that the possibility of failure should be taken into account only in the category 'Risks', for all other categories a fully performing design is assumed.

7.1.2 Performance

This criterion should give a measure of how well the design satisfies the stated functional requirements and boundary conditions. This results in the following sub categories:

Navigation performance

This aspect should describe how well the design meets the performance requirements based on the navigation function of the locks and barrier. This means the operating time of the locks, waiting times, the time of free navigation trough the locks/barrier and largest/highest possible vessels should be taken into account rating this category.

Flood protection performance

The rating of this category should be based on the ability of the design to prevent flooding in HCMC. This means the chances of a flooding due to a high tide as well as a flood due to extreme rains should be taken into account.

Outflow regulation performance

This category is rated based on how well the design can deal with specific outflow regulation situations. During the dry season it may be needed to have a small regulated outflow in order to keep a flow in the river.

Additional performance aspects

In this category other opportunities of the design can be taken into account. The possibility of building a bridge to disclose the traffic in the area for example.

7.1.3 Costs

The costs are divided in two categories, one for direct costs and one for operation and maintenance costs.

Construction costs

These costs are all costs that are made in order to get the design in place. The rating is based on the estimation of the construction costs, but also takes into account management, judicial and process costs.

Future costs

The rating of this category is based on the estimated costs the design will have in the future. Maintenance, operation and life cycle are factors that should be taken into account giving a value for this aspect. Also future environmental pollution should be taken into account in this aspect.

7.1.4 Risks

This criterion should take into account all the effects of the construction and operation that are not as planned. Therefore four categories are selected.

Construction risks

This category looks at the risks that occur during construction. This could be risks of personal harm, but also risks of increased costs due to miscalculations or other unexpected setbacks.

Operation risks

This rating should be based on the chance and effect of a failure during operation. This could be personal harm, financial losses, loss of navigation etc..

Flooding risks

The risk of a flooding caused by a failure of the lock-barrier system should be taken into account. This means the risk following a failure of the structure, lock doors, barrier, soil, foundation etc..

Future risks

Risks that influence the current or future performance of the design by external risks should be rated in this aspect. This could be an increase in shipping intensities, ship dimensions, water levels, surrounding structures and so on.

7.1.5 Environmental suitability

This category looks at the way the design fits in its surroundings. This is done by looking at the aesthetics of the design and the hindrance the local people may suffer.

7.2 Weight factors

In the same way as before (appendix D) the weight factors are determined by the design team. Then the sub weight factors are determined. Finally the MCA is filled in for the different variants. This results in the Table 32 below. For each variant and each sub criteria, a score between 1 and 5 is given.

Category		Sub criteria	Subweight	Variant A	Variant B
1. Performance	39%	Navigation performance	0.35	5	4
		Flood protection performance	0.35	5	5
		Outflow regulation performance	0.2	4	4
		Additional performance aspects	0.1	4	2
2. Costs	20%	Construction costs	0.6	3	4
		Future costs	0.4	3	4
3. Risks	24%	Construction risks	0.2	4	3
		Operation risks	0.4	4	3
		Flooding risks	0.3	5	3
		Future risk	0.1	4	3
4.Env. suitability	17%		1	3	3
Total score				4.0	3.6

Table 32: MCA of the two remaining variants

The result from the MCA is that variant A scores 0.4 points higher than variant B. This is relative a significant difference, since the two variants do not lay more than one point apart in most of the categories. The variant that is chosen is variant A. The main points that gave the advantage of variant A above variant B are:

- The high navigation performance while the barrier has to be closed, due to the two locks. The difference when the barrier is open is assumed minimal looking at the shipping intensities;
- The additional things that can be used of the design in the future, for instance it is a good foundation for a bridge;
- The risks in variant A are clearly lower, this is mainly because the moving mechanisms are all double in variant A, double locks for navigation and double gates for flood protection;
- In the future variant A can deal better with higher shipping intensities and is also easier to adjust in case the structure satisfies the needs no more.

The costs are a little bit higher than the costs for variant B, however, the extra possibilities outweigh this difference.

8. Method of construction

In this chapter the way the chosen variant can be built is explained. The steps in the building process are discussed. Some calculations on for instance the stability of the sheet piles for the cofferdam are performed.

8.1 Cofferdams

8.1.1 Overview

The total construction will take place in cofferdams. These dams will protect the working area and make it possible to pour the concrete floor into place. Two building pits are chosen in the first place, one for each of the locks. In this way the navigation can proceed during the construction of the locks. When the locks are completed they can be used for navigation while the culvert dam is being built as shown in Figure 25. Another benefit of the plan is that the sheet piles of the locks can be re-used during the construction of the culvert dam.

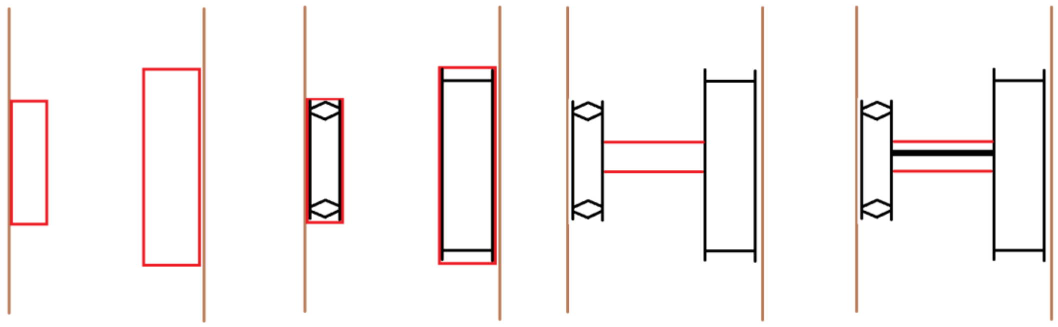


Figure 25: Construction phases

8.1.2 Sheet piles

The sheet piles used to create the building pit need to be stable. Therefore the sheet piles are tested on the total forces, the moment around the pile tip and the moment around the struts (if applied). In order to calculate these situations the Excel sheet presented in Appendix M is used. The normative boring used in this situation is HK2 as shown in chapter 2.2.7. A minimum safety factor of 1.1 is assumed enough, taken into account that the soil layers in HK2 are far worse than the other borings. Following this calculations the needed depth for a sheet pile next to the soil wall becomes MSL - 25.5 m without struts and MSL - 19 m when struts are added, both with a FOS (Factor Of Safety) of 1.10. Because the difference in depth is only 6.5 meters and the practical benefits of a free working space are big, the sheet piles without struts are chosen. For a sheet pile constructed in the water the needed depth becomes MSL - 19 m (FOS 1.11). This means the sheet piles will be placed as shown in the first frame of Figure 26. The top of the piles will be at MSL + 3 m. All sheet piles will be vibrated in from pontoons that are connected to the quay.

8.2 Excavation

After the placement of the sheet piles the soil is removed until a depth of MSL - 4.1 m for the big lock and MSL - 3.9 m for the small lock. The sheet piles had to be very deep in order to do this in dry condition, therefore it is chosen to do this in wet conditions. This step is done by an excavator operating from a pontoon. The situation then becomes as shown in the second frame of Figure 26.

8.3 Foundation on piles

Following the piles can be placed, following the pile plan described in chapter 5.4. The piles are driven from the pontoon until they are 1.6 meters into the 4a layer. The specific pile lengths for each place must be designed in the detailed design. The top of each pile should be between MSL - 3.8 m and - 3.6 m. This should be controlled by GPS or divers. Also the sheet piles to prevent from piping should be placed in this phase. This sheet pile should be placed until a depth of MSL - 13.3 m and starts at -4.0 in order to make a good connection between the sheet pile and the concrete floor. The placement of the piles is shown in the third frame of Figure 26.

8.4 U.W.C.

When the foundation piles are placed an underwater concrete floor is placed. The pile tips should be clean when this process is started in order to get a good connection between the floor and the piles. Also the sheet piles should be clean at the location where the UWC has to attach to the sheet piles for a good connection. The UWC is poured by a tube operating from the pontoon. It is important that the whole floor is placed in one go to ensure the stiffness in the floor. The UWC should be poured until a depth of MSL - 3.5 m is reached. After that the concrete can harden.

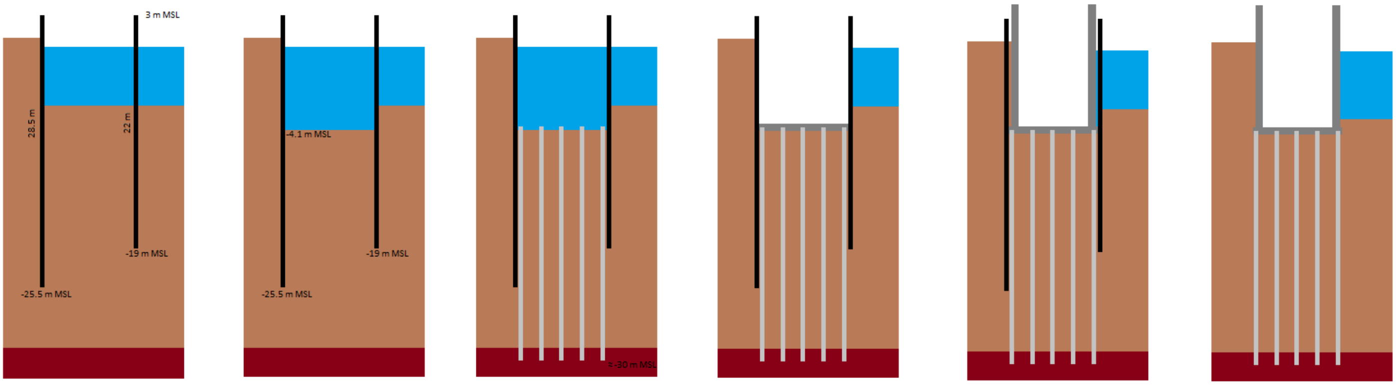


Figure 26: Phases of the building pit

8.5 Dewatering the building pit

When the concrete floor is finished it will work as a strut. A calculation shows that the sheet pile at the quay side will have a FOS of 1.24 when the pit is pumped dry. The other sheet pile has a FOS of 1.27 in this situation. Therefore the pit can now made dry without the possibility of instability of the sheet piles. Another calculation is done to ensure the floor does not float in that situation. This situation is normative and therefore if this situation is safe, all situations are. Because the floor of the small lock is the smallest this will be the normative lock. The pressure at the top of layer 4a should be smaller than the total weight of the soil and floor above. In this case the piles also add some strength to the structure because they are driven into this soil layer. The water pressure at this depth is 30*10 = 300 kN/m^2. The calculation of the soil weight is shown in Table 33. Because the downward force is 548 kN/m^2 the floor will not float. The new situation is shown in the fourth frame of Figure 26.

Layer	Weight	Top	Bottom	Thickness	Force	Total:
	[kN/m3]	[m MSL]	[m MSL]	[m]	[kN/m2]	[kN/m2]
Concrete	23	-3.5	-3.9	0.4	9.2	9.2
Layer 1	15	-3.9	-18	14.1	211.5	220.7
Layer 2	18.5	-18	-23	5	92.5	313.2
Layer 3	19.6	-23	-35	12	235.2	548.4

Table 33: Calculation floating

8.6 Construction of the locks

Now that the building pits are dry the walls and doors of the locks can be constructed. It is important to notice that at some spots the reinforcements should be placed thinking a few steps ahead. The reinforcements for the walls should already be placed at the right spots in the UWC floor and the connection reinforcements between the locks and the culvert dam should be placed during the construction of the walls. When the locks are finished the sheet piles can be removed.

8.7 Backfilling and removing of the sheet piles

Before removing the sheet piles at the quay side of the lock it is important to backfill the space between the sheet pile and the wall of the lock as shown in the fifth frame of Figure 26. At the water sides this is not needed. After removal of the sheet piles the locks should be ready to use, navigation and outflow of water should be possible using only the locks, as the culverts are probably only needed in an extreme situation in the future. The chance of this extreme situation happening during construction time is very low. The sixth frame of Figure 26 shows the finished lock.

8.8 Construction of the culvert

The construction of the culvert follows the same method as shown for the locks. For this situation both sheet piles should reach until MSL - 19 m, therefore these sheet piles can be used from the water sides of the locks. Beneath the culvert no piles are installed, the sheet pile preventing piping should be placed from MSL - 4.4 m until MSL - 13.7 m. The UWC floor should be poured from MSL - 4.5 m to MSL - 3.5 m. The connection with both locks should be cleaned before the concrete is poured. All the sheet piles except the ones used to prevent piping can be taken out and resold.

8.9 Conclusion

Following the steps explained before the total construction can be built without much obstruction for navigation. To prevent risks a FOS of 1.1 is used as a minimum for all situations. During the detailed design some improvements to this plan can be made, but this plan shows a good general method of constructing the flood barrier and locks.

9. Additional calculations

In this chapter more research is done into specific areas of the chosen variant with two locks and a dam with culverts. Failure of the structure, the mooring places for ships, the need for pumps and different gates for the culverts are for instance investigated.

9.1 Failure of the structure

The risk of failure has to be known for the chosen alternative, risk is consequence multiplied by probability. The consequences are out of the scope of this project, so only the probability of failure is determined. This hydraulic structure is a system that can fail in case of the exceedance of an ultimate limit state, this could be due to three main errors. If a human, defense or mechanical error occurs, the system fails and Ho Chi Minh City is flooded. A fault tree as displayed in figure 27 allows combining these failures in one approach. The probabilities of failure are an indication because they cannot be determined exactly. This paragraph should give insight, as complete as possible, into all possible failure mechanisms of the hydraulic structure. The embankments are out of the scope of this project because the design of it is not started yet by the institute and/or building companies.

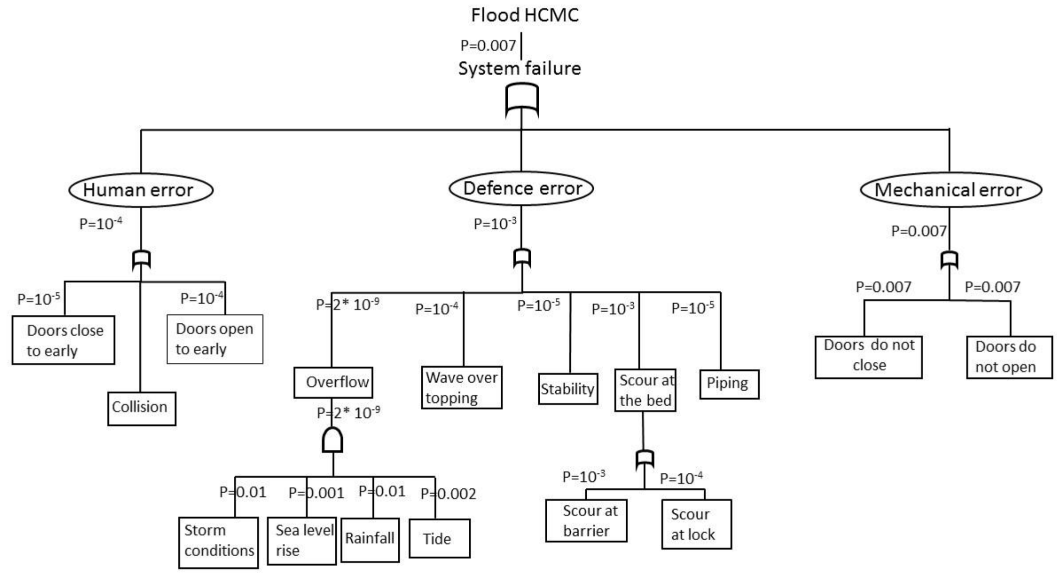


Figure 27: Fault tree failure mechanism

9.1.1 Defence errors

The failure mechanisms for the defence system that could occur at this location are shown in figure 27. The derivation of this fault tree is explained in appendix N.1. The probability of each failure mechanism in the system is explained in this paragraph.

Overflow

To achieve a maximum water level in 2100 of MSL +3.62m all four elements as can be seen in table 34 have to reach their maximum value. Because the barrier is MSL+4m, the water level which causes overflow has to be 0.38 m or higher. Because the sea level rise is an uncertain factor, it is assumed to rise with +1.15 m for a probability of 0.001 instead of +0.77m with a probability of 0.002. Because of the 'AND' gate, all probabilities are multiplied by each other. This results in a probability of $P=2*10^{-9}$. in the year 2100.

Element	Maximum value	Probability of occurrence	Comment
Storm conditions	+0.70 m	0.01	Previous 100 years this was the highest storm measured in the area around Ho Chi Minh City, so not at Phu Dinh.
Sea level rise	+0.77 m	0.002	Data provided by the IWER
	+1.15 m	0.001	Assumed probability for higher value
Rainfall	+0.18 m	0.01	Previous 100 years this was the highest rainfall intensity for one day measured in the area around Phu Dinh.
Tide	+1.97 m	0.002	Data provided by the IWER

Table 34: Overflow

Wave overtopping

The dam, barrier and locks are a vertical wall so wave overtopping is limited, the maximum wave height at the Phu Dinh barrier caused by wind waves is taken into account with the storm conditions. So a water level of MSL +4 m has to be reached, which is determined to have a probability of P=0.014 and then the storm has to be even worse to create wave overtopping. This probability is low and assumed to be $P=10^{-4}$.

Stability

If the structure is vertically, horizontally or rotationally not stable this is encountered by the piles that resist these forces. In chapter 6.3 a more detailed calculation is presented about the overall stability. The bearing capacity is most likely to fail first but only has a probability of $P=10^{-5}$ due to the safety factors that are taken into account. During maintenance the probability of failure for tension is higher than $P= 10^{-5}$, but a combination of tension failure and high water which could cause flooding is lower than $P=10^{-5}$. So an indication is $P=10^{-5}$.

Scour of the bed

Erosion occurs where the bed consists of fine sediment, scour is a special case of erosion in which the local transport exceeds the supply from upstream. The difference in transport can be caused by either a difference in velocity, turbulence or both. A solution to prevent damage to the structure is to construct scour protection.
The main function of scour protection, is not to minimize scour, but to keep the scour hole far away from the dam that needs protection. A scour hole will be formed downstream of the scour protection and when the scour hole gets to deep failure will occur. The length of the scour protection should be sufficient to prevent any damage to the dam, barrier or lock by sliding after failure.
The equations used to determine the scour depth are all presented in the updated version of the book 'introduction to bed, bank and shore protection' written by Schiereck and Verhagen (2012). In the remaining of this paragraph it is referred to as 'the book'.

Bed protection at the locks

Bed protection in front of the navigation locks is necessary for two reasons. Firstly, the emptying of the lock chamber causes a water jet outside the lock towards the bottom and a strong deceleration of the water near the bed which causes turbulence. At this point a bed protection is often applied to prevent river bed erosion.
Secondly, the propeller wash of the ships when moving at low speeds can damage the bed. A bed protection should be applied in the waiting area where the ships are waiting to enter the lock. The complete formulas, calculations and method can be found in appendix N. This section gives a concise overview of the conclusions of the calculations and the measures that have to be taken for the bed protection in front of the locks.

Emptying of chamber

The emptying of the lock chambers happens through circular filling openings in the doors of 2.5 m^2 per door. The water flowing through these openings when emptying can be schematized as a circular water jet, circular jets reduce the velocity faster than plane jets, see appendix N.2. Two situations are investigated, in the first situation the outside water level is larger than the inside water level (HLD (Head Level Difference) of 3 m) which corresponds with a maximum velocity of 6.32 m/s at the bottom at a distance of 11 m from the openings. In the second situation the outside water level is smaller than the inside water level (HLD of 1.4m) which corresponds to a maximum velocity of 4.32 m/s at the bottom. With an Izbash-type formula the stone dimension needed for the first situation is 0.86 m ($d_{n50} \rightarrow$ class 1000 – 3000 kg) and for the second situation 0.40 m ($d_{n50} \rightarrow$ 60 – 300 kg).

Ship propeller wash

The ships propeller wash can also be schematized as a waterjet which is partly directed towards the bottom. The maximum velocity of the propeller wash is based on the power of the engine, which is based on the dimensions of the governing ship. This maximum velocity is 6.18 m/s. The maximum velocity at the bottom is calculated afterwards and is 2.11 m/s. This velocity is increased due to turbulence and used in an Izbash-type formula to determine the required stone dimension on the bottom. The required stone dimension is 0.36 m ($d_{n50} \rightarrow$ 60 – 300 kg).

Location and type of bed protection
In Figure 28 the lay-out of the bed protection and the governing situation can be found. The length needed to resist the emptying jet is 25 meters of 1000 – 3000 kg. This is a quite big stone dimension with a low probability of failure because the governing situation only occurs at maximum head level difference which does not occur very often. It can be chosen to use a smaller stone class with a somewhat higher probability of failure which asks for more frequent maintenance. After the 25 meters of 1000 – 3000 kg stones, 60 – 300 kg stones are used to resist the ships propeller wash. The lay-out of the bed protection is the same for the three locks because the same governing situation occurs. However, the lengths of the 60 – 300 kg stones differ. For the big locks, this length is 263 m and for the small lock, this length is 140 m, as derived in the section about the waiting area of the ships in chapter 10.2.

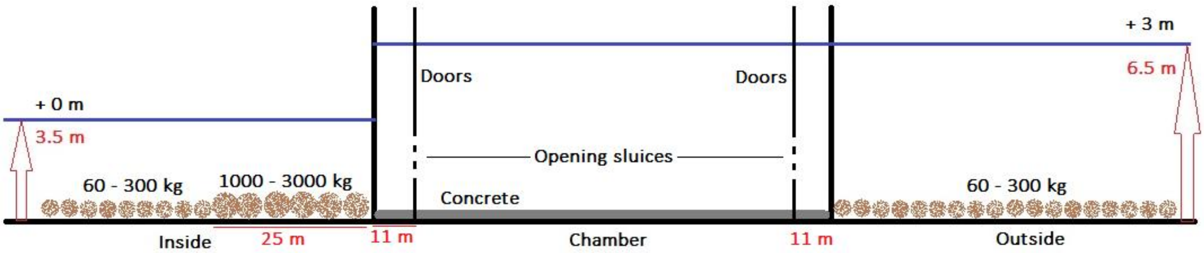


Figure 28: Governing head level difference (outside level > inside level) and bed protection lay out

In Figure 29, the situation is shown where the outside water level is smaller than the inside water level. In this case, a head level difference of 1.4 m occurs which asks for stones of 60 – 300 kg when emptying the lock chamber, however, these stones were already needed for the ships propeller wash so this does not change the bed protection lay-out.

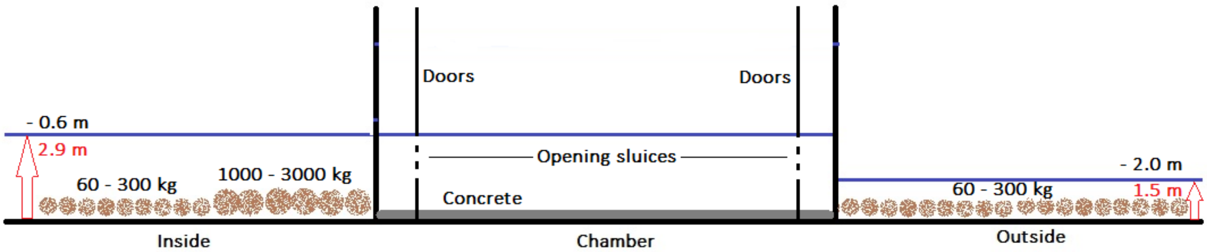


Figure 29: Governing head level difference (outside level < inside level) and bed protection lay out

When it is decided to use no bed protection at all, large scour holes will occur in the proximity of the lock heads. This can be very dangerous for the foundation of the lock and for the stability of the sheet pile embankment. So it is advised to use bed protection. For small scale projects like this project, loose rock and/or fascine mattresses will do. The client told that gabions of about 0.5 height (with stones of 0.20 cm and a geotextile) and interlocking concrete blocks are often used as a bed protection in Vietnam. The idea of fascine mattresses in combination with a geotextile and the required stone size were introduced and the client agreed that this is a good solution but that is not used very often.
It is advised to use these fascine mattresses because the durability of gabions cannot be guaranteed due to the poor water quality. However it should be investigated if the water quality is a problem for the gabions.
What should be investigated also is the need for a mattress. A geotextile is always used because of the fine sand/clay at the bottom and a mattress can be useful to keep the bed protection as a whole in the right place and to divide the weight of the stones during the long lifetime of the structure. However this need for a mattress should be investigated. If it is not needed, the maximum drop height of the stones should be calculated.

The probability of failure of the bed protection near the locks is quite low because all velocities are maximum velocities and the situations are governing. The stones will be big enough to resist the flow forces and it is not expected that the situations will be more extreme in the future. Therefore a probability of failure is set at 10^{-4}.

Bed protection at dam

The dam is a hydraulic structure with a potential of scour. There is a 2 meter high vertical constriction for the flow from MSL -5.5m until MSL -3.5 m. This is created by the floor and sheet piles below the culverts. The horizontal constriction is the decreasing width from 66 meter to 44 meter, the horizontal view of the dam can be seen in Figure 30.

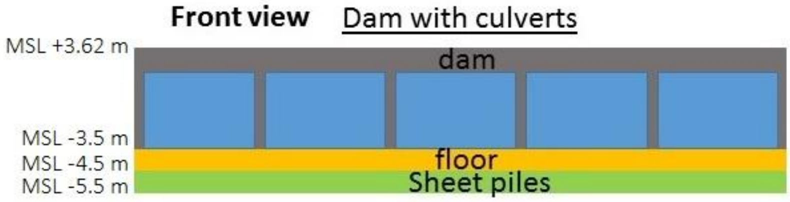


Figure 30: Front view

Scour depth

From a dimensional analysis and many experiments, the following expression was finally developed for clear-water scour behind a bed protection:

$$h_s(t) = \frac{(\alpha\bar{u} - \overline{u_c})^{1.7} * {h_o}^{0,2}}{10\Delta^{0,7}} * t^{0,4} = \frac{(2.2 * 1{,}89 - 0{,}15)^{1.7} * 5.5^{0,2}}{10 * 1{,}5^{0,7}} * 15^{0,4} = 3.3\,meter\ (\text{Eq. }4.13)$$

It is relevant to protect the hydraulic structure against scour because it is a significant depth. The several variables are explained in appendix N.5 as is the derivation of their value. There are a few critical notes about this formula:

- This is a 2D formula in which the horizontal constriction is not taken into account. This constriction is included in the velocity above the sill.
- The scour depth formula is an approximation about the scour depth, it does not say where the scour will occur and it does not take into account lateral stability, but as a first estimate it is a useful formula.
- The scour depth formula is reliable for non-cohesive material. The bed material is clay, a cohesive material. According to mister Van Den Bos (TU Delft) there are no existing formulas to determine the scour depth with a cohesive bed. It can be assumed that the variables C, Δ, u_c, have different values for cohesive material. This makes the formula more useful with cohesive material and their values can be found in appendix N.5
- It is uncertain where the scour will occur when the barrier or culverts open again. It is likely that the circumstances are the same and that the created scour hole will become deeper, but this is not proven. When the times of opening can be added up in the variable 't', the scour hole will become deeper. Due to this uncertainty, it is not known at which depth the scour will stabilise.

The formula of Trinh Cong Van is used, this formula presents the stabilised scour depth in a Mekong Delta river. An advantage is that it is not time dependent and because this is an uncertain factor in the previous displayed equation this formula is applied to determine the scour depth (Hm). The variables and the method to determine the scour depth using Excel are explained in appendix N.6.

$$Hm = \left(1 + \left(\frac{L_{gc}}{H_h}\right)^{-0.33} * \frac{U}{U_{kx}} - 1\right) * H_h$$

$$Hm = \left(1 + \left(\frac{3 * Hm}{5.5}\right)^{-0.33} * \frac{1.8}{0.35} - 1\right) * 5.5 = 11.6\ m$$

Length of scour protection

When the slope becomes too steep with a certain scour depth, it is no longer able to withstand the gravitational forces and it will slide. Clay is densely packed, a slope of 1:3 is assumed after the sliding because densely packed sand has a slope of 1:6 after sliding as is stated in figure 4-21 of the book. Mister Trinh Cong Van confirmed that a slope of 1:3 in the Mekong Delta is a good estimate.
The length of the scour protection has to be longer than the recession length to prevent damage to the dam or barrier. A margin of 20% is chosen, this is due to:

- A safety margin for the parameters of the scour depth
- Possibility for extreme events
- A higher possible slope than 1:3

Length scour protection = Hm * slope * safety factor = 11.6*3*1.2 = 41 meter.

Bed protection

The type of bed protection has to be determined by an iterative process. A Chezy value is assumed and the dn_{50} is calculated, then a new Chezy value is calculated, these iterative steps continue. The stones in the bed protection are ranged between 90 and 250 mm. A more detailed explanation can be found in appendix N.9. A combination of vertical and horizontal constriction should be investigated with a 3D mathematical model, this will give more reliable stone sizes for the bed. A safe opportunity is to choose bigger stones than the proposed 90 to 250 mm, for example 40-200 kg.
For stones with a size of 90 to 250 mm a probability of $P=10^{-3}$ is assumed. The water level is assumed to be maximum and safety margins are included but on the other hand there is an uncertainty about the applied method so a P of 10^{-3} is assumed. In appendix N.8 can be found how this failure probability is tried to be calculated using the software Prob2B.

Piping

Piping is the flow of water through a pipe like channel that has been created by internal erosion. The barrier is closed in rain season for about 15 hours a day, this duration of the water level difference is sufficiently long to start uplift. The bed material is made of impermeable clay which decreases the probability of piping, on the other hand the water level difference could be quite high. E.W. Lane based a theory about how long the seepage length has to be in order to prevent piping. According to Lane (Molenaar & Voorendt, 2016):

$L \geq y * C_l * \Delta H$
Where:
L= Seepage length
Y= safety factor
C_l= Constant of lane (depending on soil type)
ΔH= differential head

$L \geq 1.5 * 8.5 * 2.62 = 33.4$ m

The seepage length of the dam is: $(4.5 + 3.26) + \frac{1}{3} *1 + (4.5 + 1) = 14$ m, so this is 18.4 meter too less seepage length to prevent piping.
Cut off walls will be implemented under the dam and barrier to reduce the groundwater flow velocities by elongating the seepage path or to block the seepage in the dike cross section entirely. The walls will be implemented at a depth of M.S.L. – 4.5 meter with a length of 9.2 meter, so piping is prevented. For alternative B the length of the walls has to be 8.2 meter.
A cut off wall is implemented, the bed material is made of clay and the water level difference reaches very seldom a maximum so it can be concluded that piping is unlikely to take place, so $P=10^{-5}$.

The piping probability for the locks is even smaller than for the dam and barrier. This is because the seepage length is a lot longer due to the concrete floor. Though, there are sheet piles installed below the lock to prevent seepage that starts at the barrier, goes under the lock and continues to the other side of the barrier.

Human and mechanical error

Alternative A does not contain a barrier but a dam, which has less possible human and mechanical failure mechanisms. On the other hand, alternative A consists of two locks instead of one lock, this increases the possible human and mechanical failure mechanisms. So it is assumed that the probability of human and mechanical failure is equal for both alternatives.

Human error

From analysis of accidents it appears that human and organizational errors are still a major cause of failure in civil engineering. It seems that the only suitable way to reduce human errors is by the incorporation of sufficient control in the different phases of the construction process (Taerwe, 1986) and by a thorough education of all personnel involved. Therefore, an extensive interaction between the safety methodology and the quality management is a necessity in order to guarantee the safety of the hydraulic structure (Lecture Notes Probabilistic Design & Risk Management, 2015).
There are two human errors possible even though the staff is trained well:

- The doors close too early during lowering the water level in HCMC, which results in to little storage capacity during high water in HCMC.
- The doors open to early during a high water level outside HCMC, this causes the water level in HCMC to rise and when heavy rainfall will occur the city will flood.

These two listed errors could be subdivided into several causes, but this is out of the scope of this project. To stimulate navigation at Phu Dinh it is more likely that the doors are opened too early than that they are closed to early. So the probability of closing the doors too early is assumed to be: 10^{-5} and the probability of opening the doors too early is assumed to be 10^{-4}.

Collision

Another human error that can occur, is the collision of a ship and the lock. There is no historical data available about the probability of a collision in the Mekong Delta. There is a small variety of literature available, but the presented values in these papers deviate a lot. So no probability of failure is added to collision.

Mechanical error

Mechanical errors are errors which are outside the area of influence of humans and can hardly be controlled. For this project, the main mechanical error is failure of the doors, such that the doors won't open or that they won't close. It is hard to determine a realistic probability of failure for the doors in the alternatives in this project. Therefore a reasonable estimate should be done. For the new waterway storm surge barrier in the Netherlands, the derived reliability targets were (Campbell, 2001)

- Probability of not closing due to human or technical errors less than 10^{-3} on demand
- Probability of not opening due to human or technical errors less than 10^{-4} on demand

But these probabilities are target probabilities and the fault tree consists of probabilities that say something about the actual probability of failure. For the Seven Mile Dam in British Columbia the reliability analysis resulted in:

- Probability of failure of spillway gates to open due to electrical/mechanical failures $2.07*10^{-7}$
- Probability of power supply unavailability to the spillway gates $2.07*10^{-7}$

For the Thames Barrier, the probability of failure of opening the gate was $1.55*10^{-4}$ per gate per demand. These examples help to make a good estimation of the probability of failure of not opening or closing the doors. Besides, a good industrial system standard is one failure in 10^{-4} per demand (Campbell, 2001).

For this project, a probability of failure per demand of 10^{-5} for closing and opening is chosen because the complexity of the structure is lower than for the Thames barrier or the new waterway storm surge barrier. This probability of 10^{-5} per demand causes a probability of failure of 0.007 (1-((1-0.00001)^700) per year (assumed that the barrier closes 700 times each year) for closing of the gate. This causes a probability of 50% (1-((1-0.007)^100) that the gate fails to close during the lifetime which seems acceptable. The same calculation can be made for opening of the gate, so that the probability of failure per year is also 0.007 for opening the gates.

Overview of failure

The displayed probabilities in the fault tree provide an indication of the likeliness of failure. As can be seen, the probability of a mechanical failure is most likely, this is $7*10^{-3}$ in the year 2100. Assuming that between the years 2018 and 2068 the yearly probability of failure is $7*10^{-4}$ and in the years 2068-2118 the yearly probability of failure is $7*10^{-3}$. These probabilities differ because the materials get older and the water levels higher. The probability of one failure due to a mechanical error during 100 years is:
P_f= 1-(1-P)^n = 1-(1-7*10^{-4})^{50} + 1-(1-7*10^{-3})^{50})=(1-0.965) + (1-0.704) = 0.33

This failure probability is considered low. In the detailed design the client can make an adjustment for the height or dimensions of the structure when considering this probability to low and make a more cost efficient design.

Internal flooding

An important note is that the probability of internal flooding is not considered as a system failure, though this causes a higher probability of flooding than the external possibilities. In the year 2100 the tide is 0.7 meter higher than today. In the year 2100 the doors have to close earlier at about M.S.L. -0.3m, this provides a maximum storage capacity of 1.3m. Once in the twelve years a peak intensity of rainfall occurs together with the highest tide possible and causes a flood. They have to take place together, so the probability is not 1/12 but computed to be 1/30. More information about this calculation can be found in appendix F.A.

Total probability of failure

Combining the internal probability of flooding and the external of flooding leads to a total probability of failure of: 1/30 + 0.007 = 1/25, so once in the twenty five years.

9.2 Navigational Overview

In Appendix L it is shown that the following navigational model has been determined. The illustration below indicates all the paths of the vessels through the Phu Dinh project and their orientation by use of colors. Here the red color shows the path of vessels moving out of the city whereas the yellow color illustrates the path of the vessels navigating into the city, the areas where these paths cross one another are shown in orange. The green areas illustrate the waiting areas where the vessels can line up before entering the locks. The navigational overview seen in the picture below is only considering traffic when the big lock/tidal barrier has to operate as a lock, forming a closed system in order to maintain water level differences on either sides of the structure.

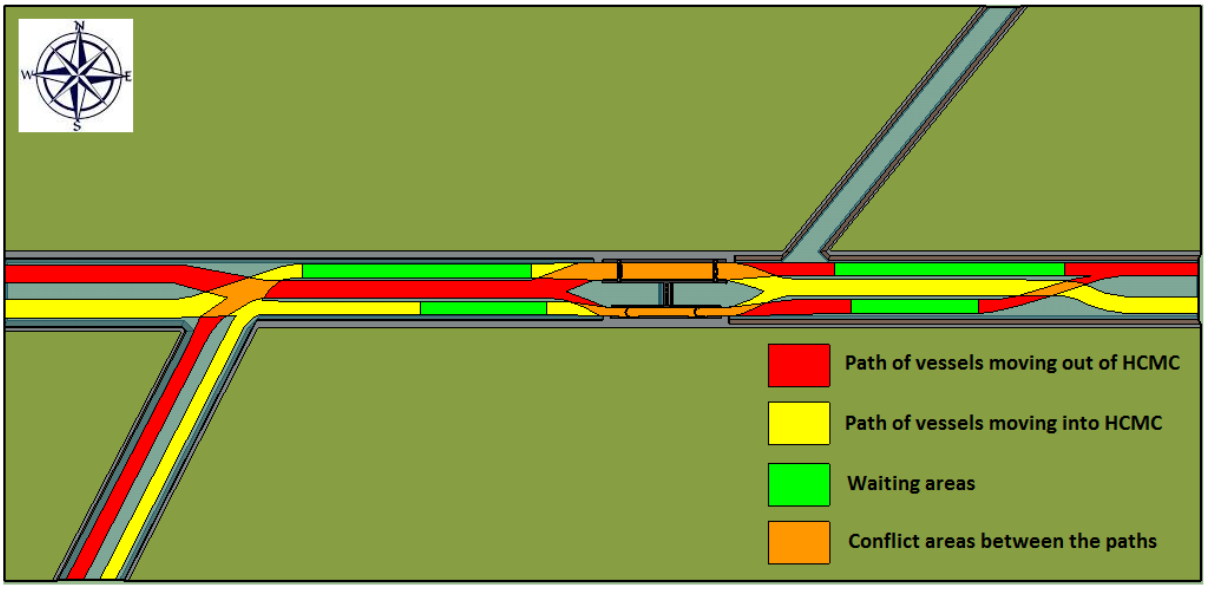


Figure 31: Navigational overview around the barrier

In Chapter 4.1.5 it is explained that for this variant the maximum waiting time that can occur is 3 hours for one barge. The barges can only go through the big lock which has a cycle period of 1 hour. From this information it is concluded that the waiting area in front of the large lock has to be two times the size of the lock plus the length of the concerning barge, leading to a length of $2 * 111 + 41 = 263m$. All boats capable of going through the small lock have a maximum waiting time that is twice the cycle time of the small lock, this means that the waiting area for the small lock has to be double the size of the lock leading to a length of $2 * 70 = 140m$.

9.3 Additional pumps in the design

In the current construction of the Phu Dinh barrier pumps are built. In both variants of the design there are no pumps because it is assumed unnecessary. Although to reduce the chances for a flooding on the inside due to rain, pumps can be used. In this paragraph a small calculation is performed to show how strong pumps should be to make a difference. Assumed are a storage area of 1.5 km^2 and a maximal closed shift of 8 hours of the lock during one tide. It can be calculated how fast the locks should pump in order to reduce the inside water level by 0.1 meter.

$$Q_{pump} = \frac{V}{T_{hours}} = \frac{(A_{storage} * \Delta h)}{T_{hours}} = \frac{1500000 * 0.1}{8} = 18750\frac{m^3}{h} = 5.2\frac{m^3}{s}$$

When the storage area is assumed constant with height, which in reality is not the case, this means that during one tidal wave, the water level lowers 0.1 meter for each 5 m^3/s the pump can pump. When applying pumps that can pump 6 m^3/s each, a total of 6 pumps can reduce the water level $\frac{36}{52} = 0.7\ m$. Therefore looking at the analysis in chapter 4.2.2, the risks of a flooding due to rain in the future becomes much lower placing pumps. Because the risks in the current situation are already very low it is recommended to design the culverts in such a way that pumps can be installed when the risks become too high in the future. In this way the costs of the barrier are spread and the pumps do not wear or decay in a period when they are not really needed.

9.4 Culvert gates

The dam with culverts has so far been assumed, as can be seen in chapter 5.2.4. The assumed dimensions are 31.4 meters in length, 8.5 meters high and 2 m wide. Culverts are assumed from MSL +1 m to MSL -3.5 m. The 5 culverts are parted by 4 slabs of concrete with a width of 0.6 meters.
So far the doors have been assumed to be hydraulic lift gates. This is a concise investigation into the differences between a lift gate and a flap gate.

Lift gates

The basic idea of lift gates is that these gates are operated by a hydraulic cylinder. This means that the gates can be opened as much as needed. The size of the gap that the water has to pass can be varied in this way. This is a benefit of lifting gates. The big negative aspect of lift gates is that they have to be operated using a system and every operation requires energy input.

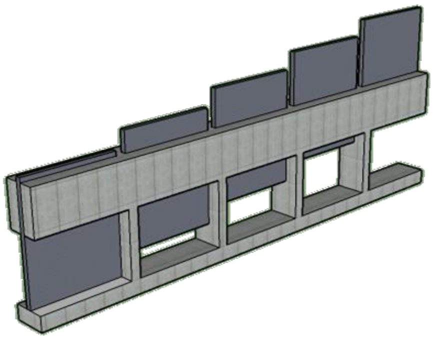


Figure 32: Dam with lift gates

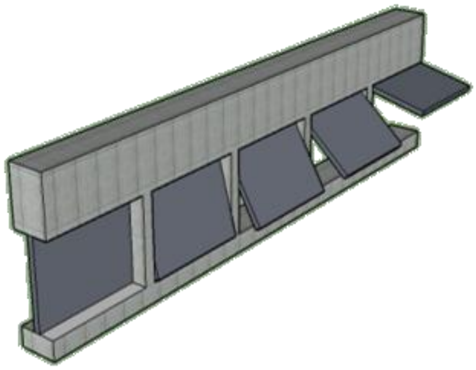


Figure 33: Dam with flap gates

Flap gates

The second option for the gates in the culvert is flap gates. These gates basically are operated by using gravity. The difference in water pressure will lead to the outflow of water. It has to be made sure that the gate is attached on the Mekong Delta side of the dam.
If high water is present at the outside of the barrier, this pressure will lead to the closing of the flap gate. When the water on the inside of the structure is higher than outside, the water pressure will open the gate and thus water will be let go, from inside to the outside.
There will only be one problem when using this gate. In the case that water has to be retained on the inside of the structure, for instance to keep the water level at MSL – 0.6 m, the gate must retain the water on the inside. This can be done, using a lock, which prevents the flap gate from opening and thus from releasing water. Another negative aspect of flap gates is that they cannot be used to let water flow to the inside of the structure.

Conclusion

The lift gates can be used to control the water level very accurate, however they will require power and programs to be used. The flap gates are operated using gravity, this leads to a low level of operating skill needed, however they must be locked at the right time, in order to retain water on the inside of the structure. The flap gates seem the more attractive option, because no power is needed to operate them.

10. Final recommendations and conclusions

The final recommendations and the conclusions of the project team are presented in this chapter.

10.1 Recommendations

The Phu Dinh Barrier will be part of an entire coherent system of various other hydraulic structures around the city. Throughout the development of the project, the design team has come across various limitations concerning the understanding of the data, methodologies applied and the coherence of the project to the entire flood defence plan for the city. Also the time frame of the project has brought up some limitations to the design. In this chapter the recommendations concerning different topics from the project are established.

10.1.1 Embankment

As to now the only thing being designed is the tidal barrier along with a structure to ensure navigation, however what is not designed is, are the embankments along the river sides. If high water would occur the hydraulic structures might be able to hold the water but this would be ineffective as it could just flow around it. Therefore the main recommendation is to increase the height of the river banks on the Mekong Delta side to the same level (MSL + 4 m) as that of the closed barrier.

10.1.2 Catchment area

The entire flood defence project is based on the MARD plan which has been reduced to the MARD plan variant which has then been reduced to a newer variant being built now. For the MARD Plan and its variant an entire hydraulic study has been done concerning the catchment area and the storage area within the barriers. For the project as it is being built now this is not done and through the studies in this report it is found that if the storage area is too small the consequences could be severe for the in case of heavy rainfall. A main recommendation is to closely investigate the water storage capacity in the city when all barriers are closed and if not large enough apply pumps or even create more storage capacity.

10.1.3 Sediment

No data was available concerning the sediment (distribution) in the river. As a solution to make sure the depth of the river is maintained at a level between MSL -5.5m and MSL -3.5m dredging is proposed. For dredging and for the bed protection, certain approximations have to be made for the dn_{50}. A recommendation is to find out more about the sediment characteristics in the river. It may also reduce the costs for the bed and bank protection and these can now be determined more accurately.

10.1.4 Bridge

The outcome variant of this report creates great possibilities for the construction of a bridge spanning over the river. This would lead to great transport benefits for overland transport for walking and motorbikes, as there are little bridges in the area. It would also take away the need for the now existing ferry service across the river which would lead to a reduction in the navigational intensity of the river.

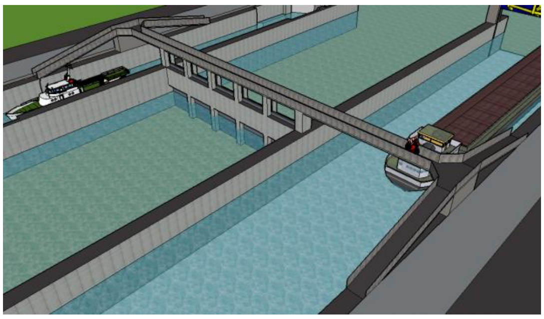


Figure 34: Possibility of a bridge across the locks

10.1.5 Lock software

The intensities per hour per ship class were measured for six days, this short time frame is considered unreliable. The intensities per hour are very useful to verify whether the capacity of the lock is big enough. It is recommended to simulate the transport flow and the lock capacity to check whether there is no development of long waiting lines. Also the influence of several dimensions on the waiting line is relevant. An example of software that could be used is Sivak, provided by Rijkswaterstaat (Netherlands).

10.1.6 Scour

For a combination of horizontal and vertical constriction it is complicated to determine accurately the bottom protection. Investigation in a scale model or a 3D mathematical model will be necessary to provide more accurate values for the bottom protection.

10.1.7 CPT

The soil research at the site has been done using the Standard Penetration Test (SPT). It is found that this method is becoming more and more out-dated and in order to find out more about the soil bearing capacities it is advised to use the Cone Penetration Test (CPT).

10.2 Comparison current design (IWER) and design of the project team

A sub goal of this project is to compare the design of the IWER to the design developed in this report. Attention is paid in this paragraph towards the two most important aspects: prevent flooding of HCMC and the navigation should be hindered barely. Also the differences in the basic elements in the designs are discussed in this paragraph.

10.2.1 Flooding

The altitude of the hydraulic structure designed by IWER is MSL +3.7m, the altitude of the hydraulic structure designed in this project is MSL +4.0m. The maximum water level in 2100 could be MSL +3.62 m, so though these values suppose one design prevents HCMC better from flooding, the higher altitude has no influence for the year 2100. All values above MSL +3.62 m are just an extra safety margin and can expand the lifetime of the structure.

The barrier designed by IWER consists of one lift gate, the dam with culverts is two metre thick. The risk of failure of the lift gate seems higher, this increases the probability of a flood in HCMC. The probability that one culvert fails is about the same as that a lift gate fails, but the consequence of one failing culvert is lower.

An internal flooding in HCMC due to rainfall is more likely to occur than an external flooding because the catchment area is too big in comparison to the storage area for the river at Phu Dinh. The outflow possibilities of the design by IWER is 40 meter wide, the outflow of the design in this report is 51 meter. The lock with vertical lift gates and the dam with culverts make a higher outflow of the water possible in the variant developed design by the project team, this decreases the probability of flooding.

10.2.2 Navigation

The design by IWER consists of one lock and a tidal barrier, the design by the project team consists of a dam and two locks. One of these two locks has vertical lift gates and can function as a barrier. An overview of the locks in both designs is displayed in Table 35.

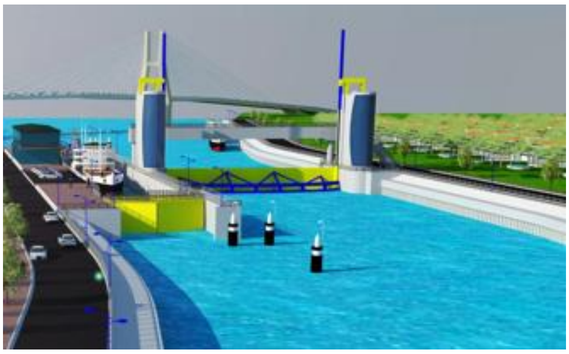
IWER	**Dimensions (m)**	**Gate type**	**Ships/hour**	**Design vessels**	**Max waiting time**
Lock	15 X 100	Mitre	19.2	1	About 2 hours
Project team				**Design vessels**	
Lock 1	21 x 111	Lift	15.2	2	1 barge 3 hours, others 2 hours or less
Lock 2	70 x 10	Mitre	10.5	1	

Table 35: Lock comparison

Navigation in the Phu Dinh river consists of 50% barges and 50% boats, of all barges 20% is 700DWT or higher. The design of the IWER does not let the barges pass through the lock, while in the design of the project team only barges larger than 700DWT are prohibited to pass through the lock. This is a difference of 40% of the fleet.

10.2.3 General

The layout of the two designs is completely different due to the different approaches of having one lock and a barrier or having two lock in combination with a dam. The dam consists of culverts and the big lock can be used to regulate to water level. Both layouts can be seen in figure 35 below. It is recommended to investigate the option of constructing a bridge over the dam and locks during the detailed design.



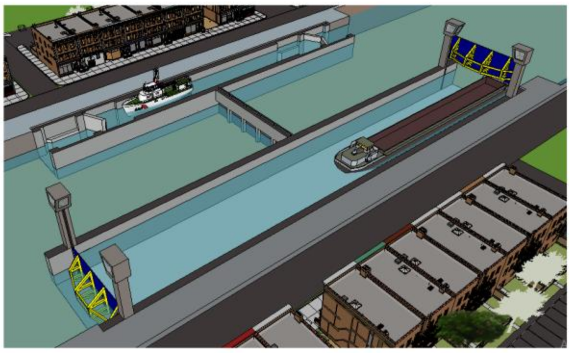


Figure 35: Comparison of the lay outs

10.3 Conclusion

The final design had to meet the following functional requirements. The structure must be able to close at MSL + 0.6 m in the wet season. In the dry season the minimum water level that may not be exceeded is MSL - 0.6 m and the maximum water level on the inside of the structure when the structure is closed may not exceed MSL + 1 m. Another requirement is that navigation must always be possible through the Phu Dinh river and the final requirement is a lifetime of 100 years.

The developed design with two locks and a culvert dam, meets these requirements. HCMC will not be flooded and navigation will be hindered barely. The doors can be operated in a way that the maximum and minimum water levels can be retained. The locks provide a possibility to navigate both during closure and opening of the barrier. The culverts in the dam provide the extra needed conveyance area, to guarantee a sufficient outflow of rain water.

The specifications of the final design are as follows:

- The width of the total structure is 66 meters.
- The **big lock** is 23 meters wide, 120 meters long, has a floor of 0.6 m thick and walls of 1 m thick. It is founded on 120 concrete piles (400*400 mm). Two vertical lift gates with a weight of 2940 kg can be lifted to a height of 6 m above MSL.
- The **small lock** is 11.6 meters wide, 85 meters long, has a 0.4 m thick floor, walls of 0.8 m thick and is founded on 45 piles. It consists of 8 mitre gates, in order to retain the water at both sides.
- The remaining width of 31.4 meters is taken by the **dam**, which is 2 meters thick. In the dam 5 culverts are 4.5 by 5.8 meters big.
- The river bed is protected against high water flow speed by a bed protection of 41 meters from the dam and for the total waiting area at both locks.

The remaining risk of a flooding due to rainwater on the inside of the structure is once every 30 years. The probability of failure leading to a flood due to a high outside water level is once every 140 years. The resulting probability of a flood thus becomes once every 25 years.

10.4 Discussion

During this project, the project team encountered problems or situations where improvisation was needed. This section describes these problems and tries to give a possible solution for these situations in future projects.

Piled foundation

The piles chosen for the final design are driven prestressed concrete piles. For this choice, it is assumed that driving piles is possible in this area. When driving piles is not possible, screwed piles can be used as an alternative. It must be taken into account that for most of the soil capacities of the pile the capacities of a screwed pile should be taken 0.9 for that of the driven piles. Another disadvantage of screwed piles is that they cannot be prestressed which asks for an increase of the reinforcement percentage.

Land subsidence

The sea level rise of 0.7 m was assumed to include the land subsidence of HCMC as well. After a meeting with Royal HaskoningDHV, it became uncertain if the land subsidence is also included in this value. Since land subsidence is a very important factor, this phenomenon should be investigated more, for a future project.

MCA criteria 'Stakeholders'

In the first MCA, the criterion 'stakeholders' received the lowest weight factor. In the second MCA, the criterion 'environment' received the lowest weight factor. During discussions it became clear that these criteria were seen as the least important ones by the project team but at some moments it also became clear that these criteria were quite important. After the first MCA was finished, the supervisor from IWER told that stakeholders, and especially the local people, are very important in Vietnam for a successful project. Also the deputy director of the HCMC Flood Management program from RoyalHaskoningDHV stated that the satisfaction of local people is necessary for the public support of the project. For this project it was hard to get a good insight of the wishes and influence of the stakeholders, but for future projects this is something to take care off.

Wall thickness calculation

The calculations of the wall thicknesses of the locks are based on a method from the manual Hydraulic Structures which is used for the calculation of concrete beams. For this method a thickness is assumed and afterwards it is checked whether this thickness can resist the bending moment in combination with a certain reinforcement percentage. This reinforcement percentage should lie between a minimum and a maximum value. It is not sure if this method is the best one to use, because it is originally used for beams.

Piles beneath the floor

In the final design, quite some foundation piles have to be used beneath the lock floor. When the floor is dimensioned a bit thicker, this causes a significant reduction of the amount of piles that have to be used. In the current situation the floor is dimensioned based on tension. A thicker floor causes more downward forces, resulting in a lower number of tension piles needed for the foundation. The process of dimensioning the floor could be executed in a more iterative way.

Criteria MCA

The selection of the criteria for the MCA is something the project group is not sure of. It is hard to determine suitable selection criteria and especially the determination of the corresponding weight factors for each criterion asks for some level of experience. This causes also uncertainty in selecting the best alternative(s). The MCA (criteria and weight factors) was discussed with the client which gave some good insights but experience in executing MCA's is really necessary. In a next project, it can be useful to discuss the criteria for the MCA extensively with the client in an early stage or discuss whether a MCA is the best method to choose alternatives.

Differences between alternatives

In the beginning of the project, 36 alternatives were considered of which 17 remain after a first selection. It was hard to filter the best options from the 36 alternatives because the alternatives had so less mutual differences. Another problem that rose was the selection of alternatives after they were scored in the MCA. Because the alternatives had such small differences, the scores did not differ significantly from each other which made it hard to point out the best options. For a future project, it can be useful to come up with more distinctive alternative.

Communication

Communication between the project team and the client (IWER) went not completely perfect at the beginning of the project. Some miscommunications about the data caused that the project team continued working with a wrong understanding of data or requirements and therefore some content had to be rewritten. This difficulty in communication was nearly impossible to solve because the discussions in English could not have been executed better. A part of the solution can be that the client prepares a document with important information, situation description, data and requirements that follow a clear systematical approach and give a total overview of the available data.

Bed protection methods

For the determination of the bed protection, a lot of assumptions have been done. Maybe the level of detail the project team wanted for this section was too high for this preliminary design. Crucial data such as the grain size of the sediment in the river was not available for this section and a proper method/formula to include a vertical and horizontal constriction for the bed protection of the barrier missed. The project team tried to give an overview of the failure probabilities in a fault tree, but due to all the assumptions and the lack of some data and formulas it is hard to say something about the certainty of the failure probabilities.

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Appendix

Appendices of the report of the conceptual design of a flood-retaining hydraulic structure at Phú Dinh.

Index

Appendix A – Reference projects

An overview of the seven reference projects that are investigated:

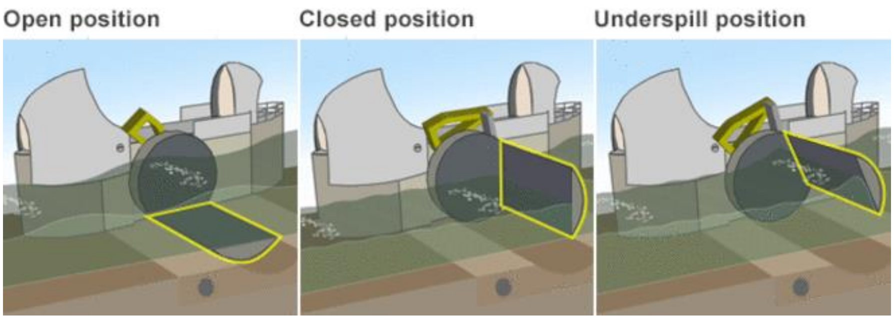
Maeslantkering

The Maeslantkering is a tidal barrier that is crucial in the Dutch delta plan. The gates are two circular shapes that when open rest in the sides of the river. The round shape is very beneficial for the loads in the steel construction, minimizing bending moments in the sheets and transferring these into normal forces in the structure. The gates are rotated inwards for closure. This mechanism is both suitable for a tidal barrier as well as the closure mechanism for a lock.

Span: 400m
Cost: € 450million
Cost/m:€ 1.125.000/m
Closure: Only in severe conditions, long closure time

Thames Barrier

The Thames Barrier is one of the main flood defences for London. It is a very remarkable structure due to its original closure mechanism as shown in the figure below. The figure shows the three possible positions of the gate, capable of closely regulating the outflow of the river.



Span:	525m
Cost:	Back in construction period in 1984, € 634million, today's price is estimated at €1540 million
Cost/m:	€3.000.000/m
Closure:	Only in severe conditions

Balgstuw Ramspol

Another alternative for a closure barrier is the Balgstuw. This tidal barrier consists of a large rubber balloon that lies on the bottom of the river when open and is filled with air when closed. The structure uses the buoyancy of the air-filled rubber tubes to form barrier.

Span: 70m
Cost: €96 million
Cost/m: €1.400.000/m
Closing time: max. 60 min.
Opening time: max. 180 min.

Hartelkering

Two large vertical gates form the Hartelkering. This structure serves as a component of the Dutch Delta Plan. Due to the large size of the gates they are developed as large framework with a round shape on both sides.

Span: 150 m (short span: 50m, long span 100m)
Cost: €200 million
Closure: Quick

Hollandse IJssel

The lock/barrier complex 'Hollandse IJssel' is a barrier in the Hollandse IJssel which serves to prevent sea water flooding into the Netherlands at high water situations. It's the oldest hydraulic structure of the 'Delta works' and closes five to six times per year.

Span barrier:	80 m
Span lock:	24 m
Cost:	€154 million
Closure:	Quick

T.J. O'Brien Lock and Dam

Another combination of a barrier in combination with a navigation lock is the T.J. O'Brien Lock and Dam. In this case the dam has the possibility of letting water through or being closed. All ship navigation always has to go through the lock

Span:	90 m
Cost:	€56 million
Closure:	Quick

The Boat Conveyor

This is a concept worked out by the company Witteveen en Bos. According to them this is a very simple and non-expensive alternative for a navigation lock. The boat is lifted up by the conveyor which is capable of transporting it from one waterway to the other. The system is powered by electro engines that have a very low loud emission. The belts are also capable of storing solar energy increasing the efficiency of the system.

This is not constructed so cost, closure and span estimation is unknown.

Appendix B – Navigational overview

Overland traffic

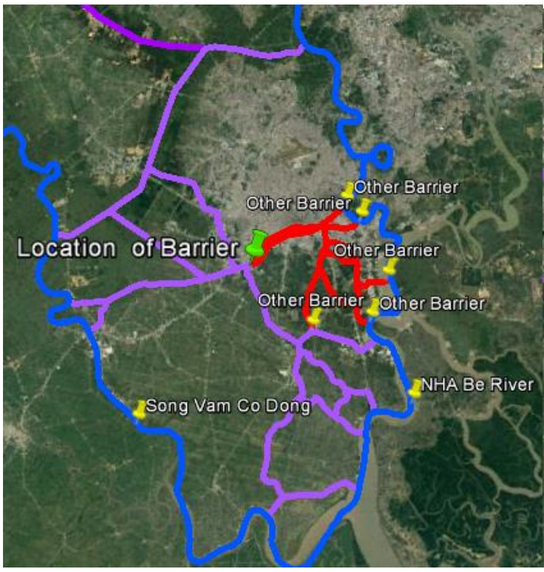
The barrier area is surrounded by a highway illustrated in the picture below. These surrounding roads contain bridges spanning over the various channels/rivers in the area. The barrier is about in the middle of the two bridges.

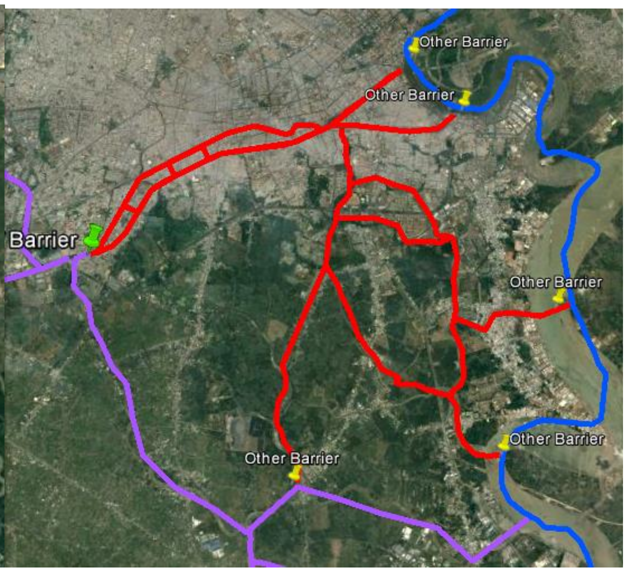
Right downstream of the barrier is a ferry service crossing the rivers. This ferry service is both available for motorbikes as pedestrians.

If the shipping rates and intensity in the river increase in the future it is interesting to seek for an alternative with a barrier/bridge combination. This would take away the ferry crossings reducing the obstructions in the river during the high intensity periods and would also benefit the accessibility for pedestrian and motor traffic.

Shipping

The barrier will be positioned in a busy waterway where about 180 ships pass by on an average daily basis. From the map below it can be seen that the location of the barrier serves as a crucial entrance to the city from the eastern waterways. The canal also creates a shortcut for vessels travelling from the Song Vam Co Dong to the NHA Be River. The blue lines display the two rivers, these are connected by a network of other rivers and canals displayed in purple. The closed off waterways are shown in red.





From the left picture it can be seen that shipping traversing HCMC to further destinations would not be affected by the closure of the tidal barrier as much since there would be enough alternate routes around the city. From the right picture however it seems clear that the barrier is located at a crucial position for the entrance of the city since it is the only eastern entrance. It is estimated that travel times would increase with a factor 3 for ships using this entrance to enter the city.

It can be concluded that the barrier is located at a crucial position for the accessibility of HCMC by boat. For this reason it is desired to be capable of keeping the barrier as open as possible to navigation for both the construction period as well as during the lifetime of the barrier.

Appendix C – Structural elements

1. Structure types

To regulate water levels and offer the possibility for navigation several structures can be thought of, namely a tidal barrier, closure dam and navigational structures. Depending on the functional requirements and boundary conditions these structures can either fulfill the desired purpose individually or in a combination. In this chapter an overview will be provided of possible navigational structures and closure mechanisms for the hydraulic structures.

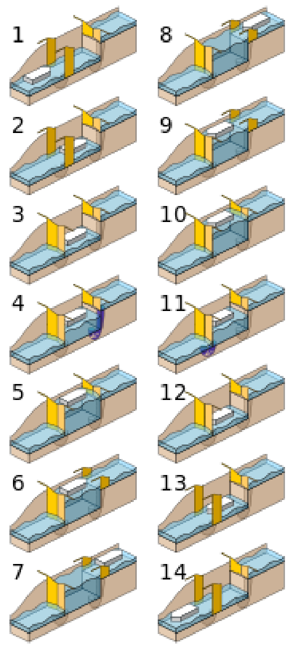
1.1 Tidal barriers and closure dams

A tidal barrier is a structure that is equipped with movable gates capable of allowing the free flow of a river or blocking the flow completely. Depending on the gates the tidal barrier can become an obstruction for shipping when opened or allow free navigation. An alternative for the tidal barrier would be a closure dam. A closure dam can also be equipped with various water bypasses such as culverts. A major difference between tidal barriers and closure dams is that dams will not be capable of allowing complete free flow in the river when water passes through, also shipping is not possible. Concerning the free flow it is important to know that the closure dam will not be able to allow as much opened area for flow as the tidal barrier resulting in higher flow velocities through the dam. This can result in unfavorable backwater curves and form restrictions for the maximum discharge of the river.
However, closure dams can be constructed much cheaper than tidal barriers and are easier to construct, this leads to significant lower costs.

1.2 Navigational structures with water level differences

The most common navigational structures to ensure shipping traffic through water level differences is the navigation lock. The principle of a navigation lock is to create an inner chamber for the ships that can be completely closed by two gates and where the water level can be controlled. **Error! Not a valid bookmark self-reference.** on the right shows how such a structure is capable of either bringing ships from high water levels to low water levels as from low water levels to high water levels:

1. ship approaches the lock
2. ship enters the lock chamber
3. gates of the lock close
4. water level in chamber is adjusted
5. water level in chamber is equal to desired height
6. gates open
7. ship departs the lock



Appendix C - figure 1: Ship through a lock

The closing structures and gate types of this kind of constructions can be very similar to those of tidal gates and are treated in chapter 3.2.
The other alternatives for navigational purposes are the ship elevator and the ship conveyor. For the ship conveyor the ship is placed on a conveyor belt lifting it out of the water, over the barrier and into the water on the other side. The ship lift is basically an elevator that consists of a chamber in which the ship enters and that is lifted or lowered into position. This system would use either a large counterweight or would be functional as a dual system in order to reduce the energy consumption. This can be seen in **Error! Reference source not found.**.

Appendix C - figure 2: Falkirk wheel and boat conveyor

2. Structure types

A basic overview of different gates and other closure mechanisms which can be used for the tidal barrier or the lock, is given here. This overview is mostly based on the manual of the course hydraulic structures of the TU Delft (Molenaar W. ,., 2011) and then specifically the lock part. The main considerations for the different gates and closure mechanisms are presented here. These considerations will later on be taken into account when deciding what doors will be used for the structure at Phu Dinh.

1. Mitre gates

Mitre gates consist of two doors, symmetrical with regard to the center line of the lock, rotating around the vertical axis. The force resisting principle is very simple; in closed position both leaves point in the direction of the upper water level. On a global level the force of the water pressure results in normal forces in the gates and spalling forces acting on the lock head.

At local force level bending moments and shear forces are introduced in the doors as well.Mitre gates in general cannot resist significant current or wave forces when partially open. For this reason mitre gates are *unsuitable for use as a weir or flood gate* but are frequently used as lock gates. In some cases, when for instance hydraulic cylinders are used to open and close the gates, the gates be made to resist a small reverse water head difference.
In open position the gates are turned in recesses in the walls of the lock head. In order to prevent damage when vessels navigate through the lock head, there should be enough distance between the gate and the face of the recess wall, or fenders should be attached either to the gate or the face of the recess wall. This will prevent the gate from being pushed into the wall, by passing ships.
The width of mitre gates is limited to approximately 20-25 meter (10-12.5 m per door).

The main advantages of mitre gates are:
• No air draught limitations. High superstructures such as lifting towers or pillars are not needed, so boats are not limited in height.
• Low cost, effective solution for smaller locks. The three-hinged mechanics scheme results in a relatively light gate structure.

• The equipment for mitre gates does not have to carry the weight of the gate; the centre of gravity remains in a horizontal plane during opening and closing. Resulting stresses during operation are reduced to those due to frictional forces at the pivots, socket and collar strap and the dynamic effect of moving the gate through the water.
• Opening and closing of the gates takes a short time.

The main disadvantages of mitre gates are:
• The length of the lock chamber needs lengthening as well as the upper head by the recesses of the gates.
• To retain water in both directions a double set of doors is required.
• Very accurate mounting and frequent checking of the contact at points of load transmission (thrust blocks and quoin blocks) is necessary.
• Mitre gates cannot be opened or closed under a water head difference
• Operation may be disturbed by debris and ice, ice is not really a problem in Ho Chi Minh, debris however is present in the river.

2. Single leaf gate

The single leaf gate is basically half a mitre gate, appearance, shape and materials used are similar, however, there are some significant differences. Where the mitre gate transfers the hydrostatic load for a considerable part by normal forces, the single leaf gate transfers all the forces by bending moments.
The single leaf gate is most frequently used for locks having a small width; this is mostly the case for the locking of recreational craft.

The main advantages of a single leaf gate are:
• Very suitable for locks of smaller width because of simple construction and operation.
• No air draught limitations.
• It is possible to lock the door on the free end, this makes it possible to retain water in two directions.
• Forces on the gates are transferred parallel to the lock wall (when the closed leaf gate is perpendicular to the lock axis).

The main disadvantages of a single leaf gate are:
• Recesses for single leaf gates are longer than those for equivalent mitre gates, thus single leaf gates need longer structures.
• Opening and closing of the gate results in a lot of water displacement.
• More rigid and thus heavy construction needed for the pivots, socket and collar strap.
• Vulnerability for ice and debris
• Not suitable for wide locks because very heavy supports will be needed.

3. Lift gate

Lift gates spanning the entire width of the lock chamber and operating in vertical recesses enable hydrostatic loads to be transmitted to the lock head. Lifting the gate opens the lock. The deadweight of the gate is balanced by counterweights in order to reduce the operating forces of the equipment. High gantries or guide towers are required in most cases to guide the gate when moved into open position.

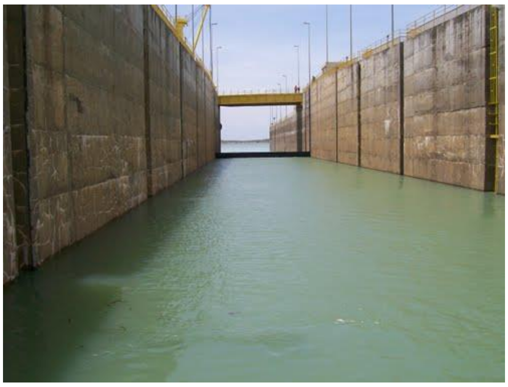
The main advantages of lift gates are:

• Little space is needed for the gates, thus the total structure length is reduced to a minimum.
• It is possible to open the gate under a water head because the force directions are in different planes.
• Not seriously affected by debris and ice in closed or opened position.
• Easy to control and repair.
• The support system of the gate is statically determinate and therefore largely insensitive to differential settlement.
• It is possible to retain water in two directions.

The main disadvantages of lift gates are:

• Air draught limitation of vessels.
• The superstructure, lift towers or columns, is large and heavy.
• Lifting the whole gate results in very strong underflow, leading to scour.
• Spillage of water on vessels passing the gate.
• Difficulties with the roller tracks because of the deflection of the bottom of the gates when they are under full head.
• Balancing the gate may be complicated and expensive.
• Opening the gate takes relatively long.

4. Submersible lift gate



The submersible lift gate is lowered for the passage of ships. The first phase of lowering can be used for filling and emptying the chamber. For purposes of repair the gate is lifted above the water level.
The submersible gate takes little horizontal space and has the advantage that vessel height is not limited. Water can be retained in two directions and strong currents at the bottom of the door.
The main disadvantage however, is that a large recess deep into the ground is needed to accommodate the gate and its construction and maintenance costs are high.

5. Rolling-, sliding or caisson gate

The caisson, rolling or sliding lock gate is preferred when large spans have to be bridged without limiting the air draught. This type of gate can only be used when sufficient space exists besides the lock, because the width of the lock head structure is about two times the doors length.
The rolling gate moves on underwater rails or sliding tracks, which bears the risk of the gate getting jammed due to accumulation of sediments or debris on those tracks. Especially if the gate frequently remains in the open position it should be considered to wipe the rail or track clean before or during gate operation. Moving the gate, reducing weight and frictional forces, thus reducing wear and tear as well, is easier when buoyancy tanks are used. When the gate is in closed position the tanks can be filled, now they serve as ballast tanks, to ensure the necessary contact pressure with the bottom.
A combination of factors results in the generally considerable thickness of rolling gates. Obviously one factor is the large width of the lock to be spanned by the gate. The need for stability results in wheels or slides on both sides of the gate and two rails or sliding tracks under one door, not just one in the center of the door, which adds to the thickness. Every so often the thickness of rolling gates can be used to provide passage to light traffic.
In closed position the rolling gate can be considered as a simply supported beam; in fully opened position there are hardly any horizontal forces on the door. In those positions stability of the gate is not a problem.
During the opening or closing process the 'upper' corner of the gate is completely unsupported. A force on this door area, e.g. due to a remaining water level difference or large flow, may result in significant stability problems. Keeping the resulting vertical force within a distance of 1/6 of the gate's thickness to the center line of the gate is a good measure for providing stability, but requires a certain thickness of the gate.

The advantages of rolling lock gates are:
- Suitable for locks of large width.
- No air draught limitation.
- It is possible to retain water in two directions.
- Easy to maintain.
- Opening and closing relatively fast

Disadvantages:
- Requires a lot of space besides the lock (head).
- Requires a large recess or lock head.
- Cannot be opened or closed under water head conditions.

6. Radial gate / sector gate

Radial gates with a vertical axis of rotation are often referred to as sector gates. The skin plate of the gate has the curved shape of a (part of) a cylindrical circle, and has to be stiffened to resist the hydrostatic water pressures. The lower and upper gate arms, triangular shaped trusses in the horizontal plane, support the stiffened skin plate horizontally and vertically. Between the upper and lower arm there will be diagonal braces or struts, to provide the required vertical stiffness. At their rear ends the gate arms transfer the loads to pivot points, which are casted into the lock head wall. Due to the shape of the skin plate the resulting force of the hydrostatic pressure has its working line through the pivot.
Sector gates have the advantage of being statistically determinate, can resist reverse heads and can be operated whether there is a water head or not. The weight of the gates may exceed that of the equivalent mitre gates, making them more expensive. Another cost setback are the large and deep recesses in the lock heads. Sector gates are more often used for guard locks or storm surge barriers where closure under free flow and unlimited air draught is needed.

The main advantages of sector gates are:
• The possibility to close the gate in flow conditions.
• Relatively easy to move because it cuts through the water with a small surface.
• No air draught limitations.
• The equipment for operating the gates does not have to carry the weight of the gate; the center of gravity remains in a horizontal plane during opening and closing. Frictional forces at the pivots and the sealing result in horizontal forces.
• Opening and closing of the gates takes a short time.
• It is possible to retain water in two directions.

The main disadvantages of sector gates are:
• Relatively large amount of material because of the developed length of skin plate and the gate arm trusses that have to be braced.
• High support loads at the pivots resulting in heavy cast-in items in the lock head wall due to weight of the gate. To mitigate this disadvantage float elements assembled to or integrated in the gate maybe used.
• The recesses required to move the gates out of the way are large and substantially increase the size of the lock head structure sideways.
• Operation may be disturbed by debris and ice.

7. Radial gate / tainter gate

Radial gates with a horizontal axis of rotation are frequently known as tainter gates.
Reference is made to the descriptions of the sector gate in the above.

Compared to sector gates the tainter gate:

• A single tainter gate spans the whole width of the lock head opening. Generally two sector gates are used.

Lifting to open for navigation:

• In lifted position there will be a limitation to the air draught of passing vessels.

• The equipment has to be able to carry the weight of the gate. Counter weights may be used to mitigate this disadvantage.

Lowering to open for navigation:

• Lowered there is no air draught limitation

• To lower the gate a deep recess in the lock head is required, significantly increasing the lock head structure, complicating construction because of the much deeper foundation, hence considerably increasing costs.

• Again, the equipment has to carry the weight of the gate. Besides counter weights, floating elements assembled to the gate may be used as well to mitigate this disadvantage. It is possible to use this type of gate for guard gates, flood defense or storm surge barriers, provided the gate body is designed high and strong enough to resist underflow and overflow.

The basic comparison between different gates

In the table below different gates are compared for the different categories. Green areas mean that this type of gate has no problem in that category; red areas mean that the type of gate is limited in operation by the given category.

Gates	Airdraft limitation	Extra vertical space	Extra horizontal space	Operate water level difference	Retains 2 directions	Vulnerable to debris
Mitre	No	No	Yes	No	No	Yes
Single leaf	No	No	Yes	Possible	Yes	Yes
Lift gate	Yes	Yes	No	Yes	Yes	No
Submersible lift gate	No	Yes	No	Yes	Yes	No
Rolling gate	No	No	Yes	Yes	Yes	Yes
Radial gate	No	No	Yes	Yes	Yes	Yes
Tainter gate	Possible	Yes	No	Yes	No	Yes

Now three less standard types of gate/weir alternatives will be mentioned:

8. Inflatable weir

Inflatable weirs or rubber dams are long tubular-shaped weirs used in channels, streams and rivers. They raise the water level when they are inflated. The outer membrane is a multi-layer fabric made of synthetic fiber which may be rubberized or has a coating. Because of the high flexibility of the material an inflatable weir can resist very large resistances. In open channel flow, inflatable weirs are used to divert water for irrigation, for raising existing dams or for flood controls. Inflatable weirs can be filled with water, air or both. Air is preferred because then the dam can be deflated or inflated more rapidly and the dam is little affected by freezing conditions.

The main advantages of inflatable weirs are:

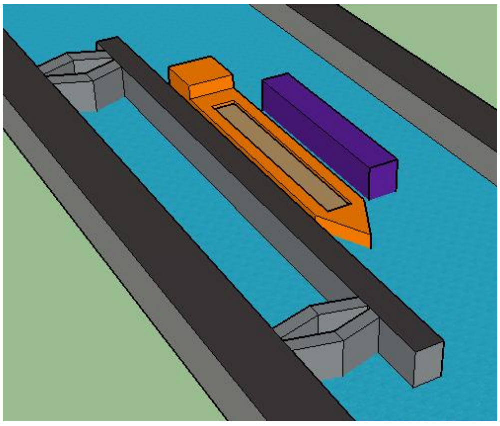
- Lower initial costs than conventional concrete dams with metal gates
- Lower maintenance costs due to the lack of gate mechanisms and need to paint
- Lower environmental impact when deflated because the sills can be constructed to conform the existing channel
- Depending on sill geometry and water level, fish passage may be possible over a deflated dam
- Low profile allows passage of flood flows with a minimal increase in upstream stage
- Low concrete sill enables bed load and suspended sediment to pass the deflated dam, reducing deposition
- When the dam is fully inflated, there is no seepage through side or bottom seals, as is often the case with conventional steel gates

The main disadvantages of inflatable weirs are:

- Shorter design life
- Vulnerability to vandalism
- Uncertainty to the newness of technology
- Dam may be damaged by debris and ice

9. Rotating barrier

The idea of a rotating weir came up during a brainstorm session. The concept is relatively easy and cheap but we think that realization is quite hard and it is undesirable when the barrier is constantly in the navigation channel.



The rotating barrier consists of a rectangle barrier which is placed in the longitudinal direction of the channel. When it is opened, so without flood conditions, ships are able to pass the barrier at both sides. When the barrier is closing, an engine drives an axis in the middle of the barrier to rotate the barrier. This is the hard part: the engine must create a very large moment to rotate the barrier and to resist the forces of the flowing water. Some solutions we have been thinking of is the use of two steel cables at both ends of the barrier (at the bottom) which can be rolled around a winch. Another solution is the use of rails on the bottom of the channel bed to guide the barrier. When the barrier is fully closed, hydraulic pins are pushed into openings in the sides of the barrier to prevent the barrier from rotating due to the forces of the flowing water. Watertight connections should be made of rubber flaps.

10. Flip-up barrier

The flip-up barrier is a relatively new type of flood control barrier. It is applied in the lagoon of Venice to protect the city of Venice against floodings coming from the Adriatic sea. The main reason to use a barrier of this type is to ensure that the aesthetics and view of the environment won't be harmed. This can be realized due to the fact that when the barrier does not have to operate, its whole structure is lying flat on the bottom. The principle of the flip-up barrier at Venice is as follows: 78 mobile gates are being laid on the seabed in the inlet channels of the lagoon. They are supported by long steel and concrete pilings driven into the lagoon bed. The mobile floodgates (largest are 29.5 * 20 * 4.5 meters, smallest are 18.5 * 20 * 3.6 meters) consist of a metal box structure in which compressed air is pumped when a tide of more than 110 cm is expected. The air causes the mobile floodgates to rise up to the surface of the water and block the tidal flow coming from the sea.

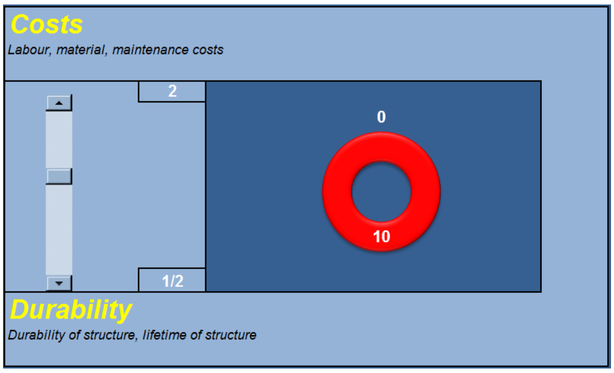
The barrier at Phu Dinh is significantly smaller than the Venice Lagoon Barrier but it is still interesting to investigate the possibilities of this flood barrier

Appendix D – Explanation of weight factors for the MCA

The first step can be found in figure 1. This step serves to get a clear overview of the main criteria (gunningscriteria) and the subcriteria (toelichting) for the project team/client before giving weights to the criteria.

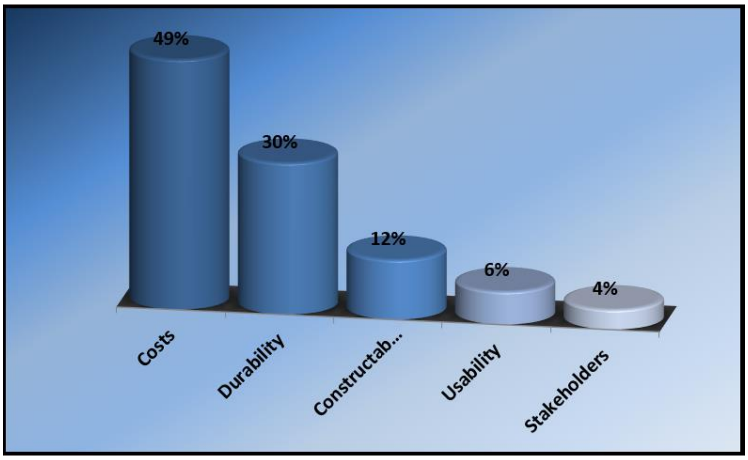
	Gunningscriteria	Toelichting
criterium 1	Costs	Labour, material, maintenance costs
criterium 2	Durability	Durability of structure, lifetime of structure
criterium 3	Constructability	Expertise/experience needed, temporary structures, complexity of structure
criterium 4	Usability	Navigation/water regulating function, adjustability, operability
criterium 5	Stakeholders	Satisfaction of stakeholders, aesthetics/landscape
criterium 6		

Appendix D - figure 1: Main criteria for the MCA



Appendix D - figure 2: Pairwise comparison

The next step is to do the pairwise comparisons. The 5 criteria give 10 combinations which have to be evaluated. By doing the comparison, it is all about proportions. It is possible to say whether for example costs are more important than durability. How much more important we as project team think that costs are in relation to durability is impossible to estimate well. That is why for each comparison there is a choice whether it is equally important or one criterion is twice as important. For example: giving costs 2 and durability 1/2, means that costs are more important than durability.



Appendix D - figure 3: Possible result for weight factors of the different criteria

Eventually the pairwise comparison calculates the weight factors. The comparison will be done by several people. All these weight factors will be averaged to obtain the final weight factors which will be used in the MCA.

Appendix E

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Appendix F.A – Waterflow through a barrier

Water flow

Matlab script

Goal:

The flow through the tidal barrier is calculated using a Matlab script in order to obtain two goals:

- Easily see how much time the barrier is closed and/or is not available for navigation;
- Calculate if the conveyance area of the barrier is large enough to discharge the rainwater during the opening times in the rain season.

To find this the result of the Matlab script should be a graph that shows the water level inside and outside the barrier in time.

Input data

The input consists of the following data, a more thorough description on how it is obtained can be found in appendix F.B.

- Conveyance area of the gate (possibly dependent on the water level);
- Tidal waves outside of the barrier;
- The normative rains and the area over which it should be discharged with this barrier;
- The relative storage area for this barrier;

Requirements

The following requirements are given in the Matlab script to simulate the use of the barrier:

- The barrier closes at MSL + 1.0 meter and MSL + 0.6 meter in respectively the dry and rain season;
- The barrier closes when the water level becomes lower than -0.6 meter;
- There is no navigation possible when the flow speed of the water is too high.

Working of the script

To find the wanted results the Matlab script uses a differential equation. This equation relates the water height and the flow through the barrier as follows:

$$v(t) = \sqrt{\Delta h(t) * 2g} = \sqrt{h_{in}(t) - h_{out}(t) * 2g}$$

$$Q_{out}(t) = v(t) * A_{conveyance}(t) = v(t) * B_{width} * h_{in}(t)$$

$$\frac{dh_{in}}{dt} = \frac{\left(Q_{rain}(t) - Q_{out}(t)\right)}{A_{storage}} = \frac{\left((q_{rain}(t) * A_{rain}) - Q_{out}(t)\right)}{A_{storage}}$$

The formula for h_{out} is a combination of sinus and cosine functions based on the tidal data provided by the Thuy Loi University. The total formula can be found in appendix F.B. In Matlab these formulas are used to calculate the results with the Runga Kutta method. The final Matlab script can be found in the same appendix.

Results

The results of the calculation provide a graph that shows the inner and outer water level in time. Additional a graph is given that shows the possibility of navigation in time.

Flood protection

Before making a design it is important to look at the minimum conveyance capacity of the barrier. In this chapter the width of the barrier in relation with the water height and rain in the normative situation is analysed. This is done by looking at rain events that happen once in 100 years (180 mm/day) and once in 20 years (130 mm/day). The origin of the values for the areas is explained in appendix F.B. The width of the barrier is taken at different values to get a general understanding of needed design values.

To find the most extreme situation, a lot of factors need to be taken in to account. The following questions need to be asked:

- When the peak rain intensity starts in comparison with the tide;
- What the rain intensity is before the peak rain;
- If the peak rain can be foreseen in all cases;
- If the tidal barrier is used in the most effective way;

These questions make it hard to find the normative situation, because the worst case scenario will be a peak rain intensity that is unforeseen and at the strikes at the worst moment possible. While the peak rain happens only once in a 100 years, the probability of the combination of these worst case factors is very low. Therefore the analysis of this problem will be focussed on the situation where the barrier is used in the most effective way and the peak rain can be foreseen. The chance of the peak rain occurring at the worst possible moment in a month is about 1/8.

Current sea level

In general it can be said that during the closing period the water level in the storage area should not exceed the level of 1.0 meters above MSL. When the width of the gate does not limit the outflow, the gate can be closed at the minimum water level of the tide. From the moment the gate closes until it opens again the difference in water level in the storage area can be calculated easily:

$$\Delta h = T_{closed} * q_{peak,rain} * \frac{A_{rain}}{A_{storage}} = T_{closed} * \frac{0.18}{24} * \frac{50 * 10^6}{3 * 10^6} = T_{closed} * 0.125$$

This shows that the rise of the water level will be 12.5 cm each hour. When looking at the minimum water level during a cycle in the rainy season, the highest minimum found is around 0.6 meter above MSL. Closing the gate at that minimum will certainly lead to a flooding, therefore in that situation the gate has to be closed at the previous minimum, which is around -1.0 m MSL. In this situation the water level can rise 2.0 meters before flooding occurs, but the time the barrier is closed can become around 16 hours. This means that the water level rises 2.0 meters, just on the edge of a flooding. It should be noted that this situation will take place when the rain once in 100 years takes place at the worst possible moment (1/8 chance), but with the best use of the barrier. The probability of this situation to happen will therefore be lower than 1/500 each year.

When looking at the rain intensity that happens once in 20 years (130 mm/day) the probability increases, but there will be no flood. Therefore it can be said that with unlimited width of the barrier the probability of a flooding will be much lower than 1/100 each year.

Sea level rise

When taking into account a sea level rise of 0.7 meter, the situation becomes totally different. During the peak rain intensity 1/100 years this will lead to a water level of 1.7 m MSL, which is 0.7 meters above the stated maximum. It must be noted that this will not become 1.7 m MSL in reality, because when flooding occurs, the storage area will rise due to the flooding. The effects of a flooding, major or minor, are not discussed in this analysis.

The peak rain intensity 1/20 years will lead to the following water level:

$$h_{peak,in} = h_{min} + \Delta h = h_{min} + T_{closed} * q_{peak,rain} * \frac{A_{rain}}{A_{storage}} = -0.3 + 1.44 = 1.14$$

This shows that the peak rain once in 20 years will lead to flooding too. The maximum rain intensity at which flooding occurs at a sea level rise becomes:

$$h_{peak,in} = h_{min} + \Delta h = -0.3 + \Delta h = 1.0 => \Delta h = 1.3$$

$$q_{peak,rain} = \frac{1.3}{T_{closed}} * \frac{A_{storage}}{A_{rain}} = 0.117$$

So at a rainfall of 117 mm/day a flooding will not occur. As shown in appendix F.B the probability for this rainfall to occur becomes:

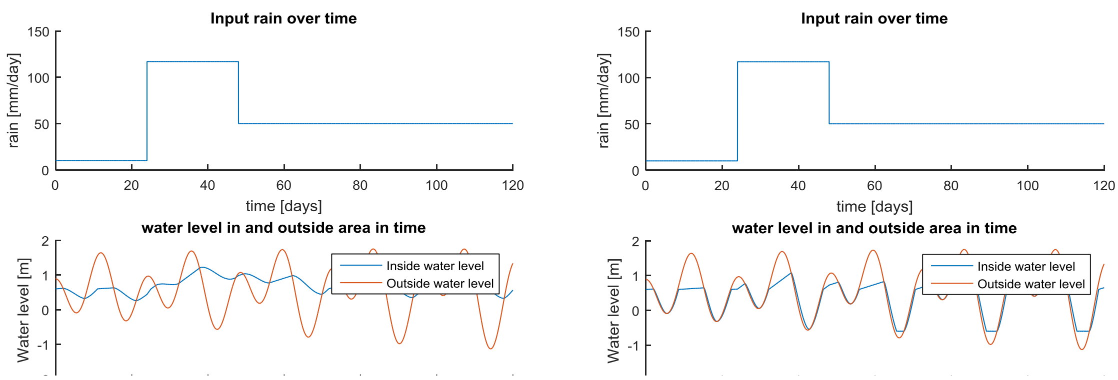
$$P = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(X-\mu)^2}{2\sigma^2}} \qquad \mu = 10, \qquad \sigma = 50, \qquad X = 117$$

$$P = \frac{1}{125} e^{-\frac{11449}{5000}} \approx 8\%$$

Concluding it can be said that this peak rain can occur once every 12 years, when the peak rain is higher it also leads to flooding, even when it not happens at the worst possible moment. This analysis does not take into account the possible increase of rain in the future and the other barriers that will have their effect on the total system. Also the risk has to be taken into account that the barrier is not used optimal. Resulting in a probability of flooding that will be between 1/12 years and 1/100 years. An estimated guess leads to a flooding once in 30 years when the sea level rise of 0.7 meters occurs.

Width of the barrier

In case of peak rain fall it is important to open the gate at the minimum level possible. When the width of the barrier is limited, the water level inside will follow the outer water level with a delay. This influences the minimal water level at which the barrier can be closed, and therefore influences the probability of a flooding. It is desirable to keep the delay in water level as small as possible, and therefore



Appendix F.A - figure 1: Water delay at barrier of 5m (left) and 60m (right) width

find a desirable minimum in the width of the barrier. The Matlab script presented in appendix F.B shows how fast the inner water level follows the outer water level. In Appendix F.A - figure 1 the water levels are shown at a relative small and wide barrier.
From multiple attempts it can be concluded that the width of the barrier causes an unacceptable delay when smaller than 10 m. The delay is still serious, and should be considered during design when between 10 m and 30 m. When the width is more than 30 m, the delay is negligible and has no effect on the flooding probability. In general it is advised to design the barrier with a minimum of 30 meters.

Navigation possibility

Goal

In order to design the optimal dimension for the lock and the barrier, a good estimation of the total closing time is important. Therefore the Matlab script described in appendix F.B is used to show the duration navigation is not possible due to closing of the barrier or a water speed that is too high.

Boundary Conditions

The parameters used in the script are explained in appendix F.B. The variables that are still open are the width of the barrier and the flow speed of the water at which navigation is still possible. In this case the maximum water flow speed is 1.5 m/s or 2.0 m/s. For a barrier width of 5,10,20,30,40,45,50 and 60 these flow speeds are tested and the time for which no navigation is not possible are presented.
During extreme rainfall the importance of navigation is neglected with respect to the safety of the water level in the city. Therefore concerning the navigation the average rain intensity during the wet season of 10 mm/day is taken into account but no other extreme rains.

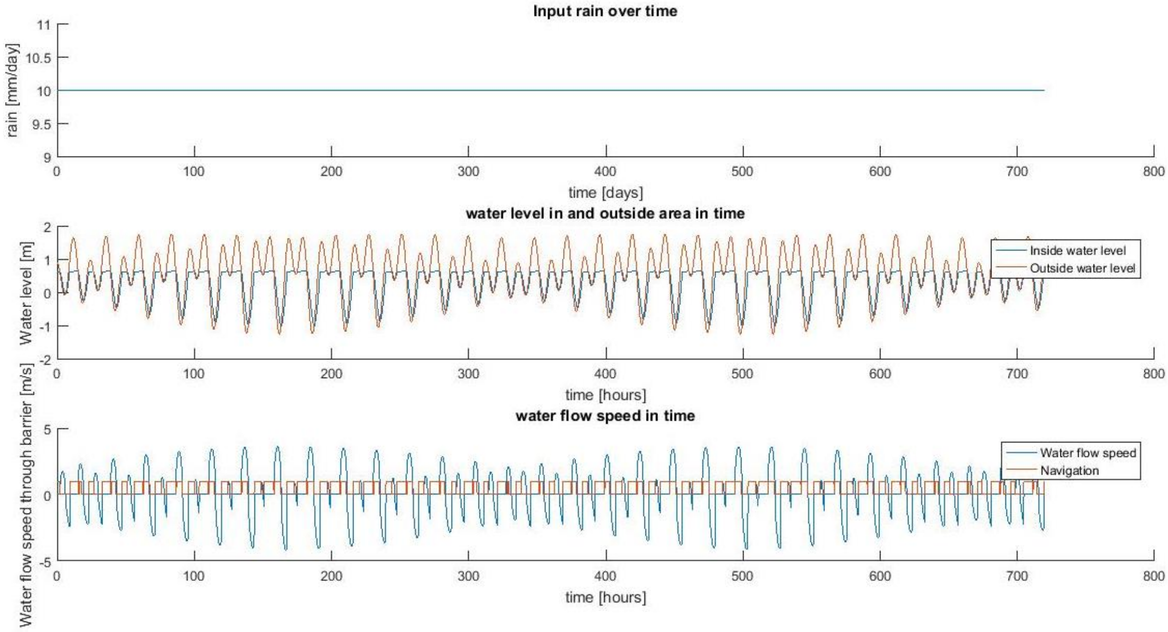
Results

Dry season

During the dry season the closing times are not normative for the design. Therefore in this phase these values are not calculated. When a more detailed design is made, the time navigation is not possible will be calculated for the dry season.

Wet season

The result of the calculation with a barrier width of 30 meters is presented in Appendix F.A - figure 2. For the other widths the result values are presented in Figure 2.



Appendix F.A - figure 2: Rain intensity, water levels, water flow speed and navigation possibility

Barrier Width	Max navigation flow speed	Maximum reached water flow speed	Non-navigation time normative day	Non-navigation time normative week	Total opening percentage
[m]	[m/s]	[m/s]	[hours]	[hours]	[%]
5	1.5	5.08	16.3	109.9	34.4
	2.0		15.9	102.1	39.0
10	1.5	4.26	16.3	109.5	34.2
	2.0		15.6	98.4	39.9
20	1.5	3.15	16.1	103.0	38.4
	2.0		16.0	98.9	42.2
30	1.5	2.44	15.9	101.3	40.7
	2.0		14.9	92.4	46.3
40	1.5	1.96	15.4	95.5	44.3
	2.0		12.4	85.4	50.4
45	1.5	1.77	14.9	92.3	46.2
	2.0		12.4	85.9	50.2
50	1.5	1.62	14.3	89.0	48.0
	2.0		12.6	86.0	50.0
60	1.5	1.38	12.7	86.4	49.9
	2.0		12.7	86.4	49.9

Table 1: Water flow speed and navigation possibility in different situations

The table shows that a width between 40 and 45 meters would be ideal when working with a maximum water flow speed of 2.0 m/s. In this situation the navigation is possible the highest percentage of the time. When the maximum navigation speed is 1.5 m/s, the even bigger barriers of 50-60 meters are preferable. These values are taken into account during the more detailed design phase. When the designs are definitive the resulting values for the dry and wet season are calculated.

Appendix F.B – Origin of the parameters and formulas

Origin of the parameters and formulas for the Matlab script

Barrier conveyance area

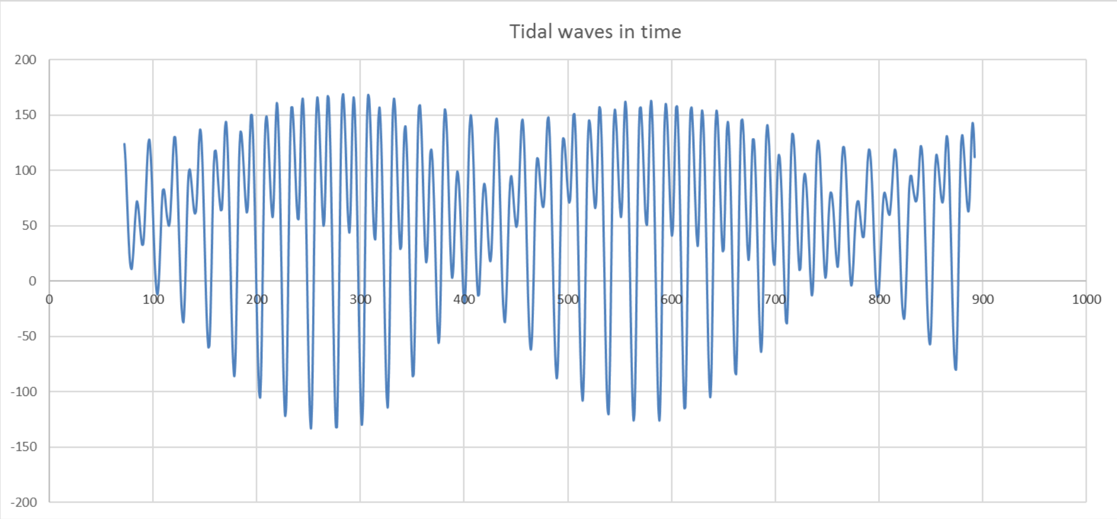
The barrier conveyance area is given by the space the water has to flow through the tidal barrier. This space is given by the width of the barrier and the water level at a specific time. Therefore the conveyance area depends on the specific design of the barrier. In all the cases considered in this project the width of the conveyance area does not differ with the height. Therefore the formula for the conveyance area of all barriers is as follows:

$$A_{conveyance} = B_{width} * (h_{in}(t) - h_{bottom})$$

The B_{width} and h_{bottom} are parameters specific for a design.

Tidal waves outside the barrier

The input data for the tidal waves outside the barrier is based on data from Thuy Loi University. Appendix F.B - figure 1 shows the water levels throughout the year and over a month. This data can be described by a formula that gives the water levels for a specific month. Two sets of parameters are obtained, one for the dry and one for the wet season.



Appendix F.B - figure 1: Water levels throughout one year and in one month

The resulting formula consists of three parts:

$$P_1 = mean + A * \cos\left(t * \frac{\pi}{6}\right) * \left(1 + \left(B * \sqrt{\sin\left(t * \frac{\pi}{336}\right)^2}\right)\right)$$

Part one gives the normal tidal waves its amplitude and mean. The amplitude differs over half a month.

$$P_2 = C * \sin\left(t * \frac{\pi}{12}\right) * \sqrt{\sin\left(t * \frac{\pi}{336}\right)^2}$$

Part two corrects the minimum water level, because as observed the minimal water level reached after a maximum differs a lot in a period of two minima. This phenomenon also differs over the span of a month.

$$P_3 = D * \cos\left(\left(t * \frac{\pi}{12}\right) + \pi\right) * \sqrt{\cos\left(t * \frac{\pi}{336}\right)^2}$$

The same phenomenon described for the minima reached, is in a relatively minor way true for the maxima. Therefore the third part adjusts the maximum water levels reached.
The resulting formula gives the water height over time:

$$h_{out} = P_1 + P_2 + P_3$$

The parameters are found by an iterative process and are shown in Appendix F.B - table 1.

Parameter	Dry season	Rain season
Mean	0.160	0.575
A	0.675	0.675
B	0.4	0.4
C	0.9	0.9
D	0.36	0.36

Appendix F.B - table 1: Parameters found

Normative rain

The normative rain is found by looking at the weather data provided by various sources. (http://en.climate-data.org/location/4235/ and https://weather-and-climate.com/average-monthly-Rainfall-Temperature-Sunshine,Ho-Chi-Minh-city,Vietnam). Based on this data 320 mm of rain will fall during the wettest month (rain season) and only 10 mm will fall during a month in the dry season. Because in the dry season the worst case scenario is no rain at all, no rain will be used when modeling the dry season.

During the rainy season the average rain on one day is about 10 mm. The maximum rain in one day given by IWER (see chapter 2) is 180 mm. Therefore the normative situation is given by one week where the rainfall on the first day is 180 mm, and the following 6 days 50 mm. The rest of the month the rain will be 10 mm a day. In this way the rain is high above the average of 320 mm for a month, but this is a good worst case scenario. The script to model this rain has a starting day and gives the rain for each moment, defined as ‘t’ in the month.

The 180mm rainfall situation however is taking into account an occurrence of once in a 100 years. For certain design criteria it can be opted to increase the allowance of failure and flooding of the system to an occurrence of once in 20 years. This can statistically be done with the following calculations:

$$180mm = \frac{1}{100}$$

$$P = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(X-\mu)^2}{2\sigma^2}}$$

$$\mu = 10, \qquad \sigma = 50, \qquad X = 130$$

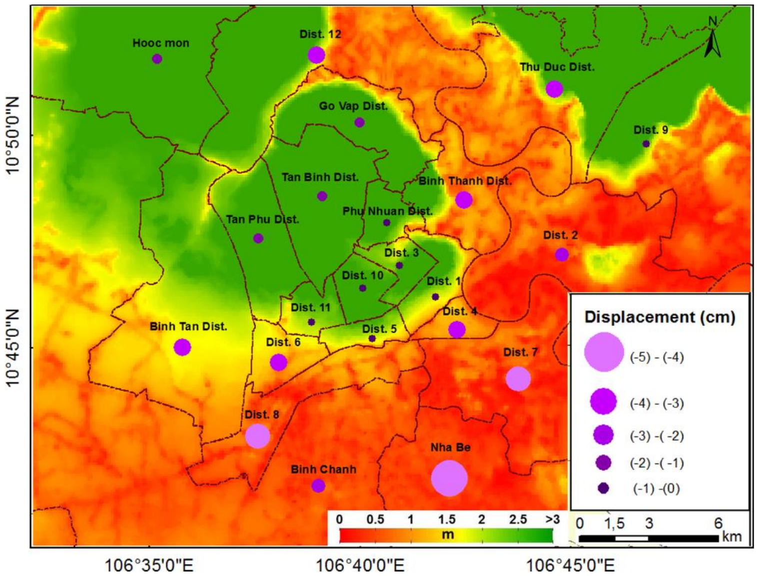
$$P = \frac{1}{125} e^{-\frac{14400}{5000}}$$

$$P = 6\%$$

So a rainfall of 130mm or more would occur approximately once in 20 years.

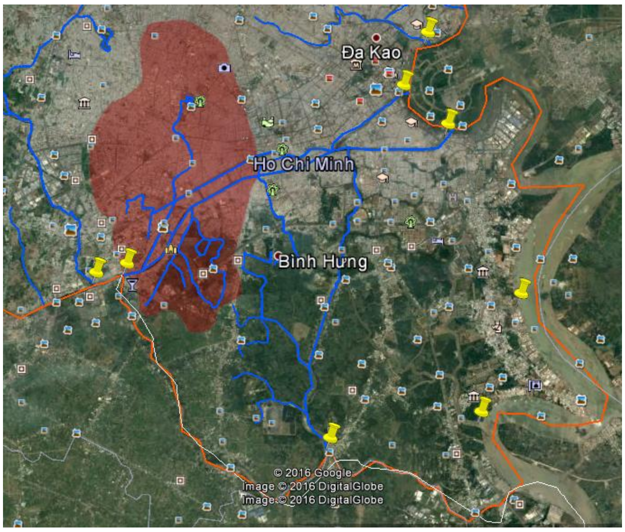
Area

In order to determine the required discharge capacities of the hydraulic structure the conveyance area needs to be chosen, any rain that falls within this area flows through rivers and canals that eventually flow out through the river at Phu Dinh. This area is estimated by assuming that the rain water always flows to the nearest waterway when the surface is flat and no slopes are present in the land surface. For topographic height differences has been used and been taken into account to make better estimations of how the rainwater is drained:



Appendix F.B - figure 2: Topographic height map

Throughout these estimations the following surface has been made which comes down to $50km^2$



Appendix F.B - figure 3: The bodies of water around Phu Dinh

Appendix F.B - figure 4: Low lying areas

Storage area

The water storage is determined by total water surface within the conveyance area. For this the bigger waterways have been taken into account along with the low-lying water fields southern in the region. The low-lying water fields are assumed to fluctuate freely with the water levels in the waterways in case of flooding conditions and the hydraulic storage area then is equal to$3.1 km^2$. For the non-flooding conditions however the area of the fields does not have to be taken into account. The hydraulic storage area then reduces to about $1.5 km^2$ and this only consists of the waterways.

Matlab script

Main script

```
%% Flow trough a closing tidal barrier
% ------------------------------------------------------------- %
%% Start program
close all
clear

%% Model parameters

% Time
modPar.rain_dry = 0;            % [1 or 0] if 1 -> dry, if 0 -> rain season
t_end = 720;                    % time [hours] until end of calculation
number_time_steps = 10000;      % number of steps shown in graphs
acc_change = 0.01;              % the acceptable change in one timestep
acc_timestep = 0.002;           % the minimum timestep if acceptable change gives a to low value

% Dimensions
modPar.B_c = 60 ;                                   % [m] conveniance width
modPar.h_bottom = -3.5;                             % [m] bottom depth of the barrier
modPar.A_storage = 1500000 ;                        % [m^2} storage area
modPar.A_rain = 50000000;                           % [m^2] rain area
modPar.time = 0.0;                                  % First value for time array
modPar.delta_time = t_end/(number_time_steps-1);    % The delta_time shown in the resulting graph.

% Navigation speed
modPar.navspeed = 1.5;      % [m/s] Speed until which navigation is possible

% Rain conditions
modPar.rain_peak = 10;      % [mm/day] peak rainfall (180)
modPar.rain_high = 10;      % [mm/day] high rainfall (50)
modPar.rain_mean = 10;      % [mm/day] average rainfall (10)
modPar.time_start_rain = 1; % [days] until peak rain starts
modPar.time_high = 6;       % [days] the high rainfall continues

% Tidal conditions
if modPar.rain_dry == 1;
    % Parameters dry season
    modPar.mean = 0.375;
    modPar.AA = 0.675;
    modPar.BB = 0.4;
    modPar.CC = 0.9;
    modPar.DD = 0.36;
    % Functional Requirements dry season
    modPar.close_flood = 1.0;    % [m] Closing water level in case of flood
    modPar.close_env = -0.6;     % [m] Closing water level in case of draught
else
    % parameters rain season
    modPar.mean = 0.575;
    modPar.AA = 0.675;
    modPar.BB = 0.4;
    modPar.CC = 0.9;
    modPar.DD = 0.36;
    % Functional Requirements rain season
    modPar.close_flood = 0.6;    % [m] Closing water level in case of flood
    modPar.close_env = -1.6;     % [m] Closing water level in case of draught
end

%% Domain discritisation

t_span=linspace(0,t_end,number_time_steps);        % Creates a vector with all the timesteps that have to be shown in the graphs.

% Initial value vectors
t = 0;
h_out_initial = fun_h_out(t,modPar);               % First outer water level consistent with the given formula.
h_in_initial = 0.6;                                % First inner water level is given by 0.6 meters.
```

```
%% Runga Kutta method
dt = 0;
h_in_result = [h_in_initial,0];                  %Initial condition for the inner water level
t=t_span(1);                                     %Start with first input for time t(0)
h_in=[h_in_initial,0];                           %Start with initial foxes and rabbits
idx.t_span=1;                                    %Counter for timesteps taken (start at 1)
k11 = diffeq(t,h_in,modPar);
tic
while t <= t_end                                 %While loop for t until end time
    % Function:
    k11 = diffeq(t,h_in,modPar);                 %Find k values
    k1 = k11(:,1);
    k22 = diffeq(t,(h_in+(0.5*dt*k1)),modPar);
    k2 = k22(:,1);
    k33 = diffeq(t,(h_in+(0.5*dt*k2)),modPar);
    k3 = k33(:,1);
    k44 = diffeq(t,(h_in+(dt*k2)),modPar);
    k4 = k44(:,1);
    y_prime= (1/6)*(k1+2*(k2)+2*(k3)+k4);        %Use lotkafunc for calcuating y_prime
    h_in(:,2) = k11(:,2);
    % Step-size
    dt_span=(t_span(idx.t_span)-t);              %Stepsize dt for dt_out
    val = acc_change*h_in;
    dt_sim=val/(y_prime);                        %Stepsize dt for dt_sim, depending on max acceptable change
    dt1=min(dt_span,(min(abs(dt_sim))));         %Dt used is minimum of dt_out and dt_sim
    dt=max(acc_timestep,dt1);
    t=t+dt;
    h_in(:,1)=h_in(:,1)+(y_prime*dt);            %Euler (Spatial derivative function)
        if (t-t_span(idx.t_span))>=0             %Define zero, if t-t_out<eps,
           if idx.t_span <= (t_end/modPar.delta_time)
               h_in_result(idx.t_span+1,:)= h_in;   %A becomes output
               idx.t_span=idx.t_span+1;         %Counter idx increases with 1
           else
                t = t_end+0.1;
           end
        end
     % Counter
     Time_done_Runga_Kutta = t                   % gives the time passed in the calculation
end
toc
%% Create the total output vector for the outside water level
h_out = fun_h_out(t_span,modPar);

%% Create the total output vector for the rain over the time
jj =size(t_span);
k = 0;
ii = 1;
nav_week = 0;
%% Opening times
while ii <= jj(2)
    rain_total(ii) = Rain_in_time(k,modPar)*24;
    if h_in_result(ii,2) == 0
        no_nav(ii) = 1;
    else if h_in_result(ii,2) >= modPar.navspeed
            no_nav(ii) = 1;
        else
            no_nav(ii) = 0;
        end
    end
        ii = ii + 1;
    k = k + modPar.delta_time;
end
v_water = h_in_result(:,2);
```

```
% Normative day
xx=0;
yy=1;
while (xx+24) <= t_end
    ls = round(((xx+1)/modPar.delta_time));
    rs = round(((xx+24)/modPar.delta_time));
    nav_day(yy) = sum(no_nav(ls:rs))*modPar.delta_time;
    xx = xx + 24;
    yy = yy + 1;
end
% Normative week
xx=0;
yy=1;
while (xx+164) <= t_end
    ls = round(((xx+1)/modPar.delta_time));
    rs = round(((xx+164)/modPar.delta_time));
    nav_week(yy) = sum(no_nav(ls:rs))*modPar.delta_time;
    xx = xx + 164;
    yy = yy + 1;
end
%% Output values
max_nav_day = max(nav_day)                                  % Hours barrier closes on worst day
max_nav_week = max(nav_week)                                % Hours barrier closes in worst week
per_no_nav = (sum(no_nav)*modPar.delta_time)/t_end;         % Percentage barrier is closed in total
per_nav = 1 - per_no_nav                                    % Percentage barrier is open in total
max_v_water = max(v_water)                                  % Maximum water speed through the barrier

%% Plot the results
figure()
% Plot of the rain
subplot(3,1,1)
hold on
plot(t_span,rain_total(:,:));
title('Input rain over time')
xlabel('time [days]')
ylabel('rain [mm/day]')
```

```
% Plot of the water levels
subplot(3,1,2)
hold on
plot(t_span,h_in_result(:,1));
hold on
plot(t_span,h_out);
title('water level in and outside area in time')
xlabel('time [hours]')
ylabel('Water level [m]')
legend('Inside water level','Outside water level')

% Plot of the water speed and navigation possibility
subplot(3,1,3)
hold on
plot(t_span,v_water);
hold on
plot(t_span,no_nav);
title('water flow speed in time')
xlabel('time [hours]')
ylabel('Water flow speed through barrier [m/s]')
legend('Water flow speed','Navigation')
```

Differential equation function

```
function [dh_dt] = diffeq(t,h_inres,modPar)
%% Outflow
Q = Flux_Q(t,h_inres,modPar);
Q_w = Q(:,1);
v_water = Q(:,2);
%% Rain
rain_t = Rain_in_time(t,modPar);
Q_rain = rain_t/1000 * modPar.A_rain;

%% Flow combination in and out:
dh_dt = [(Q_rain - Q_w)/modPar.A_storage,v_water];
end
```

Flux function

```
function [Q_w] = Flux_Q(t,h_inres,modPar)
h_out = fun_h_out(t,modPar);
delta_h = h_inres(1,1) - h_out;
if delta_h >= 0
    Q_w1 = (modPar.B_c.*(h_inres(:,1)-modPar.h_bottom)).*sqrt(2*9.81*(sqrt((delta_h).^2)))*3600;
    v_water = sqrt(2*9.81*(sqrt((delta_h).^2)));
else
    Q_w1 = -(modPar.B_c.*(h_inres(:,1)-modPar.h_bottom)).*sqrt(2*9.81*(sqrt((delta_h).^2)))*3600;
    v_water = -sqrt(2*9.81*(sqrt((delta_h).^2)));
end

%% flood situation
if h_out >= modPar.close_flood && h_inres(:,1) >= modPar.close_flood
    if delta_h < 0
        Q_w2 = 0;
        v_water = 0;
    else
        Q_w2 = Q_w1;
    end
    %% draught situation
else if h_inres(:,1) <= modPar.close_env
        if delta_h < 0
            Q_w2 = Q_w1;
        else
            Q_w2 = 0;
            v_water = 0;
        end
        %% situation between
else
Q_w2 = Q_w1;
    end
end
Q_w = [Q_w2,v_water];
end
```

Tidal waves function

```
function [h_out] = fun_h_out(t,modPar)
%% This function gives the water level at a specific point in a year
part1 = modPar.mean+(modPar.AA.*cos(t.*pi./(6)).*(1+(modPar.BB.*sqrt(sin(t.*pi./(336)).^2))));
part2 = modPar.CC.*sin(t.*pi./(12)).*sqrt((sin(t.*pi./(336)).^2));
part3 = modPar.DD.*cos((t.*pi./(12))+pi).*sqrt(cos(t.*pi./(336)).^2);
h_out = part1+part2+part3;
end
```

Rain function

```
function [rain_t] = Rain_in_time(t,modPar)
%% This function gives the rain intensity at one point t
t_day = t/24;

%% Find rain intensity for rain season
if t_day >= modPar.time_start_rain && t_day < (modPar.time_start_rain + 1)
    rain_t = modPar.rain_peak/24;
else if  t_day >= (modPar.time_start_rain + 1) && t_day < (modPar.time_start_rain + 1 + modPar.time_high)
        rain_t = modPar.rain_high/24;
    else
        rain_t = modPar.rain_mean/24;
    end
end

%% Make rain zero for dry season
if modPar.rain_dry == 1
    rain_t = 0;
end

end
```

Appendix G – Dimensions of vessels in Vietnam

Table provided by the IWER, it shows the dimensions of the several ship categories. This data is interpolated to calculate the dimensions for the vessels passing Phu Dinh and to obtain the dimensions for actual measured ships.

DWT (T)	Vessels						Barges		
	2000	1000	300	200	100	40	500	400	100
Length (m)	90	75	38	34	15	8	40	41	27
Width (m)	12	10.5	7.0	6.6	5	3	10	11,2	6,4
Draught (m)	3.5	2.8	2.2	1.7	1.0	0.8	1.7	1.3	1.0

Appendix G - figure 1: Dimensions for design vessels of different DWT

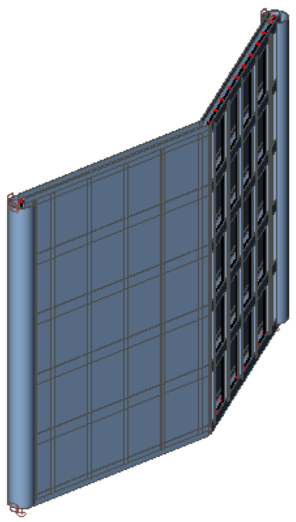
Appendix H – Door designs made in Scia

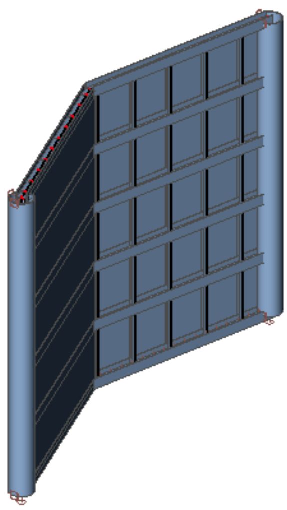
SCIA designs

For the SCIA models simplified structures have been developed only dealing with the resulting water pressures. Self-weight has not been taken into account since this will result in more complex structures that are not determined in this design phase and belong more to the detailed design of the chosen variant.

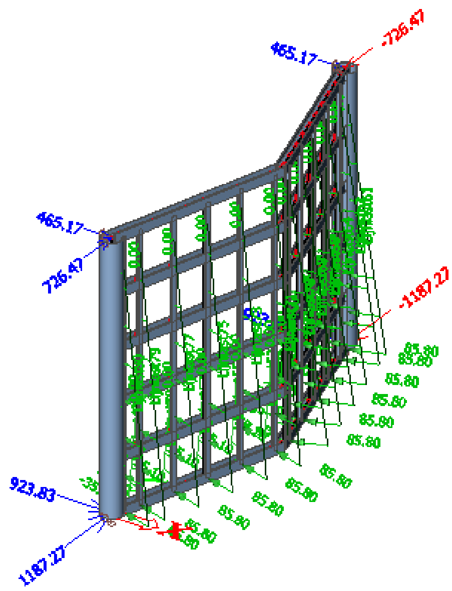
In the following illustrations the green arrows illustrate the modeled water pressures on the structure including the safety factors, the blue and the red arrows show the reaction forces on the supports in positive or negative direction according to the local axis.

Mitre gates for 10m span navigation lock for outer high water





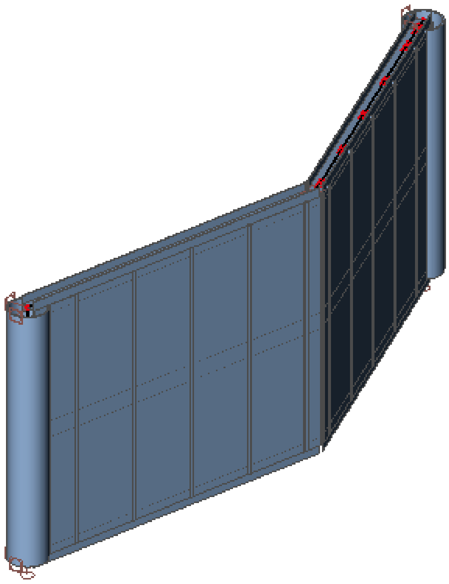
Load transfer:

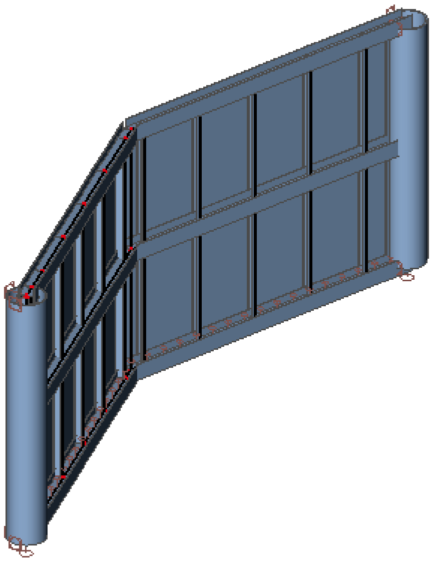


Governing water difference	3.62m
Amount of steel used	131KN
Amount of profiles	2

The gates have been designed in such a way that the doors stand under an angle of 1:3 with the walls. This angle is the most optimal in the load transfers through the structure and is maintained in the design of all the mitre gate designs to follow

Mitre gates for 10m span navigation lock for inner high water





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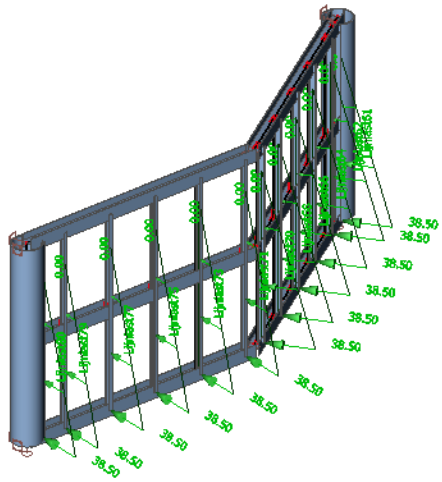
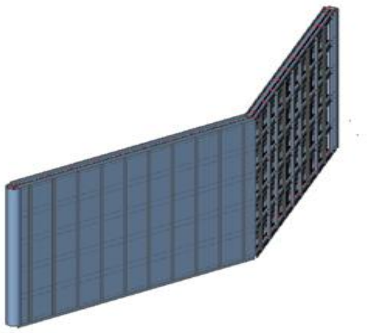


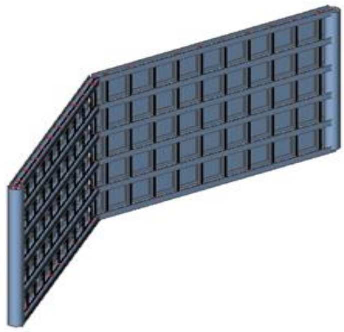
Water difference	3.5m
Amount of steel used	57KN
Amount of profiles	2

Mitre gates for 21m span navigation lock for outer high water

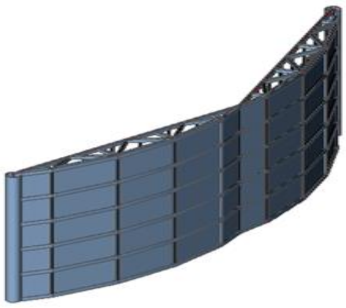
Beam variant

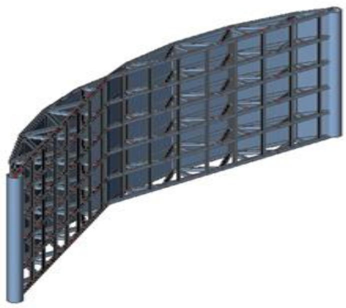
For the beam variant three alternatives have been developed. Alternative 1 is where the entire structure is made out of two profiles, alternative 2 where the structure is made out of three profiles and alternative 3 where the structure is made out of two profiles but where the lock is set to regulate in such a way that the maximum water level difference on one gate is 2.5m.



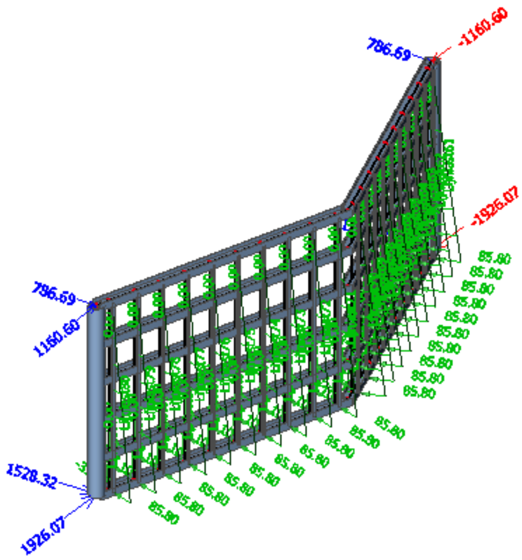


Truss variant





Load transfer:



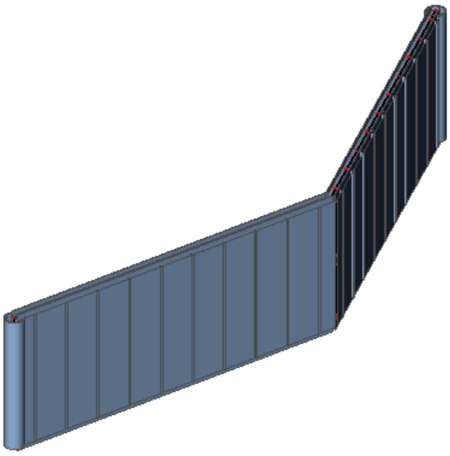
Alternative 1	
Water difference	3.62m
Amount of steel used	421KN
Amount of profiles	2

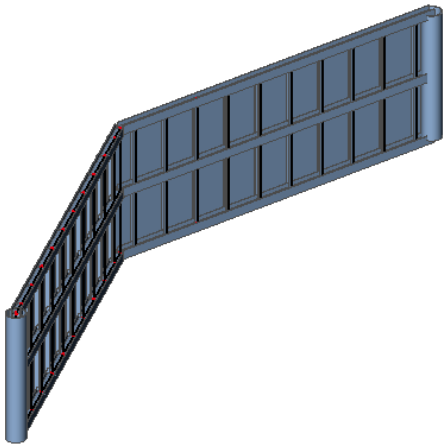
Alternative 2	
Water difference	3.62m
Amount of steel used	404KN
Amount of profiles	3

Alternative 3	
Water difference	2.5m
Amount of steel used	362KN
Amount of profiles	2

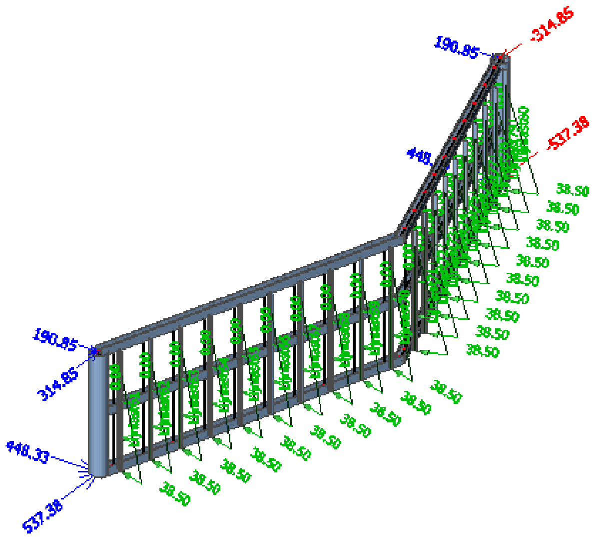
Truss variant	
Water difference	2.5m
Amount of steel used	291KN
Amount of profiles	4

Mitre gates for 21m span navigation lock for inner high water



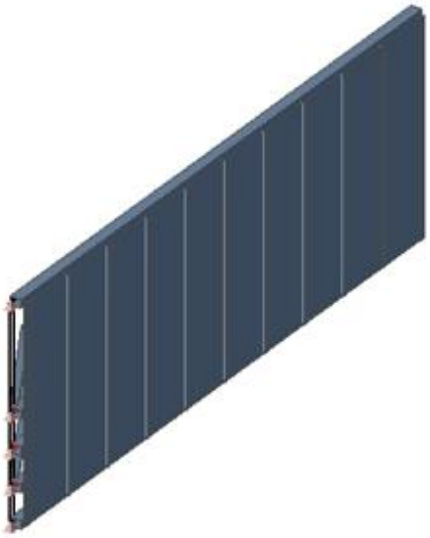


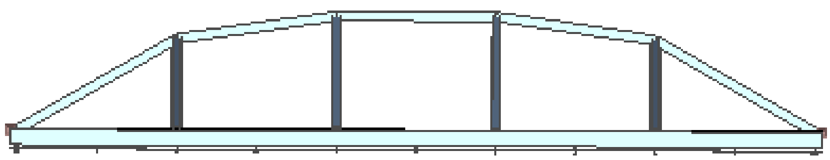
Load transfer:

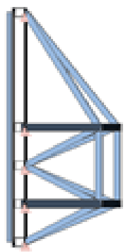


Water difference	3.5m
Amount of steel used	111KN
Amount of profiles	2

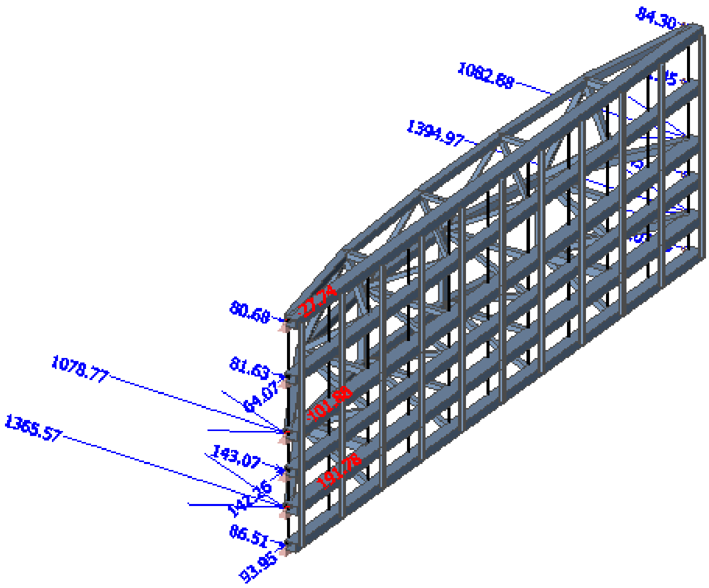
Vertical lift gate for 21m span navigation lock







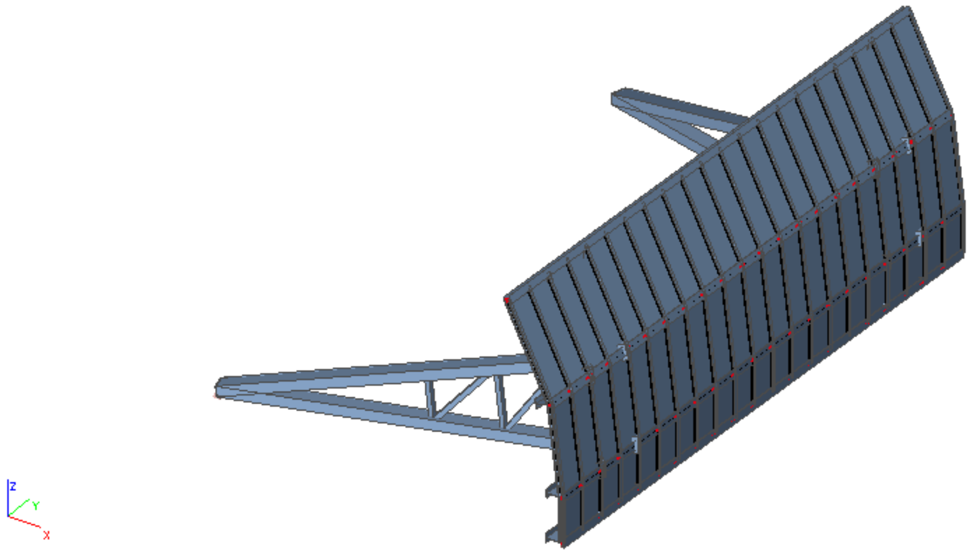
Load transfer:

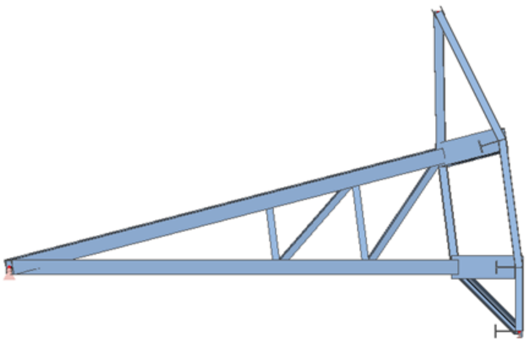


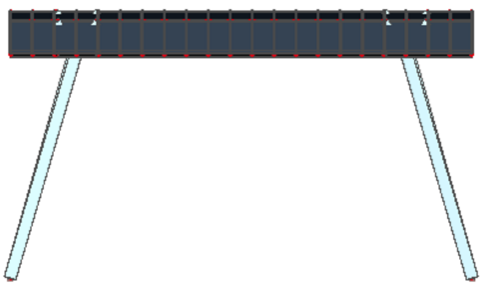
Water difference	3.62m
Amount of steel used	294KN
Amount of profiles	8

Initiallty a gate was develloped using no framework. The outcome however has led to vary large amounts of steel needed due to the large beams that would be necessary in such a design. Adding width to the cross-section of the gate makes it a lot easier for the gate to deal with the large bending moments imposed on the structure leading to a significant reduction in steel needed.

Radial gate for 21m span navigation lock

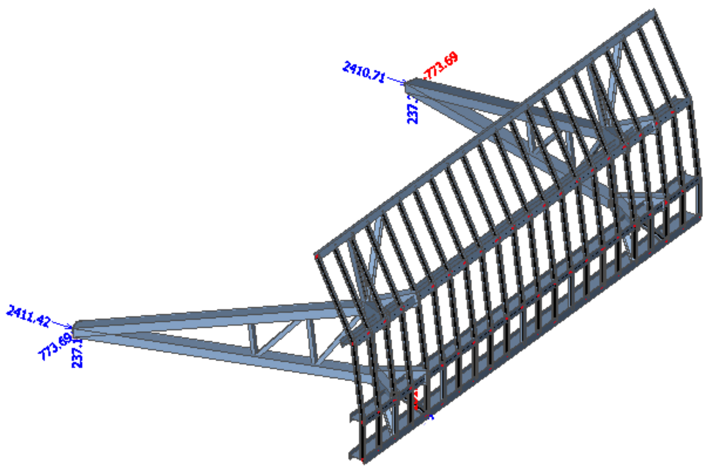






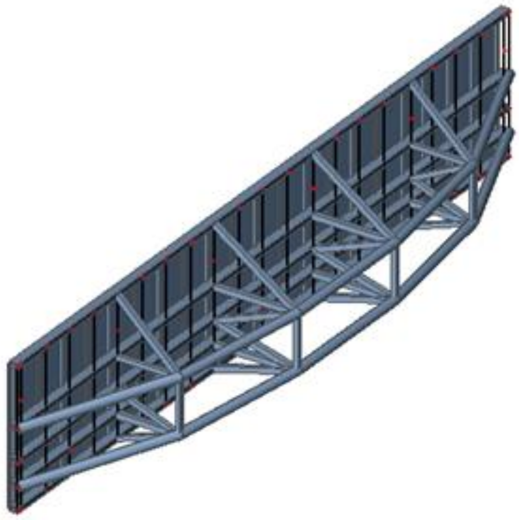
Load transfer:

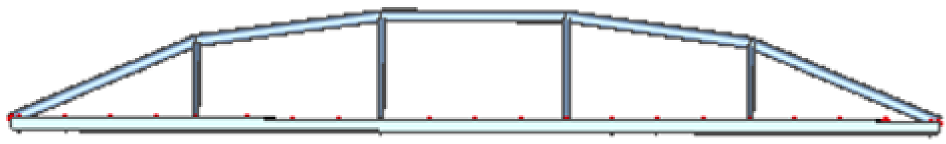
Water difference	3.62m
Amount of steel used	301KN
Amount of profiles	7

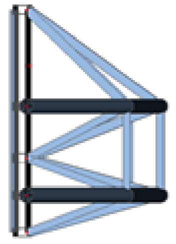


The radial gate is lifter up through rotation around the supports of the large longitudinal elements connection the hinges to the round shaped sheet. The supporting beams are placed under an angle in order to support the sheet along its width reducing the bending moments in the gate and thus the dimensions of the large beams in the width direction.

Vertical lift gate for tidal barrier







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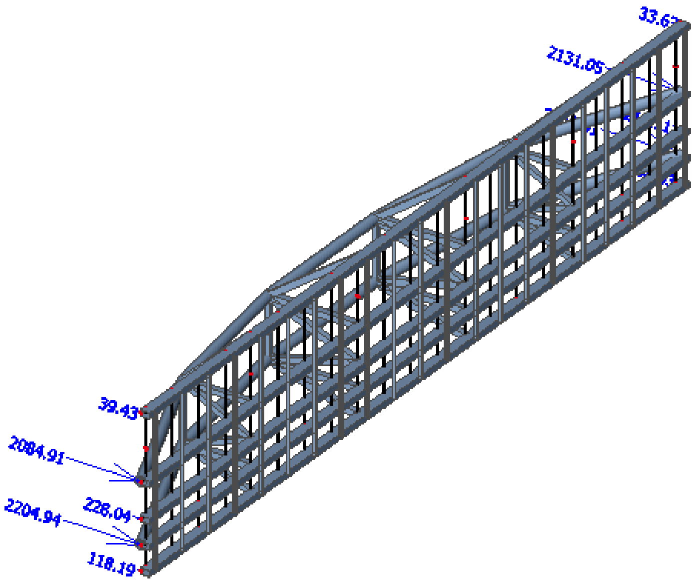
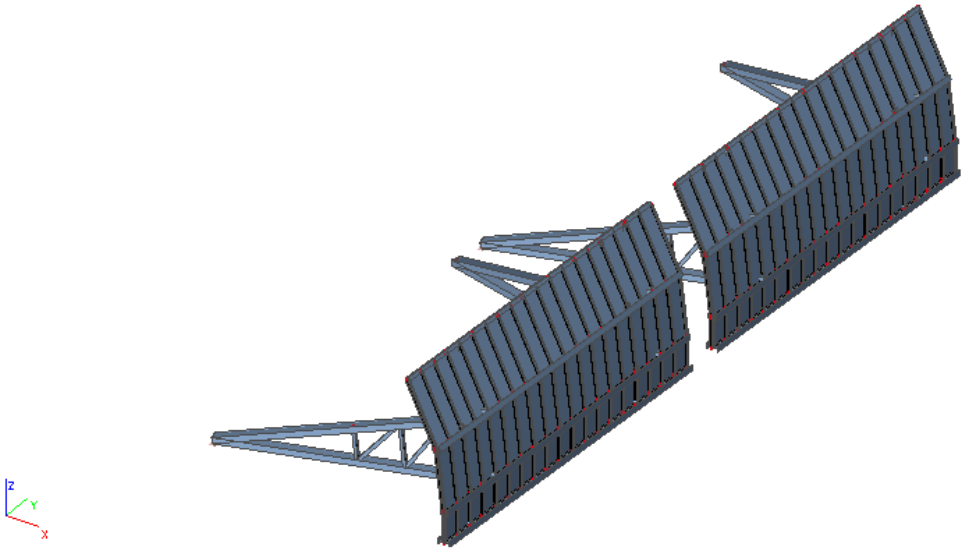
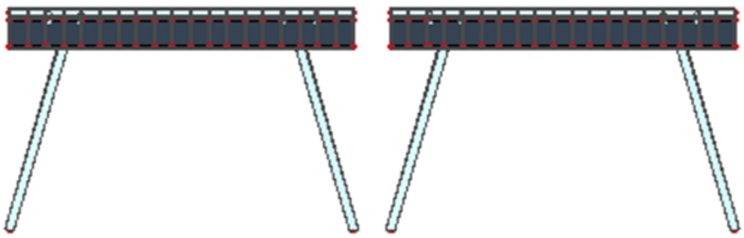


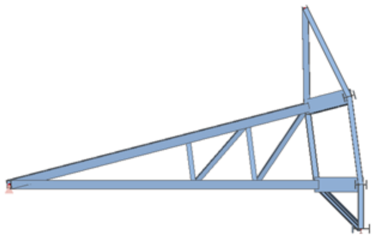
Water difference	3.62m
Amount of steel used	525KN
Amount of profiles	7

For this lift gate the same principle has been used as for the 21m lift gate designed for the navigation lock except that the width/thickness of the structure has been increased due to the larger bending moments caused by the wider span.

Radial gate for tidal barrier







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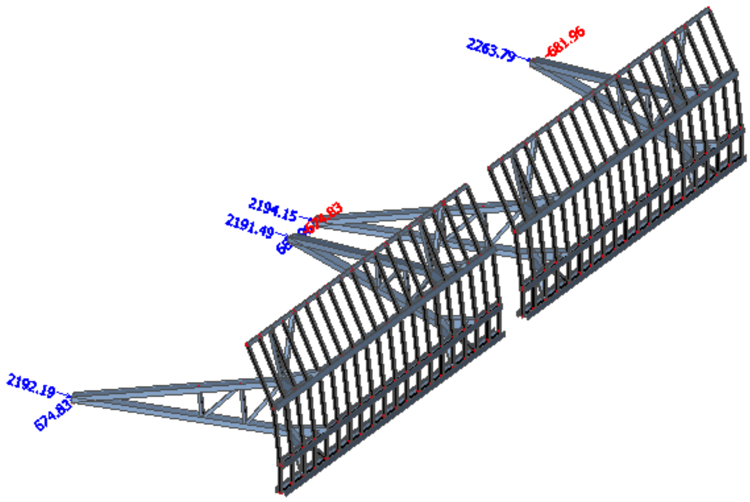


Water difference	3.62m
Amount of steel used	526KN
Amount of profiles	7

To apply a radial gate structure for the 40m span it is found that the most optimal is to apply two smaller gates. The reason for this is that the large beams in width direction would have to become frameworks in order to cover the entire span. This would however lead to a very complex and large design.

Appendix I – Dimensioning the walls for the locks

Dimensioning the lock wall, for the big lock of variant A

Designing the lock wall is an iterative process. For the design steps, the process as described in chapter 35 of the manual hydraulic structures (Molenaar & Voorendt, 2016), which from now on will be referred to as the manual, is followed. Four parameters are needed for this calculation. First the concrete class has to be selected. A normal class, that can easily be accessed around the world and that can also be produced in Vietnam, is C35/45. This is the first parameter.
The design strength of the concrete f_{cd} is obtained by dividing the strength over the safety factor (f_{ck}/γ_c).
In this case f_{cd} is calculated to be: $\frac{35\frac{N}{mm^2}}{1.2} = 29.2\frac{N}{mm^2}$

The second parameter that is already known is the height of the sluice. This height is equal to 7.5 m. This takes the maximum height of the water into account, the depth of the lock and the height of the lock above the water.

The third parameter that has to be taken into account for an economical design is the percentage of reinforcement (ρ) that is needed. In the case of concrete class C35/45 the value for has a maximum (ρ_{max}) of 2.49 and a minimum value (ρ_{min}) of 0.21. The selected value for ρ has to be between these values.

Governing forces and bending moment

The fourth parameter is the force and bending moment that the concrete has to transfer. Both walls of the lock will be evaluated to determine the governing bending moment. First, the lock wall adjacent to the quay will be elaborated. Secondly, the wall adjacent to the barrier will be elaborated.

Lock wall adjacent to quay:

- This wall is subjected to a maximum water level of +3.62 m and a soil level of +1.4 m. Besides, the walls of the lock exert a force on the wall. The force exerted by the soil per meter can be calculated as follows:

$$Q = \frac{1}{2}\gamma * h^2 * K_p^* - 2 * c * h\sqrt{K_p}$$

In which:
c [kN/m²] = cohesion = 7.6 kN/m²
γ [kN/m³] = volumetric soil weight = 15 kN/m³
h [m] = length of soil layer = 4.9 m

$$K_p = \frac{1+\sin(\varphi)}{1-\sin(\varphi)}\ (unsaturated) \qquad K_p^* = K_p\left(1-\frac{\gamma_w}{\gamma}\right)+\frac{\gamma_w}{\gamma}\ (saturated)$$

In which:
ϕ (°) = angle of repose = 5
γ_w [kN/m³] = volumetric water weight = 10 kN/m³

- For the length of the soil layer, a ground level of +1.4 m is chosen because this is the lowest ground level at the embankments. This gives an h of 3.5 + 1.4 = 4.9 m. Now the force exerted by the ground can be calculated:

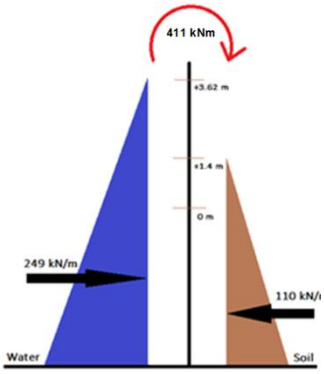
$$K_p = 1.19 \qquad K_p^* = 1.06 \qquad Q = 110\ kN/m$$

- The force exerted by the water per meter can be calculated as follows:

$$q_h = \int_{-3.5}^{3.62} \rho g h \, dh = \frac{1}{2}\rho g h^2 = 0.5 * 1000 * 9.81 * 7.12^2 = 249 \; kN/m$$

- The resulting force becomes 249 – 110 = 139 kN/m in the situation of maximum high water inside the lock chamber. This results in a clockwise moment:

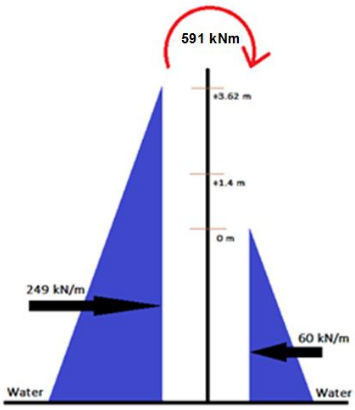
$$M = \left(\frac{1}{3} * 110 * 4.9\right) - \left(\frac{1}{3} * 249 * 7.12\right) = 411 \; kNm$$



Appendix I - figure 1: Overview of the forces on the wall adjacent to the quay wall

An overview of the situation and the forces and moment can be found in Appendix I - figure 1.

The bending moment due to the resultant force of the soil and the water is 411 kNm. The lift gate will transfer all the force of the water in the longitudinal direction. This plays no role for the bending moment in sideways direction.

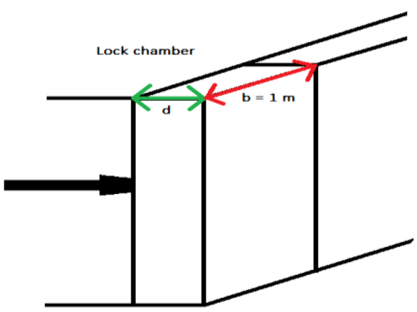


Appendix I - figure 2: Overview of the forces on the wall with water on both sides

Lock wall adjacent to barrier

The second wall is the wall adjacent to the barrier. This wall is not subjected to soil forces but only to hydrostatic forces. The bending moment due to the resultant force of the water inside and outside the lock chamber is 592 kNm. This is larger than the wall adjacent to the quay and therefore this moment is governing.

Using the given parameters, a first thickness of the door is assumed. This thickness is used to calculate the following parameter: $\frac{M_u(breaking\ moment)}{b\ (width) \cdot \mathrm{d\ (height)}^2 \cdot f_{cd}(design\ strength)}$. The value of this parameter can be looked up in table 35-8 of the manual hydraulic structures. This leads to a value for ρ, which has to be between the given values (between 0.21 and 2.49).



Appendix I - figure 3: Schematization of wall

For the calculation of the parameter, the lock wall is schematized as a beam with a width of 1 meter, because the dimensions and reinforcement have to be obtained per meter, and a varying height (which is in fact the thickness of the wall). The varying heights can be found in Figure 3 and lead to different reinforcement ratios. Thicknesses of 0.3 to 0.7 m satisfy the reinforcement ratio. A thickness of 0.4 m is the most economic option. For now this thickness is chosen for the walls. It is possible that it changes after other checks.

d (m)	M_u/bd^2f_{cd}	ρ	Concr (m^3)	Steel (m^3)	Concr (dong)	Steel (dong)	Total cost
0.3	224.88584	1.61	2.25	0.036225	2863635.75	4088570.85	6952207
0.4	126.49829	0.84	3	0.0252	3818181	2844223.2	6662404
0.5	80.958904	0.52	3.75	0.0195	4772726.25	2200887	6973613
0.6	56.221461	0.35	4.5	0.01575	5727271.5	1777639.5	7504911
0.7	41.305563	0.25	5.25	0.013125	6681816.75	1481366.25	8163183

Appendix I - table 1: Results for different widths of the concrete wall

Shear stress check

To check if the concrete walls of the lock can handle the shear forces a shear resistance check is performed. First the shear resistance of the concrete ($V_{Rd,c}$) is calculated, without the use of shear reinforcement, as found in the manual .

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d \quad [\mathrm{N}]$$

With a minimum of:

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad [\mathrm{N}]$$

In this formula, the parameters are specified as follows:

f_{ck} [N/mm^2] = characteristic compressive cylinder strength of concrete at 28 days in MPa, in this case concrete of class C35/45 is used, so f_{ck} = 35 N/mm^2

k [-] = $1 + \sqrt{\frac{200}{d}} \leq 2.0 \; with \; d \; in \; mm$ The value of d follows from the previous assumed thickness of the concrete wall. In this case $k = 1 + \sqrt{\frac{200}{400}} = 1.71$

ρ1 [-] = reinforcement ratio for longitudinal reinforcement, the assumed ratio of before is used, in this case that is 0.59%

σ_{cp} [N/mm^2] = compressive stress in the concrete from axial load or pre-stressing, in this case there is no pre stress or axial load present. This means that this value becomes 0. This also means that k_1 is not needed.

$C_{RD,c}$ [-] = a coefficient, in the Netherlands: 0.12 From discussion with our client it followed that Dutch values are used, because they are definitely safer than Vietnamese standards.

v_{min} [] = $0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}{}^{\frac{1}{2}} = 0.035 * 1.71^{\frac{3}{2}} * 35^{\frac{1}{2}} = 0.46$

Filling in the formula leads to:

$$V_{Rd,c} = \left[0.12 \cdot 1.47 \cdot (100 \cdot 0.84 \cdot 35)^{\frac{1}{3}}\right] \cdot 1000 \cdot 400 = 1011 \; kN$$

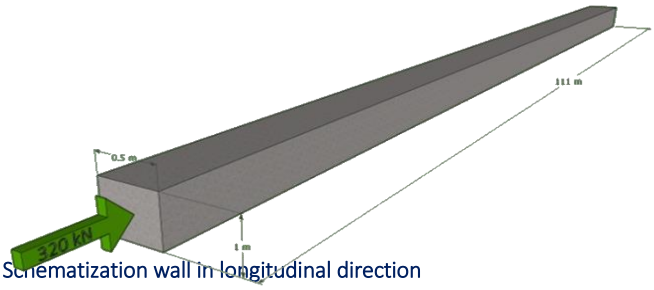
The minimum value is also calculated:

$$V_{Rd,c} = (0.46) \cdot 1000 \cdot 400 \; = 184 \; kN$$

The design value of the shear force on the structure is 189 kN/m (resulting water force). This means that the resistance is bigger than the force, so there is no shear reinforcement needed.

Normal force check

The main forces that will have to be transferred from the lift gate, will be exerted at both walls at the end of the lock. The total force, which is also checked using Scia, is 2400 kN. It is assumed that the force is evenly spread over the entire height of the wall, of 7.5 m. This leads to a force of 320 kN per meter height. The wall is schematized as a horizontal column, this is shown in Figure 4.



Appendix I - figure 4: Schematization wall in longitudinal direction

The force that can be absorbed by the concrete is 29.2 N/mm^2, which is equal to 29200 kN/m^2. The total area of the side is 0.40 m * 1 m = 0.40 m^2. This is able to retain 11680 kN in normal force. This is way more than the 320 kN needed. So it seems that only a thickness of 0.40 m is needed for the wall of the big lock. However, problems are foreseen related to buckling of the wall. This can however not be calculated because of a lack of time and enough knowledge of this subject. For this reason the wall is made much bigger then calculated. The wall thickness is made 1 m, which seems reasonable, compared to the walls in the big lock of the other alternative.

Dimensioning the lock wall, for the small lock of variant A

Designing the lock wall is an iterative process. For the design steps, the process as described in chapter 35 of the manual is followed. Four parameters are needed for this calculation. First the concrete class has to be selected. A normal class, that can easily be accessed around the world and that can also be produced in Vietnam, is C35/45. This is the first parameter. The design strength of the concrete f_{cd} is obtained by dividing the strength over the safety factor (f_{ck}/γ_c).

In this case f_{cd} is calculated to be: $\frac{35\frac{N}{mm^2}}{1.2} = 29.2\frac{N}{mm^2}$

The second parameter that is already known is the height of the sluice. This height is equal to 7.5 m. This takes the maximum height of the water into account, the depth of the lock and the height of the lock above the water.

The third parameter that has to be taken into account for an economical design is the percentage of reinforcement (ρ) that is needed. In the case of concrete class C35/45 the value for has a maximum (ρ_{max}) of 2.49 and a minimum value (ρ_{min}) of 0.21. The selected value for ρ has to be between these values.

Governing forces and bending moment

The fourth parameter is the force and bending moment that the concrete has to transfer. Both walls of the lock will be evaluated to determine the governing bending moment. First, the lock wall adjacent to the quay will be elaborated. Secondly, the wall adjacent to the barrier will be elaborated.

Lock wall adjacent to quay:

- This wall is subjected to a maximum water level of +3.62 m and a soil level of +1.4 m. Besides, the walls of the lock exert a force on the wall. The force exerted by the soil per meter can be calculated as follows:

$$Q = \frac{1}{2}\gamma * h^2 * K_p^* - 2 * c * h\sqrt{K_p}$$

In which:
c [kN/m^2] = cohesion = 7.6 kN/m^2
γ [kN/m^3] = volumetric soil weight = 15 kN/m^3
h [m] = length of soil layer = 4.9 m

$$K_p = \frac{1+\sin(\varphi)}{1-\sin(\varphi)}\ (unsaturated) \qquad K_p^* = K_p\left(1 - \frac{\gamma_w}{\gamma}\right) + \frac{\gamma_w}{\gamma}\ (saturated)$$

In which:
ϕ (°) = angle of repose = 5
γ_w [kN/m^3] = volumetric water weight = 10 kN/m^3

- For the length of the soil layer, a ground level of +1.4 m is chosen because this is the lowest ground level at the embankments. This gives an h of 3.5 + 1.4 = 4.9 m. Now the force exerted by the ground can be calculated:

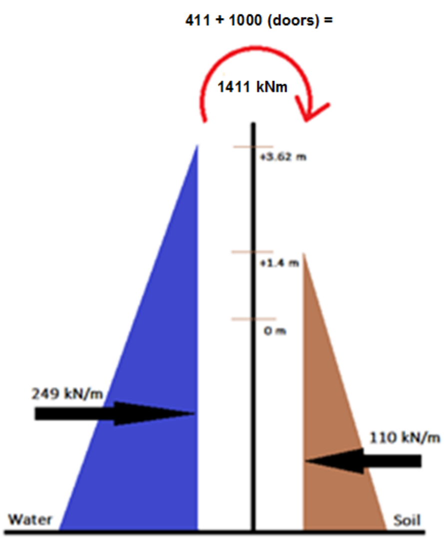
$$K_p = 1.19 \qquad K_p^* = 1.06 \qquad Q = 110\ kN/m$$

- The force exerted by the water per meter can be calculated as follows:

$$q_h = \int_{-3.5}^{3.62} \rho g h\, dh = \frac{1}{2}\rho g h^2 = 0.5 * 1000 * 9.81 * 7.12^2 = 249\ kN/m$$

- The resulting force becomes 249 – 110 = 139 kN/m in the situation of maximum high water inside the lock chamber. This results in a clockwise moment:

$$M = \left(\frac{1}{3} * 110 * 4.9\right) - \left(\frac{1}{3} * 249 * 7.12\right) = 411\ kNm$$



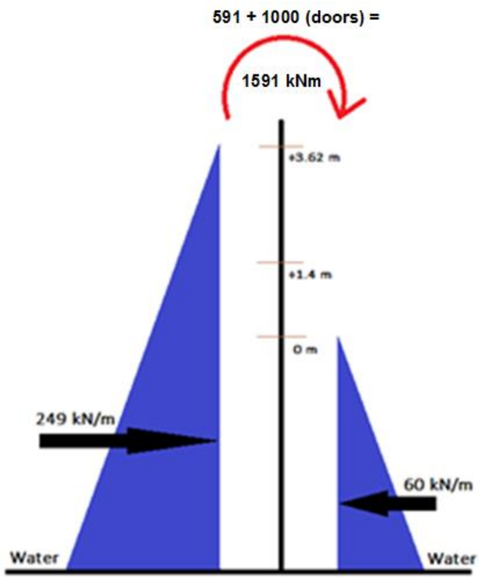
Appendix I - figure 5: Resulting forces

An overview of the situation and the forces and moment can be found in Appendix I - figure 5.

The bending moment due to the resultant force of the soil and the water is 411 kNm. There will be another bending moment in the wall, due to the gates which are transferring the hydrostatic pressure to the wall. This moment is not calculated exactly but it is assumed to be in the order of magnitude of 1000 kNm, because this is the average of different calculations with the program Scia. This moment works in the same direction as the moment due to the ground and water force in the lock chamber. Therefore, the walls of the lock have to withstand a moment of 1411 kNm.
NOTE: the moment is not calculated exactly because there is a lack of knowledge about the transferring of moments by the hinges of the doors.

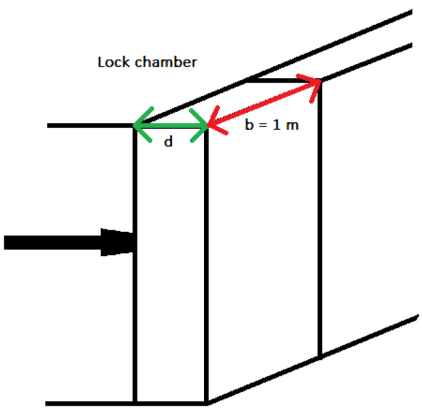
Lock wall adjacent to the dam

The second wall is the wall adjacent to the dam. This wall is not subjected to soil forces but only to hydrostatic forces and the forces from the doors. The bending moment due to the resultant force of the water inside and outside the lock chamber is 591 kNm. The same bending moment due to the doors has to be added so the total bending moment becomes 1591 kNm. This is larger than the wall adjacent to the quay and therefore this moment is governing. This is shown in Appendix I - figure 6.



Appendix I - figure 6: Resulting forces small lock

Using the given parameters, a first thickness of the door is assumed. This thickness is used to calculate the following parameter: $\frac{M_u(breaking\ moment)}{b\ (width) \cdot \mathrm{d\ (height)}^2 \cdot \mathrm{f}_{cd}(design\ strength)}$. The value of this parameter can be looked up in table 35-8 of the manual hydraulic structures. This leads to a value for ρ, which has to be between the given values (between 0.21 and 2.49).



Appendix I - figure 7: Schematization of wall

For the calculation of the parameter, the lock wall is schematized as a beam with a width of 1 meter, because the dimensions and reinforcement have to be obtained per meter, and a varying height (which is in fact the thickness of the wall). The varying heights can be found in appendix I – table 2 and lead to different reinforcement ratios. Thicknesses of 0.5 to 1.1 m satisfy the reinforcement ratio. A thickness of 0.8 m is the most economic option, this seems a good thickness for the walls of the small lock, so this value is chosen.

d (m)	M_u/bd^2f_{cd}	ρ	Concr (m^3)	Steel (m^3)	Concr (₫)	Steel (₫)	Total cost (₫)
0.5	217.94521	1.57	3.75	0.058875	4772726.25	6644985.75	11417712
0.6	151.35084	1.02	4.5	0.0459	5727271.5	5180549.4	10907821
0.7	111.19653	0.58	5.25	0.03045	6681816.75	3436769.7	10118586
0.8	85.134846	0.43	6	0.0258	7636362	2911942.8	10548305
0.9	67.267039	0.35	6.75	0.023625	8590907.25	2666459.25	11257367
1	54.486301	0.28	7.5	0.021	9545452.5	2370186	11915639
1.1	45.030001	0.23	8.25	0.018975	10499997.75	2141632.35	12641630

Appendix I - table 2: Different lock thicknesses

Shear stress check

To check if the concrete walls of the lock can handle the shear forces a shear resistance check is performed. First the shear resistance of the concrete ($V_{Rd,c}$) is calculated, without the use of shear reinforcement, as found in the manual.

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d \quad [\mathrm{N}]$$

With a minimum of:

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad [\mathrm{N}]$$

In this formula, the parameters are specified as follows:

f_{ck} [N/mm^2] = characteristic compressive cylinder strength of concrete at 28 days in MPa, in this case concrete of class C35/45 is used, so f_{ck} = 35 N/mm^2

k [-] = $1 + \sqrt{\frac{200}{d}} \leq 2.0\ with\ d\ in\ mm$ The value of d follows from the previous assumed thickness of the concrete wall. In this case $k = 1 + \sqrt{\frac{200}{800}} = 1.5$

ρ1 [-] = reinforcement ratio for longitudinal reinforcement, the assumed ratio of before is used, in this case that is 0.43%

σ_{cp} [N/mm^2] = compressive stress in the concrete from axial load or pre-stressing, in this case there is no pre stress or axial load present. This means that this value becomes 0. This also means that k_1 is not needed.

$C_{RD,c}$ [-] = a coefficient, in the Netherlands: 0.12 From discussion with our client it followed that Dutch values are used, because they are definitely safer than Vietnamese standards.

v_{min} [] $= 0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} = 0.035 * 1.50^{\frac{3}{2}} * 35^{\frac{1}{2}} = 0.38$

Filling in the formula leads to:

$$V_{Rd,c} = \left[0.12 \cdot 1.47 \cdot (100 \cdot 0.43 \cdot 35)^{\frac{1}{3}}\right] \cdot 1000 \cdot 800 = 1617\ kN$$

The minimum value is also calculated:

$$V_{Rd,c} = (0.38) \cdot 1000 \cdot 800 = 304\ kN$$

The design value of the shear force on the structure is 189 kN/m (resulting water force) and the values determined using Scia were a maximum of around 1000 kN/m. So the maximum total shear resistance has to be at least 1189 kN.

The shear resistance is well above the maximum shear resistance that is needed. This means that no shear reinforcement is needed in the walls of the small lock.

Dimensioning the lock wall, for the lock of variant B

Designing the lock wall is an iterative process. For the design steps, the process as described in chapter 35 of the manual hydraulic structures (2016) is followed. Four parameters are needed for this calculation. First the concrete class has to be selected. A normal class, that can easily be accessed around the world and that can also be produced in Vietnam, is C35/45. This is the first parameter. The design strength of the concrete f_{cd} is obtained by dividing the strength over the safety factor (f_{ck}/γ_c). In this case f_{cd} is calculated to be: $\frac{35\frac{N}{mm^2}}{1.2} = 29.2\frac{N}{mm^2}$

The second parameter that is already known is the height of the sluice. This height is equal to 7.5 m. This takes the maximum height of the water into account, the depth of the lock and the height of the lock above the water.

The third parameter that has to be taken into account for an economical design is the percentage of reinforcement (ρ) that is needed. In the case of concrete class C35/45 the value for has a maximum (ρ_{max}) of 2.49 and a minimum value (ρ_{min}) of 0.21. The selected value for ρ has to be between these values.

Governing forces and bending moment

The fourth parameter is the force and bending moment that the concrete has to transfer. Both walls of the lock will be evaluated to determine the governing bending moment. First, the lock wall adjacent to the quay will be elaborated. Secondly, the wall adjacent to the barrier will be elaborated.

Lock wall adjacent to quay:

- This wall is subjected to a maximum water level of +3.62 m and a soil level of +1.4 m. Besides, the walls of the lock exert a force on the wall. The force exerted by the soil per meter can be calculated as follows:

$$Q = \frac{1}{2}\gamma * h^2 * K_p^* - 2 * c * h\sqrt{K_p}$$

In which:
c [kN/m²] = cohesion = 7.6 kN/m²
γ [kN/m³] = volumetric soil weight = 15 kN/m³
h [m] = length of soil layer = 4.9 m

$$K_p = \frac{1+\sin(\varphi)}{1-\sin(\varphi)}\ (unsaturated) \qquad K_p^* = K_p\left(1 - \frac{\gamma_w}{\gamma}\right) + \frac{\gamma_w}{\gamma}\ (saturated)$$

In which:
ϕ (°) = angle of repose = 5
γ_w [kN/m³] = volumetric water weight = 10 kN/m³

- For the length of the soil layer, a ground level of +1.4 m is chosen because this is the lowest ground level at the embankments. This gives an h of 3.5 + 1.4 = 4.9 m. Now the force exerted by the ground can be calculated:

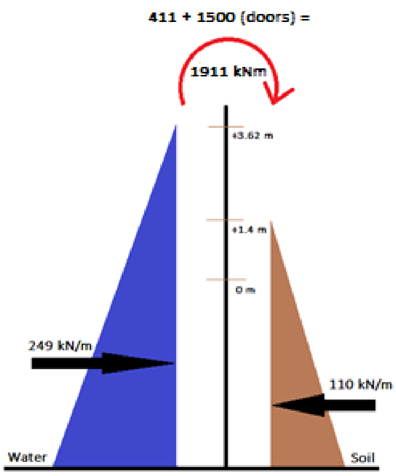
$$K_p = 1.19 \qquad K_p^* = 1.06 \qquad Q = 110\ kN/m$$

- The force exerted by the water per meter can be calculated as follows:

$$q_h = \int_{-3.5}^{3.62} \rho g h \, dh = \frac{1}{2}\rho g h^2 = 0.5 * 1000 * 9.81 * 7.12^2 = 249\ kN/m$$

- The resulting force becomes 249 – 110 = 139 kN/m in the situation of maximum high water inside the lock chamber. This results in a clockwise moment:

$$M = \left(\frac{1}{3} * 110 * 4.9\right) - \left(\frac{1}{3} * 249 * 7.12\right) = 411\ kNm$$



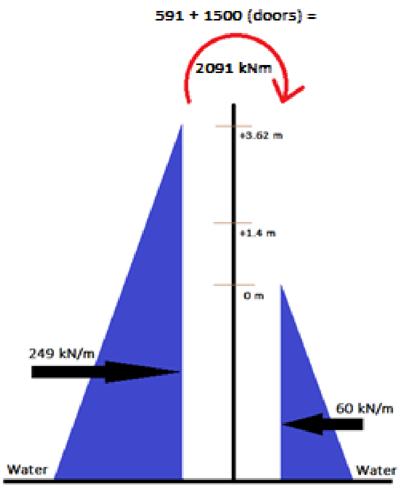
Appendix I - figure 8: Resulting forces

An overview of the situation and the forces and moment can be found in figure 8.

The bending moment due to the resultant force of the soil and the water is 411 kNm. There will be another bending moment in the wall, due to the gates which are transferring the hydrostatic pressure to the wall. This moment is not calculated exactly but it is assumed to be in the order of magnitude of 1500 kNm, because this is the average of different calculations with the program Scia. This moment works in the same direction as the moment due to the ground and water force in the lock chamber. Therefore, the walls of the lock have to withstand a moment of 1911 kNm.

NOTE: the moment is not calculated exactly because there is a lack of knowledge about the transferring of moments by the hinges of the doors.

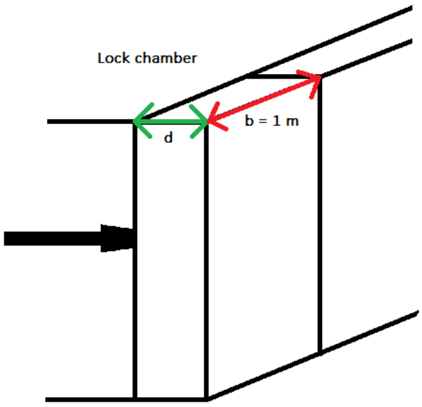
Lock wall adjacent to barrier



Appendix I - figure 9: Resulting forces

The second wall is the wall adjacent to the barrier. This wall is not subjected to soil forces but only to hydrostatic forces and the forces from the doors. The bending moment due to the resultant force of the water inside and outside the lock chamber is 592 kNm. The same bending moment due to the doors has to be added so the total bending moment becomes 2092 kNm. This is larger than the wall adjacent to the quay and therefore this moment is governing.

Using the given parameters, a first thickness of the door is assumed. This thickness is used to calculate the following parameter: $\frac{M_u(breaking\ moment)}{b\ (width) \cdot \mathrm{d\ (height)}^2 \cdot \mathrm{f}_{cd}(design\ strength)}$. The value of this parameter can be looked up in table 35-8 of the manual hydraulic structures. This leads to a value for ρ, which has to be between the given values (between 0.21 and 2.49).



Appendix I - figure 10: Wall schematization

For the calculation of the parameter, the lock wall is schematized as a beam with a width of 1 meter, because the dimensions and reinforcement have to be obtained per meter, and a varying height (which is in fact the thickness of the wall). The varying heights can be found in Appendix I - table 3 and lead to different reinforcement ratios. Thicknesses of 0.6 to 1.4 m satisfy the reinforcement ratio. A thickness of 0.6 m is the most economic option, but a thickness of 0.9 m is chosen because this is only slightly more expensive and it makes the construction easier because less reinforcement has to be used, this will save in the use of labor and time that is needed to place more reinforcement. These factors are not included in the given costs.

d (m)	M_u/bd^2f_{cd}	ρ	Concr (m^3)	Steel (m^3)	Concr (dong)	Steel (dong)	Total cost
0.6	199.01065	1.41	4.5	0.0441	5727271.5	4977390.6	10704662
0.7	146.21191	0.98	5.25	0.038325	6681816.75	4325589.45	11007406
0.8	111.94349	0.73	6	0.0354	7636362	3995456.4	11631818
0.9	88.44918	0.59	6.75	0.026325	8590907.25	2971197.45	11562105
1	71.643836	0.45	7.5	0.03375	9545452.5	3809227.5	13354680
1.1	59.209782	0.39	8.25	0.032175	10499997.75	3631463.55	14131461
1.2	49.752664	0.32	9	0.0288	11454543	3250540.8	14705084
1.3	42.392802	0.25	9.75	0.024375	12409088.25	2751108.75	15160197
1.4	36.552977	0.22	10.5	0.0231	13363633.5	2607204.6	15970838

Appendix I - table 3: Thickness of the wall

Shear stress check

To check if the concrete walls of the lock can handle the shear forces a shear resistance check is performed. First the shear resistance of the concrete ($V_{Rd,c}$) is calculated, without the use of shear reinforcement, as found in the .

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{\frac{1}{3}} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d \quad [\mathrm{N}]$$

With a minimum of:

$$V_{Rd,c} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d \quad [\mathrm{N}]$$

In this formula, the parameters are specified as follows:

f_{ck} [N/mm²] = characteristic compressive cylinder strength of concrete at 28 days in MPa, in this case concrete of class C35/45 is used, so f_{ck} = 35 N/mm²

k [-] = $1 + \sqrt{\frac{200}{d}} \leq 2.0$ *with d in mm* The value of d follows from the previous assumed thickness of the concrete wall. In this case $k = 1 + \sqrt{\frac{200}{900}} = 1.47$

ρ1 [-] = reinforcement ratio for longitudinal reinforcement, the assumed ratio of before is used, in this case that is 0.59%

σ_{cp} [N/mm²] = compressive stress in the concrete from axial load or pre-stressing, in this case there is no pre stress or axial load present. This means that this value becomes 0. This also means that k_1 is not needed.

$C_{RD,c}$ [-] = a coefficient, in the Netherlands: 0.12 From discussion with our client it followed that Dutch values are used, because they are definitely safer than Vietnamese standards.

v_{min} [] = $0.035 \cdot k^{\frac{3}{2}} \cdot f_{ck}^{\frac{1}{2}} = 0.035 * 1.47^{\frac{3}{2}} * 35^{\frac{1}{2}} = 0.37$

Filling in the formula leads to:

$$V_{Rd,c} = \left[0.12 \cdot 1.47 \cdot (100 \cdot 0.45 \cdot 35)^{\frac{1}{3}}\right] \cdot 1000 \cdot 900 = 2022\ kN$$

The minimum value is also calculated:

$$V_{Rd,c} = (0.37) \cdot 1000 \cdot 900 = 333\ kN$$

The design value of the shear force on the structure is 189 kN/m (resulting water force) and the values determined using Scia were a maximum of around 2000 kN/m. So the maximum total shear force has to be at least 2189 kN.

This means that the walls are too slim. By increasing the wall thickness to 1000 mm, the following result is achieved, using the corresponding value for ρ and a value of 1.45 for k:

$$V_{Rd,c} = \left[0.12 \cdot 1.45 \cdot (100 \cdot 0.59 \cdot 35)^{\frac{1}{3}}\right] \cdot 1000 \cdot 1000 = 2022\ kN$$

This means that no shear reinforcement is needed.

Appendix J – Foundation design

Preface

This appendix describes the method and calculations which lead to the pile plan for the two variants of the project.

Normative soil conditions and pile types

The soil parameters used are based on 5 SPT's and borings as shown in chapter 2.2.7. It is assumed that the layers change linear between the measured locations. The tips of the piles should be in layer number 4a. Therefore the depth of the piles differs over the construction area, with a minimum at HK1 (-29.55 m MSL) and the maximum at HK2 (-35.46 m MSL). This means that the depth of the deepest pile will be in the order of 30 meters as the bottom of the barrier and lock will be on -5 meters.
For this situation concrete piles are a good option to use, because they can bear a high load capacity, can be driven through the soft layers and are resistant to corrosion. Because of the high length of the piles they should have a diameter of more than 400 mm. The pile types shown in Table J- 1 are taken into consideration during this design, they all have a concrete class of C45/55, and the reinforcement percentage is taken 0.4 %. It is assumed that driving the piles will be no problem for the area because there are no historical or high buildings near the building location. If driving the piles turn out to be not possible, screwed piles can be used. In this case the reinforcement percentage should be higher because they have to take the tension stresses without the possibility of prestressing the pile.

Diameter	A	Internal capacity				Mass/m	Bearing capacity		Tension capacity	
		Pressure	Tension	$EI_{tension}$	M_{max}		$R_{c;d;sls}$	$R_{c;d;uls}$	$R_{s;k;sls}$	$R_{s;k;uls}$
[mm]	[mm^2*10^3]	[kN]	[kN]	[kNm^2]*10^{-6}	[kNm]	[kg]	[kN]	[kN]	[kN]	[kN]
400	160	8614	742	10.8	278	384	4799	3692	1718	1235
420	176.4	9497	818	13.1	307	423	5185	3989	1846	1327
450	202.5	10902	940	17.2	352	486	5791	4454	2047	1471
500	250	13460	1160	26.3	435	600	6869	5284	2401	1726

Table J- 1: Pile properties

In order to calculate the bearing and tension capacities of the pile, a normative SPT is taken. In this case HK3 is taken as normative, because the tip bearing has a lower value at this point due to the small layer thickness of the sand layer and the friction will have a relative low value due to the short pile length. The top level of the pile is presumed -5 m MSL.

Capacity of piles

Bearing capacity

In order to calculate the bearing capacity following Eurocode 7, the shaft resistance and base resistance are calculated following the method of Shariatmadari et al. (2008). The following formulas apply.

$$R_{c;d} = R_{b;d} + R_{s;d} = \frac{R_{b;k}}{\gamma_b} + \frac{R_{s;k}}{\gamma_s}$$

$$R_{b;k} = A_b * q_{b;k} \qquad R_{s;k} = \sum_i A_{s;i} * q_{s;i;k}$$

$$q_{b;k} = 385 * N_{g;b}$$

$$q_{s;i;k} = 3.65 * N_{g;s}$$

Where γ_b and γ_s are both 1.0 for SLS and 1.3 for ULS following the Eurocode, $N_{g;b}$ is the average of the N values between 8D above and 4D below the pile base and $N_{g;s}$ is the geometrical average of N values along the pile. $R_{c;d}$ is the resulting pile resistance.

In order to gain the maximum resistance the pile tip needs to be 8 times the diameter into the sand layer. For each pile the resistance is calculated, the results are shown in Table J- 2. These results are a good estimation for the resistance of each pile in the pile plan design process.

Diameter	A	Tip	Lenght	$N_{g;b}$	$N_{g;s}$	$q_{b;k}$	$q_{a;k}$	$A_{s;i}$	$R_{b;k}$	$R_{s;k}$	$R_{c;d;sls}$	$R_{c;d;uls}$	Prestress
[mm]	[mm^2]	[m MSL]	[m]	[blows]	[blows]	[kN/m^2]	[kN/m^2]	[m^2]	[kN]	[kN]	[kN]	[kN]	[N/mm^2]
400	160000	-32.92	27.92	45.8	12.1	17646	44.2	44.7	2823	1976	4799	3692	3.1
420	176400	-33.08	28.08	45.1	12.3	17356	45.0	47.2	3062	2124	5185	3989	2.9
450	202500	-33.32	28.32	44.1	12.7	16969	46.2	51.0	3436	2354	5791	4454	2.6
500	250000	-33.72	28.72	42.7	13.2	16427	48.1	57.4	4107	2762	6869	5284	2.3

Table J- 2: Pile bearing capacity

Tension capacity

The tension capacity of the piles is easy to find based on previous calculations. Following Eurocode 7 the shaft friction can be assumed the same in both directions. Therefore the following formulas apply and the tension capacities are shown in Table J- 1. The value of $\gamma_{s;t}$ is defined as 1.15 in SLS and 1.6 in ULS.

$$R_{t;d} = \frac{R_{t;k}}{\gamma_{s;t}} \qquad R_{t;k} = R_{s;k} = \sum_i A_{s;i} * q_{s;i;k}$$

Concluding it can be seen that the tension strength of the pile itself limits the capacity of the pile. Therefore, when needed, prestressing of the pile can be used to increase the tension strength of the piles. In order to strengthen the piles enough they have to be prestressed until they have the strength of the tension capacity of the soil. The needed prestress is shown in Table J- 2.

Lateral capacity

In order to calculate the horizontal capacity of the piles the method of Baguelin (1972) and Briaud (1997) are combined and used. The capacity is given by the following equations:

$$y_{0;1} = \frac{H * L_c^{\ 3}}{12EI} \qquad y_{0;2} = \frac{H}{l_0 k_m B} \qquad M_z = -\frac{Hl_0}{2}$$

Where $y_{0;1}$ is the simple method described by Tomlinson (1994) and $y_{0;2}$ the extended method taking soil properties into account. The second method is used because of the high length of the piles, this leads to a big displacement at the top when the soil effects are not taken into account. M_z is the maximal moment in the pile. The following formulas describe the parameters (L. Behpoor, xxxx):

$$l_0 = \left(\frac{4EI}{K}\right)^{\frac{1}{4}} \qquad \frac{1}{k_m} = \frac{B}{E_m}\frac{4(2.65)^{\alpha} + 3\alpha}{18}$$

$$K = 2.3E_m \qquad E_m = 180 * N_a$$

The piles are modeled with fixed head, because of the significance the depth is taken the same for each pile at -32.92 m MSL. The spring value is taken from a depth of -6.5 m MSL, because the first 1.5 meters should be neglected for this calculation. The resulting E_m value becomes 2.3 * 10^3 kN/m^2, K is 5.2 * 10^3 kN/m^2.

With these factors the values of k_m and l_0 are calculated for each pile diameter and as a result the maximum allowable lateral force on a pile is obtained. If the maximum allowable displacement at the top is taken at 0.01 meter, the normative horizontal force is given by the moments at the top of the piles and not by the displacements. The results are shown in Table J- 3.

Pile type	EI	l_0	k_m	M_z/H	$y_{0;2}$/H	$H_{max;SLS}$	$H_{max;ULS}$
[mm]	[kNm^2]	[m]	[kN/m]	[kNm/kN]	[mm/kN]*10^{-3}	[kN]	[kN]
400	10.8	0.30	9804	-0.15	0.85	1854.3	1158.9
420	13.1	0.32	9337	-0.16	0.81	1947.0	1216.9
450	17.2	0.34	8714	-0.17	0.75	2086.1	1303.8
500	26.3	0.38	7843	-0.19	0.68	2317.9	1448.7

Table J- 3: Lateral pile capacities

Settlements

The settlements of the total construction are given by the book of R. Rajapakse (2008) following the given equation:

$$S_{total} = S_a + S_t = \frac{(Q_p + aQ_s) * L}{AE_p} + \frac{C_p * Q_p}{B * q_0} \ (Vesic, 1977)$$

Following the results of the bearing capacity, the pile tip point load (Q_p) is assumed 3/5 of the total load and the shaft load (Q_s) 2/5 of the bearing capacity. For q_0 a value of 19.2 kN/m^2 is taken, following the equations presented under bearing capacity and a blow count of 50. The value of alpha is taken 0.5 for clayey soils and C_p 0.04 for driven piles in loose sand. The pile length is taken maximal for the project, resulting in a length of 35 meters. The results are presented in Table J- 4.

Pile type	EA	S_t/Q	S_p/Q	S_{total}/Q	$S_{total;des;SLS}$
[mm]	[kN/m^2]	[mm/kN]	[mm/kN]	[mm/kN]	[mm]
400	8480	0.0031	0.0062	0.0093	34.4
420	9349	0.0030	0.0056	0.0086	34.3
450	10733	0.0028	0.0049	0.0077	34.2
500	13250	0.0025	0.0040	0.0065	34.1

Table J- 4: Pile settlements

The results show the settlement per kN of axial force applied. Also the total settlement is shown in case of the design load resulting from the bearing capacity, this shows that the settlements are within the boundary of 50 mm. Therefore the bearing capacity will be normative for the design and not the settlements.

Normative situations

In order to design the pile plan, the normative situations have to be found. First the assumption that the locks can be considered completely stiff is argued. Following the limit states for both designs (A and B) are calculated. Based on this limit states the pile plan is designed.

Stiffness

In order to find the normative situations first the assumption that all the locks are infinite stiff has to be controlled following the criterion of van Tol (2006). The least stiff lock is the one with the floor thickness of 0.4 meter. A load of 70 kN/m^2 is taken as a guess of the resulting load. Through a small calculation a maximal pile distance needed is found 6 meters. This results in the following calculation:

$$\Delta k = \frac{\Delta_{found}}{\Delta_{free}} = 2.27$$

Where:

$$\Delta_{found;min} = D^2 * q * \frac{S_{total}}{q} = 6^2 * 70 * 0.0065 = 16.4\ mm$$

$$\Delta_{free;max} = \frac{1}{384} * \frac{q * l^4}{EI} = \frac{1}{384} * \frac{70 * 12^4}{\frac{1}{12} * h^3 * E} = 7.2\ mm$$

A value for Δk above 2.0 suggests a stiff lock. In the shown case the value is just above two, but in case of a smaller distance between the piles this value quickly becomes higher. Therefore the assumption the locks are infinite stiff is correct.

The basic weights

In this chapter different forces are calculated for different scenarios. It is assumed that a horizontal force is positive if it works in the direction of Ho Chi Minh City, so coming from the Mekong delta side. A moment is considered positive if it is in clockwise direction and vertical forces are positive if they are working in the direction of gravity.

Variant A

Big lock

As shown in the calculation for the floor of the big lock, the weight for the walls is 16 kN/m^2, the weight for the doors is 0.4 kN/m^2 and the weight of the floor is (23kN/m^3*0.57 m =) 13.11 kN/m^2. The lift towers weigh in at ((4 * 354 kN)/(140 m * 23 m) = 0.4 kN/m^2. The total weight of the concrete structure of the big lock, without water present is thus: 30 kN/m^2.

Small lock

It also follows from the floor calculations that the weight of the small lock will be: the floor at (23 * 0.4 =) 9.2 kN/m^2 + the walls at 31 kN/m^2 and the gates at 0.3 kN/m^2, thus the total weight of the concrete is equal to 40.5 kN/m^2.

Culvert dam

The culvert dam is 31.4 meters wide and it is 9.5 meters high. From +1 MSL to -3.5 MSL there are culverts. The 5 culverts are parted by 4 slabs of 0.6 meters width. The dam is 2 meters wide. This results in a weight of:

$$W = A * D * \gamma = \big(31.4 * (1 + 3) + (4 * 4.5 * 0.6)\big) * 2 * 23 = 6320\ kN$$

This force needs to be taken into account half for each of the locks. So 6320/2 = 3160 kN added weight for both locks.

Variant B

Big lock

The big lock is almost the same as the big lock in variant A. However mitre gates are used instead of lift gates. This means that the lift towers will not be present. The difference in weight for the doors can be neglected, just as it is done for the dimensioning of the floors of the lock.
The total weight of the big lock of variant B becomes thus 29.6 kN/m^2.

Tidal barrier

The tidal barrier only has its two towers at 354 kN each. The weight of the gate is 525 kN. The sill is 1 m by 1 m by 40 m. So a total of 40 m^3 at a weight of 25 kN/m^3 is 1000 kN. The total weight of the structure is thus 354+354+525+1000 = 2233 kN.

Different scenarios

Different scenarios are investigated to determine the maximum forces on the structure in the governing cases and thus the governing forces on the foundation. The different scenarios are found in Table J- 5.

	Water level Mekong (in m, MSL)	Water level Lock (in m, MSL)	Water level City (in m, MSL)
ULS 1	3.62	3	0
ULS 2	-1.5	1	1
ULS 3	1	1	1
ULS 4	2	-3.5	0.6
SLS	2	2	0.6

Table J- 5: ULS and SLS scenarios

ULS 1 is the situation with a maximum water level at the outside of the structure, while a buffer capacity is maintained at the inside.
ULS 2 is the situation that water is retained at the city side, for health and navigation purposes
ULS 3 is the situation where the maximum water levels are present for free flow.
ULS 4 is the situation that repairs are being performed on the lock, so the lock is pumped dry.

ULS 1, high water at the Mekong delta side

Variant A, big lock

In this case, water is at its highest at the outside of the structure. The water level here is MSL + 3.62 m. The water inside of the structure is at its lowest here and is equal to MSL. The water level inside the lock is maximum here, for still being able to lock, so MSL + 3.0 m.

The bottom of the lock is found at MSL - 3.5 m. The water levels are visualized in Figure J- 1, including the resulting forces. The resulting force per m is calculated using the following formula for the Mekong delta side:

$$F' = \frac{1}{2} * \rho * g * h^2 = \frac{1}{2} * 1000 * 9.81 * 7.12^2 = 249 \frac{kN}{m}$$

The total force over the full 23 m width of the lock thus becomes 5727 kN. This can be schematized by a force acting at a height of $\frac{1}{3} * 7.12\ m = 2.37\ m = MSL - 1.13\ m$.

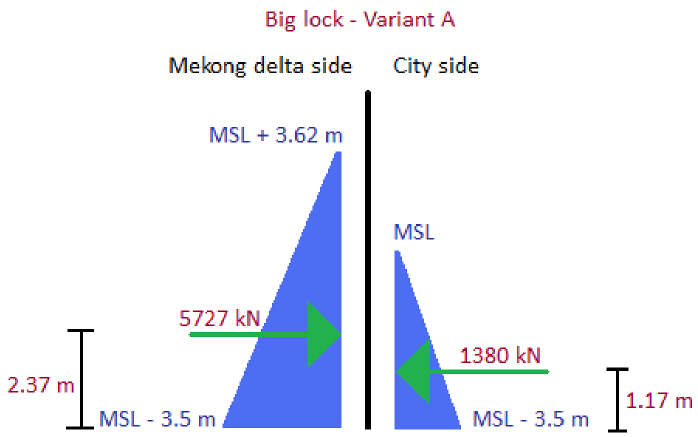


Figure J- 1: Water levels and forces

For the city side it is: $F' = \frac{1}{2} * 1000 * 9.81 * 3.5^2 = 60 \frac{kN}{m}$. Thus over the full length of 23 m this force becomes 1380 kN. This force acts at a height of $\frac{1}{3} * 3.5 = 1.17\ m = MSL - 2.33\ m$.

The resulting horizontal force is equal to 5727 kN – 1380 kN = 4347 kN. With the horizontal force of the culvert added this becomes: 4347 kN + 2967 kN = 7314 kN.

The water level in the lock is equal to MSL + 3.0 m. This means that a total weight of $\frac{B_w * L_w * h * \rho * g}{B * L} = \frac{21*120*3*1000*9.81}{23*120} = 27 \frac{kN}{m^2}$ is added to the lock. The total weight of the lock thus becomes (27+30)*23*120 = 157320 kN. Plus 3160 kN of the culverts result in 157320 + 3160 = 160480 kN. The upward force of the water pressure is not taken into account, this would lead to a lower value for the vertical force, so it is not taken into account for the Ultimate Limit State.

The resulting moment is: $5727\ kN * 2.37\ m - 1380\ kN * 1.17\ m = 11958\ kNm$ in the direction of the clock. Plus the moment of the culvert dam makes it: $11958\ kNm + 8163\ kNm = 20121\ kNm$.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
Without factor	7314	160480	20121
With factor (ULS)	9508	208624	26157

Table J- 6: ULS1 forces - Variant A, Big lock

Variant A, small lock

The horizontal force on the Mekong delta side works over a length of 11.6 m and is thus equal to $11.6\,m * 249\frac{kN}{m} = 2888\,kN$. This force also acts at a height of $MSL - 1.13\,m$.

The horizontal force on the city side is equal to: $11.6\,m * 60\frac{kN}{m} = 696\,kN$, also acting at a height of $MSL - 2.33\,m$.

The resulting horizontal force is equal to 2888 kN – 696 kN = 2192 kN. With the horizontal force of the culvert added this becomes: 2192 kN + 2967 kN = 5159 kN.

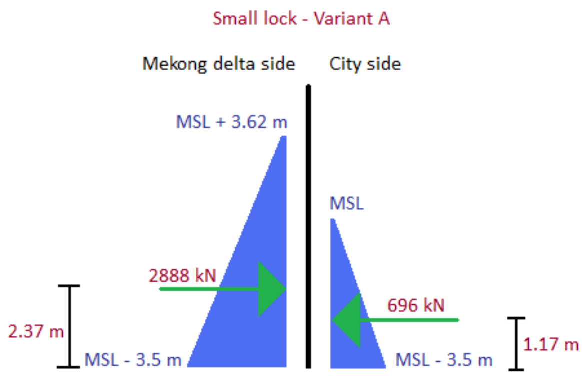


Figure J- 2: Water levels and forces

The vertical force in the small lock becomes: $\frac{B_W*L_W*h*\rho*g}{B*L} = \frac{10*70*3*1000*9.81}{11.6*85} = 21\frac{kN}{m^2}$. So the total force in vertical direction becomes: $(21 + 40.5) * (11.6 * 5) = 60639\,kN$. Plus 3160 kN of the culverts result in 60639 + 3160 = 63799 kN.

The resulting moment is: $2888\,kN * 2.37\,m - 696\,kN * 1.17\,m = 6030\,kNm$ in the direction of the clock. Plus the moment of the culvert dam makes it: $6030\,kNm + 8163\,kNm = 14193\,kNm$.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
Without factor	5159	63799	14193
With factor (ULS)	6706	82938	18451

Table J- 7: ULS1 forces - Variant A, Small lock

Variant A, culvert dam

The horizontal force on the Mekong delta side works over a length of 31.4 m and is thus equal to $31.4\,m * 249\frac{kN}{m^2} = 7819\,kN$. This force also acts at a height of $MSL - 1.13\,m$.

The horizontal force on the city side is equal to $31.4\,m * 60\frac{kN}{m^2} = 1884\,kN$, also acting at a height of $MSL - 2.33\,m$.

The resulting horizontal force is equal to 7819 kN – 1884 kN = 5935 kN.

There is no vertical force on the culvert dam due to the water.

The resulting moment is: $7819\,kN * 2.37\,m - 1884\,kN * 1.17\,m = 16326\,kNm$ in the direction of the clock. Because half of these forces need to be taken by the locks on both sides of the culvert the forces added become: F_h = 5935/2 = 2967 kN and M = 16326/2 = 8163 kNm.

Variant B, tidal barrier

The force on the tidal barrier is now exerted for 40 m. The forces are the same per m as shown for the big lock in variant A.

$$249 \frac{kN}{m} * 40\ m = 9960\ kN$$

In the other direction:

$$60 \frac{kN}{m} * 40\ m = 2400\ kN$$

The resulting horizontal force becomes: 9960 kN – 2400 kN = 7560 kN.

This is schematized in Figure J- 3.

The resulting moment in clockwise direction thus becomes:

$$9960\ kN * 2.37\ m - 2400\ k * 1.17\ m = 20792\ kNm$$

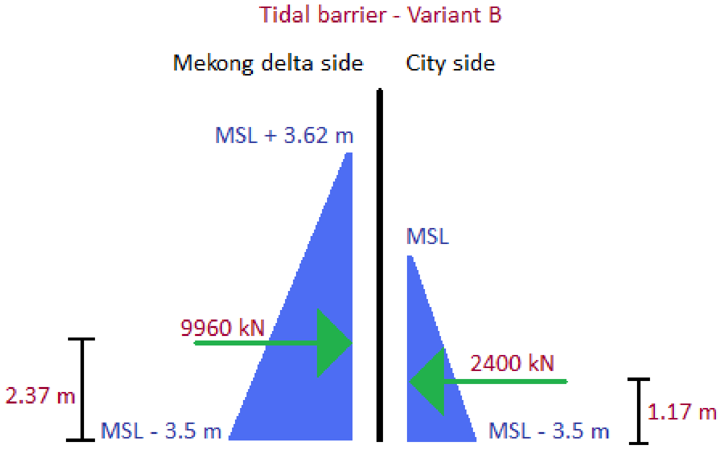


Figure J- 3: Water levels and forces

The vertical force is equal to the weight of the gate + two lift towers + the sill = 525 kN + (2 * 354 kN) + (1 m * 1 m * 40 m * 25 kN/m^3) = 2233 kN. Half of this forces have to be included by the pillar on one side of the canal, and the other by the big lock. This results in half the forces of: F_h = 3780 kN, F_v = 1117 kN and M = 10396 kNm.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
Without factor	3780	1117	10396
With factor (ULS)	4914	1452	13515

Table J- 8: ULS1 forces - Variant B, Tidal barrier

Variant B, big lock

The horizontal forces and the moment are the same as for the big lock in variant A. The only parameter that changes is the vertical force, without lift. Using mitre gates instead of lift gates, this force becomes:

$$\frac{B_w * L_w * h * \rho * g}{B * L} = \frac{21 * 111 * 3 * 1000 * 9.81}{23 * 140} = 21 \frac{kN}{m}$$

The total vertical force thus becomes: $(21 + 29.6) * (23 * 140) = 162932\ kN$

The weight, lateral force and moment of the big lock plus half the tidal barrier become:

$F_v = 162932 + 1117 = 164049\ kN, F_h = 4347 + 3780 = 8027\ kN,\ \ M = 11958\ kNm + 10396\ kNm = 22354\ kNm.$

	F_h	F_v	M
	[kN]	[kN]	[kNm]
Without factor	8027	164049	22354
With factor (ULS)	10435	213264	29060

Table J- 9: ULS1 forces - Variant B, Big lock

ULS 2, high water at the city side

In this case, water is at its highest on the inside of the structure. The water level here is MSL + 1.0 m. The water outside of the structure is at its lowest here and is equal to MSL – 1.5 m. The water level inside the lock is maximum here, so also MSL + 1.0 m.

The resulting forces will be smaller than all the resulting forces calculated for ULS 1. This means that ULS 2 is not governing and thus will not be further investigated.

ULS 3, water free flowing

In this case, no horizontal forces are present, due to the fact that the water level is equal at all sides of the structures, because the big lock of variant A and the tidal barrier are open. The small lock of variant A and the big lock of variant B are both closed in this case, but water levels are assumed equal on each side and around MSL +1 m. This will not lead to governing forces for horizontal or vertical pressure. One possibility however is that wind blows against an open lift gate. The resulting moment is also calculated in the design of the lift towers.
The resulting moment due to wind forces will be the highest for the tidal barrier, due to the fact that it has a greater span. Is calculated before, the maximum moment due to the wind is $234\ kN * 13.25\ m = 3100\ kNm$. This is quite low, compared to the maximum moment that is exerted when the barrier is closed, so this moment is not considered governing.

SLS, settlement

Variant A, big lock

In this case, the average height at each side of the structure is taken into account. These average loads are used to determine the settlement of the entire structure. The water level at the Mekong delta side is MSL + 2.0 m. The water inside of the structure is equal to MSL + 0.6 m. The water level inside the lock is equal to the maximum water level here, and thus equal to MSL + 2.0 m.
The bottom of the lock is found at MSL - 3.5 m. The resulting force per m is calculated using the following formula for the Mekong delta side:

$$F' = \frac{1}{2} * \rho * g * h^2 = \frac{1}{2} * 1000 * 9.81 * 5.5^2 = 148 \frac{kN}{m^2}$$

The total force over the full 23 m width of the lock thus becomes 3413 kN. This can be schematized by a force acting at a height of $\frac{1}{3} * 5.5\ m = 1.83\ m = MSL - 1.67\ m$.
For the city side it is: $F' = \frac{1}{2} * 1000 * 9.81 * 4.1^2 = 82 \frac{kN}{m}$. Thus over the full length of 23 m this force becomes 1896 kN. This force acts at a height of $\frac{1}{3} * 4.1 = 1.37\ m = MSL - 2.13\ m$.
The resulting horizontal force thus becomes: 3413 kN – 1896 kN = 1517 kN. . With the horizontal force of the culvert added this becomes: 1517 kN + 1036 kN = 2553 kN.
The water level in the lock is equal to MSL + 2.0 m. This means that a total weight of $\frac{B_W * L_W * h * \rho * g}{B * L} = \frac{21*120*2*1000*9.81}{23*120} = 18 \frac{kN}{m^2}$ is added to the lock. The total weight of the lock thus becomes (18+30)*23*120 = 132480 kN. Plus 3160 kN of the culverts result in 132480 + 3160 = 135640 kN. The upward force of the water pressure is not taken into account, this would lead to a lower value for the vertical force, so it is not taken into account for the Ultimate Limit State.
The resulting moment is: $3413\ kN * 1.83\ m - 1896\ kN * 1.37\ m = 3648\ kNm$ in the direction of the clock. Plus the moment of the culvert dam makes it: $3648\ kNm + 2485\ kNm = 6133\ kNm$.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
SLS	2553	135640	6133

Table J- 10: SLS forces - Variant A, Big lock

Variant A, small lock

The horizontal force on the Mekong delta side works over a length of 11.6 m and is thus equal to $11.6\,m * 148\frac{kN}{m} = 1717\,kN$. This force also acts at a height of $MSL - 1.67\,m$.

The horizontal force on the city side is equal to $11.6\,m * 82\frac{kN}{m} = 951\,kN$, also acting at a height of $MSL - 2.13\,m$.

The resulting horizontal force is thus equal to: 1717 kN – 951 kN = 766 kN. With the horizontal force of the culvert added this becomes: 766 + 1036 kN = 1802 kN.

The vertical force in the small lock becomes: $\frac{B_w * L_w * h * \rho * g}{B * L} = \frac{10*85*2*1000*9.81}{11.6*85} = 17\frac{kN}{m^2}$. So the total force in vertical direction becomes: $(17 + 40.5) * (11.6 * 85) = 56695\,kN$. Plus 3160 kN of the culverts result in 56695 + 3160 = 59855 kN.

The resulting moment is: $1717\,kN * 1.83\,m - 951\,kN * 1.37\,m = 1839\,kNm$ in the direction of the clock. Plus the moment of the culvert dam makes it: $1839\,kNm + 2485\,kNm = 4319\,kNm$.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
SLS	1802	59855	4319

Table J- 11: SLS forces - Variant A, Small lock

Variant A, culvert dam

The horizontal force on the Mekong delta side works over a length of 31.4 m and is thus equal to $31.4\,m * 148\frac{kN}{m^2} = 4647\,kN$. This force also acts at a height of $MSL - 1.67\,m$.

The horizontal force on the city side is equal to $31.4\,m * 82\frac{kN}{m} = 2575\,kN$, also acting at a height of $MSL - 2.13\,m$.

The resulting horizontal force is thus equal to: 4647 kN – 2575 kN = 2072kN.

There is no vertical force in the culvert due to the water level.

The resulting moment is: $4647\,kN * 1.83\,m - 2575\,kN * 1.37\,m = 4977\,kNm$ in the direction of the clock.

Half of these forces are taken by the locks on the sides of the culvert.

Variant B, tidal barrier

The force on the tidal barrier is now exerted for 40 m. The forces are the same per m as shown for the big lock in variant A.

$$148\frac{kN}{m^2} * 40\,m = 5920\,kN$$

In the other direction:

$$82\frac{kN}{m^2} * 40\,m = 3280\,kN$$

The resulting force in horizontal direction is thus: 5920 kN – 3280 kN = 2640 kN.

The resulting moment in clockwise direction thus becomes:

$$5920\,kN * 1.83\,m - 3280\,kN * 1.37\,m = 6340\,kNm$$

The vertical force is equal to the force of ULS 1. So it is 2233 kN. Half of these forces are taken by the big lock, the other half is taken by the foundation at the other side. Leaving the forces there: F_h = 1320 kN, F_v = 1117 kN and M = 3170 kNm.

	F_h	F_v	M
	[kN]	[kN]	[kNm]
SLS	1320	1117	3170

Table J- 12: SLS forces - Variant B, Tidal barrier

Variant B, big lock

The horizontal forces and the moment are the same as for the big lock in variant A. The only parameter that changes is the vertical force, without lift. Using mitre gates instead of lift gates, this force becomes:

$$\frac{B_w * L_w * h * \rho * g}{B * L} = \frac{21 * 140 * 2 * 1000 * 9.81}{23 * 140} = 18\frac{kN}{m^2}$$

The total vertical force thus becomes: $(18 + 29.6) * (23 * 140) = 153272kN$

The weight, lateral force and moment of the big lock plus half the tidal barrier become:

$F_v = 153272 + 1117 = 154389\ kN, F_h = 1517 + 1320 = 2837\ kN,\ M = 3648\ kNm + 3170\ kNm = 6818\ kNm.$

	F_h	F_v	M
	[kN]	[kN]	[kNm]
SLS	2837	154389	6818

Table J- 13: SLS forces - Variant B, Big lock

ULS 4, maintenance to the lock

The forces are the same as for the SLS case, where they are already calculated. However the vertical forces and moments will be different if uplift is taken into account, because this is the worst case scenario, in which the locks might float.

Variant A, culvert dam

In order to calculate the tension forces due to the empty locks, the floating is suggested a high as possible. Therefore the floating of the culvert also needs to be taken into account. This leads to an area of 31.4 *2 = 62.8 m^2. When the upward pressure is 54 kN/m^2 this leads to a upward force of 54 * 62.8 = 3391.2 kN. Half of it is distributed to both of the locks, which leads to a F_v of -3391.2/2 = -1696 kN.

Variant A, big lock

The resulting horizontal force is 3413 kN – 1896 kN = 1517 kN.
The resulting moment is 3648 kN.

The total weight of the lock is 30*23*120 = 82800 kN. Plus half the weight of the culvert makes this: 82800 + 3160 = 85960 kN

The upward force of the water is calculated using the following water levels:

A water level of MSL+2.0 m, leads to a water height of 5.5 meter because the floor is located at MSL-3.5m. A water level of MSL + 0.6m is the level at the city side, to a total height of 4.1 m. Both water levels are schematized as forces that work on half the floor zone of the lock.

The upward pressure of the two zones is:

P1=ρ*g*h1 = 1000*9.81*5.5 = 54 kN/m^2

P2=ρ*g*h2 = 1000*9.81*4.1 = 40 kN/m^2

The total upward pressure on the floor is P1*zone 1 = 54 kN/m^2 * (23 m * 60 m) = 74520 kN and P2*zone 2 = 40 kN/m^2 * (23 m * 60 m) = 55200 kN, thus the total force upwards is: 129720 + 1696 = 131416 kN.

The moments following the water pressure are $\frac{60}{2} * F$. In this case the resulting moments are 74520*30 clockwise and 55200*30 counter clockwise. Adding the moments of the water in the canal this leads to the presented moments.

	F_v	M
	[kN]	[kNm]
Without factor positive	85960	2241733
Without factor negative	131416	1656000
ULS	-93476.8	1423853

Table J- 14: ULS4 forces - Variant A, Big lock

Variant A, small lock

The resulting horizontal force $1717\ kN - 951\ kN = 766\ kN$.
The resulting moment is $1839\ kNm$.
The total weight of the lock is 40.5*11.6*85 = 39933 kN. Plus half the weight of the culvert makes this: 39933 + 3160 = 43093 kN
The upward force of the water is calculated using the same water pressures as for the big lock, only the zones have dimensions of 11.6 m by 42.5 m.
The total upward pressure on the floor is P1*zone 1 = 54 kN/m^2 * (11.6 m * 42.5 m) = 26622 kN and P2*zone 2 = 40 kN/m^2 * (11.6 m * 42.5 m) = 19720 kN, thus the total force upwards is: 46342+1696 = 48038 kN.
The moments following the water pressure are $\frac{42.5}{2} * F$. In this case the resulting moments are 26622*21.25 clockwise and 19720*21.25 counter clockwise. Adding the moments of the water in the canal this leads to the presented moments.

	F_v	M
	[kN]	[kNm]
Without factor positive	43093	570036
Without factor negative	48038	419050
ULS	-23666	363902

Table J- 15: ULS4 forces - Variant A, Small lock

Variant B, big lock

The resulting horizontal force and the resulting moment are the same as for the big lock in variant A. The only parameter that changes is the resulting vertical force.
The weight of the lock becomes: $29.6 * (23 * 140) = 95312\ kN$. With the added weight of the tidal gate this becomes: $95312 + 1117 = 96439kN$.
The area of a zone is also different, namely 23 m by 70 m. Water pressures are the same as for the big lock in variant A. So the upward force is equal to:
P1*zone 1 = 54 kN/m^2 * (23 m * 70 m) = 86940 kN and P2*zone 2 = 40 kN/m^2 * (23 m * 70 m) = 64400 kN, thus the total force upwards is: 151340 kN.
The moments following the water pressure are $\frac{70}{2} * F$. In this case the resulting moments are 86940*35 clockwise and 64400*35 counter clockwise. Adding the moments of the water in the canal this leads to the presented moments.

	F_v	M
	[kN]	[kNm]
Without factor positive	96439	3049716
Without factor negative	151340	2254000
ULS	-109947	1936031

Table J- 16 - ULS4 forces - Variant B, Big lock

Variant B, tidal barrier

No forces are calculated for the tidal barrier, since for maintenance the gate can be lifted out of the water, while the sill is always under water. If this has to be maintained, a construction pit will have to be build.

Design of the Pile plan

With the found limit forces (Table J- 17) and capacities of the piles (Table J- 18) the pile plan can be made. Before that it will be explained how the moments are taken into account.

Part	Bearing (ULS)		Tension (ULS)		Lateral (ULS)	Settlement (SLS)		
	$F_{max;down}$	M_{max}	$F_{max;up}$	M_{max}	$F_{max;hor}$	$F_{v'SLS}$	$F_{h;SLS}$	M_{SLS}
	[kN]	[kNm]	[kN]	[kN]	[kN]	[kN]	[kN]	[kNm]
A – big lock	208624	25157	-93477	1423853	9508	135640	2553	6133
A – small lock	82938	18451	-23666	363902	6706	59855	1802	4319
B – big lock	213264	29060	-109947	1936031	10435	154389	2837	6818
B – pillar	1452	13515	-	-	4914	1117	1320	3170

Table J- 17: Normative forces on all parts

Diameter	Internal capacity		Soil capacitiy			Movement	
	Pressure	Tension	Lateral	Bearing	Tension	Vertical	Lateral
	$R_{p;p}$	$R_{p;t}$	$H_{max;ULS}$	$R_{c;d;uls}$	$R_{s;k;uls}$	S_{total}/F_v	$y_{0;2}/F_h$
[mm]	[kN]	[kN]	[kN]	[kN]	[kN]	[mm/kN]	[mm/kN]
400	8614	742	1159	3692	1235	0.0093	0.00085
420	9497	818	1217	3989	1327	0.0086	0.00081
450	10902	940	1304	4454	1471	0.0077	0.00075
500	13460	1160	1449	5284	1726	0.0065	0.00068

Table J- 18 - Normative pile capacities

Moments

In order to apply the moments on the foundation the reaction forces of the foundation are schematized as a continuous load as shown in Figure J- 4.

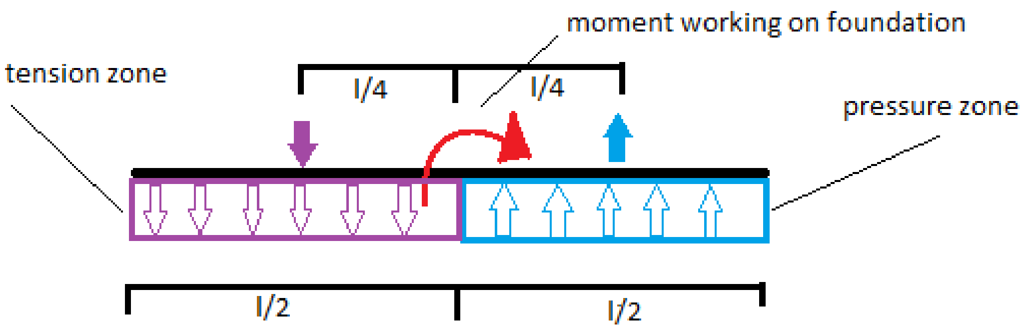


Figure J- 4: Moment forces on the lock floor

Following the moment should result in a reduction or increase in the pile forces. For each situation the normative forces are calculated for the worst part of the foundation. In this way the other side could be a little over dimensioned. In a detailed design this could be improved. Table J- 19 shows the resulting forces. The length and width for the pillar in variant B are designed in this step.

Part	Lenght	Width	Bearing (ULS)			Tension (ULS)			Settlement (SLS)		
			F_v	F_m	F_t	F_v	F_m	F_t	F_v	F_m	F_t
	[m]	[m]	[kN/m^2]			[kN/m^2]			[kN/m^2]		
A – big lock	120	23	75.59	0.30	75.89	-33.87	-17.20	-51.06	49.14	0.07	49.22
A – small lock	85	11.6	84.12	0.88	85.00	-24.00	-17.37	-41.37	60.70	0.21	60.91
B – big lock	140	23	66.23	0.26	66.49	-34.15	-17.18	-51.32	47.95	0.06	48.01
B – pillar	25	2	29.04	43.25	72.29	29.04	-43.25	-14.21	22.34	10.14	32.48

Table J- 19: Moments and forces added to kN/m2 forces

Tension (ULS)

Following it must be said that the high tension values following the empty lock will influence the design a lot. When looking at the pile capacities, they can take about three times as much pressure as tension. Because the values for tension are more than 2/3 of the forces for bearing capacity, the foundation needs to be double as strong, just so that the lock can be maintained in dry condition. There will be negligible tension in the structures if the normative situation of the calculation of the floor thickness is taken into account. Therefore there are two options:

- Design the foundation in order to take all the tension forces during maintenance with an empty lock. In this case the foundation will be governed by the tension of the piles and is over dimensioned with respect to the bearing capacity.
- Design the foundation with the normative situation during navigation. In this case maintenance cannot be done with a dry lock, but the foundation can be made less strong.

After deliberation the first option is chosen as best. It is assumed that the higher costs of the foundations do not outweigh the benefits of having the possibility for dry maintenance. Therefore the foundation is designed based on tension capacity.

Variant A – Big lock

Tension

The tension on the big lock is 51 kN/m^2. The tension capacity of the piles are in a range of 1200 - 1700 kN. When the piles are placed 5 meters apart in both directions each pile needs to bear 51 kN/m^2 * 25 m^2 = 1275 kN. This is a bit too much for the smallest piles. Taking the width of 23 meters into account it is suggested to place piles from left to right as shown in Figure J-5. In the depth of the lock the piles can be placed 5 meters apart. This results in a total of $\frac{120}{5} * 5 = 120\, piles$. Each pile needs to take a load equal to $\frac{120*23}{120} * 51 = 23\, m^2 * 51 \frac{kN}{m^2} = 1173\, kN$. Therefore the smallest piles of 400*400 mm shown in Table J- 18 can be used for the foundation. In this case the piles need to be prestressed to 3.1 N/mm^2. The factor of safety in this case becomes: $FOS = \frac{1235\, kN}{1173\, kN} = 1.05$

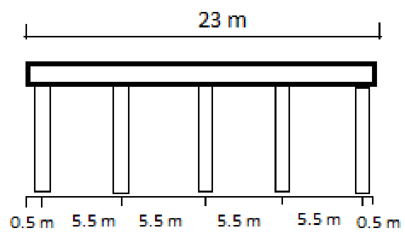


Figure J- 5: Pile distribution big lock

Because the pile are apart more than 8 times its own diameter group effects can be neglected.
The 'clod criterion' is checked in order to see if the weight between the piles is enough to take the tension forces. This means that the weight of the soil minus the water weight between the piles has to be higher than the total 'floating' force. The weight of the soil has a minimum value of 15 kN/m^2. This would result in a downward force of 5 kN/m^2, the total underestimated downforce between the piles becomes: $F_{soil} = 120\, m * 23\, m * 30\, m * 5\, kN = 414000\, kN$ while the total upward forces are 93277 kN (Table J- 17). There is no risk that the clod of the pile would be to light because the factor of safety becomes: $FOS = \frac{414000\, kN}{93277\, kN} = 4.43$

Bearing

The pressure on the big lock is 75.9 kN/m^2. The bearing capacity of the chosen pile is 3692 kN. When the piles are placed as explained in previous paragraph, each pile needs to bear 75.9 kN/m^2 * 23 m^2 = 1746 kN. This is far beneath the bearing capacity. The factor of safety in this case becomes: $FOS = \frac{3692\ kN}{1746\ kN} = 2.11$

Shear forces

The total lateral force in the normative situation on the lock is 9508 kN. There are 120 piles to take this lateral force, leading to a lateral force of $\frac{9508}{120} = 80\frac{kN}{pile}$. This is far below the stated lateral capacity of 1159 kN. The factor of safety in this case becomes: $FOS = \frac{1159\ kN}{80\ kN} = 14.48$. Therefore the foundation meets the ULS requirements.

Settlements and displacements

Although the settlements and displacements are earlier shown not to be normative the displacements in the foundation are calculated. The maximal horizontal and vertical forces that work on a pile are $F_v = \frac{135640}{120} = 1130\ kN$ and $F_h = \frac{2553}{120} = 21\ kN$. This results in a displacements of $u_v = 1130\ kN * 0.0093\ mm/kN = 10\ mm$ and $u_h = 21\ kN * 0.00085 = 0.018\ mm$.

Variant A – Small lock

Tension

The tension on the small lock is 41 kN/m^2. The tension capacity of the piles are in a range of 1200 - 1700 kN. When the piles are placed 6 meters apart in both directions each pile needs to bear 41 kN/m^2 * 36 m^2 = 1476 kN. This is a bit too much for chosen 400D piles. Taking the width of 11.6 meters into account it is suggested to place piles from left to right as shown in Figure J- 6. In the depth of the lock the piles can be placed 5.66 meters apart. This results in a total of $\frac{85}{5.66} * 3 = 45\ piles$. Each pile needs to take a load equal to $\frac{85*11.6}{45} * 41 = 21.9\ m^2 * 41\frac{kN}{m^2} = 898\ kN$. Therefore the chosen piles meet the requirements. The factor of safety in this case becomes: $FOS = \frac{1235\ kN}{898\ kN} = 1.37$ Because the pile are apart more than 8 times its own diameter group effects can be neglected.

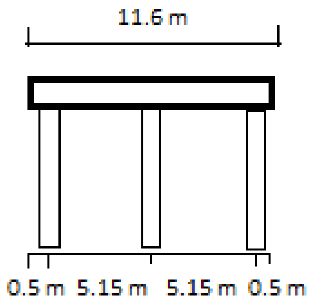


Figure J- 6: Pile distribution small lock

The 'clod criterium' is checked in order to see if the weight between the piles is enough to take the tension forces. This means that the weight of the soil minus the water weight between the piles has to be higher than the total 'floating' force. The weight of the soil is at minimum 15 kN/m^2 at minimum. This would result in a downward force of 5 kN/m^2, the total underestimated downforce between the piles becomes: $F_{soil} = 85\ m * 11.6\ m * 30\ m * 5\ kN = 147900\ kN$ while the total upward forces are 23666 kN (Table J- 17). There is no risk that the clod of the pile would be to light because the factor of safety becomes: $FOS = \frac{147900\ kN}{23666kN} = 6.24$

Bearing

The pressure on the big lock is 85 kN/m^2. The bearing capacity of the chosen pile is 3692 kN. When the piles are placed as explained in previous paragraph, each pile needs to bear 85 kN/m^2 * 21.9 m^2 = 1861 kN. This is far beneath the bearing capacity. The factor of safety in this case becomes: $FOS = \frac{3692\ kN}{1861\ kN} = 1.98$.

Shear forces

The total lateral force in the normative situation on the lock is 9508 kN. There are 120 piles to take this lateral force, leading to a lateral force of $\frac{6706}{45} = 149\frac{kN}{pile}$. This is far below the stated lateral capacity of 1159 kN. The factor of safety in this case becomes: $FOS = \frac{1159\ kN}{149\ kN} = 7.77$.Therefore the foundation meets the ULS requirements.

Settlements and displacements

Although the settlements and displacements are earlier shown not to be normative the displacements in the foundation are calculated. The maximal horizontal and vertical forces that work on a pile are $F_v = \frac{59855}{45} = 1330\ kN$ and $F_h = \frac{1802}{45} = 40\ kN$. This results in a displacements of $u_v = 1330\ kN * 0.0093\ mm/kN = 12\ mm$ and $u_h = 40\ kN * 0.00085 = 0.034\ mm$.

Variant B – Big lock

Tension

The tension on the big lock is 51.3 kN/m^2. The tension capacity of the piles are in a range of 1200 - 1700 kN. When the piles are placed 5 meters apart in both directions each pile needs to bear 51.3 kN/m^2 * 25 m^2 = 1282 kN. This is a bit too much for the smallest piles. Taking the width of 23 meters into account it is suggested to place piles from left to right as shown in Figure J- 5. In the depth of the lock the piles can be placed 5 meters apart. This results in a total of $\frac{140}{5} * 5 = 140\ piles.$ Each pile needs to take a load equal to $\frac{140*23}{140} * 51.3 = 23\ m^2 * 51.3\frac{kN}{m^2} = 1180\ kN.$ Therefore the smallest piles of 400*400 mm shown in Table J- 18 can be used for the foundation. In this case the piles need to be prestressed to 3.1 N/mm^2. The factor of safety in this case becomes: $FOS = \frac{1235\ kN}{1180\ kN} = 1.05$

Because the pile are apart more than 8 times its own diameter group effects can be neglected.

The 'clod criterion' is checked in order to see if the weight between the piles is enough to take the tension forces. This means that the weight of the soil minus the water weight between the piles has to be higher than the total 'floating' force. The weight of the soil has a minimum value of 15 kN/m^2. This would result in a downward force of 5 kN/m^2, the total underestimated downforce between the piles becomes: $F_{soil} = 140\ m * 23\ m * 30\ m * 5\ kN = 483000\ kN$ while the total upward forces are 109947 kN (Table J- 17). There is no risk that the clod of the pile would be to light because the factor of safety in this case becomes: $FOS = \frac{483000\ kN}{109947\ kN} = 4.39$

Bearing

The pressure on the big lock is 66.5 kN/m^2. The bearing capacity of the chosen pile is 3692 kN. When the piles are placed as explained in previous paragraph, each pile needs to bear 66.5 kN/m^2 * 23 m^2 = 1529 kN. This is far beneath the bearing capacity. The factor of safety in this case becomes: $FOS = \frac{3692\ kN}{1529\ kN} = 2.41$

Shear forces

The total lateral force in the normative situation on the lock is 10435 kN. There are 140 piles to take this lateral force, leading to a lateral force of $\frac{10435}{140} = 74.5\frac{kN}{pile}$. This is far below the stated lateral capacity of 1159 kN. The factor of safety in this case becomes: $FOS = \frac{1159\ kN}{74.5\ kN} = 15.55$. Therefore the foundation meets the ULS requirements.

Settlements and displacements

Although the settlements and displacements are earlier shown not to be normative the displacements in the foundation are calculated. The maximal horizontal and vertical forces that work on a pile are $F_v = \frac{154389}{140} = 1103\ kN$ and $F_h = \frac{2837}{140} = 20\ kN$. This results in a displacements of $u_v = 1103\ kN * 0.0093\ mm/kN = 10\ mm$ and $u_h = 20\ kN * 0.00085 = 0.017\ mm$.

Variant B – Pillar

Bearing

In contrast to the other foundations, the foundation of the pillar is not governed by the tension but by the bearing capacity of the piles. The weight of the bearing wall itself has to be added to the total weight of the pillar because it is designed in combination with the foundation. When the pillar is 2 meters wide, 25 meters long and 9.5 meters high the total weight becomes $2 * 25 * 9.5 * 23 = 10925\ kN$. When multiplied by the factors and added to the earlier found weights this leads to total weights presented in Table J- 20.

Force:	Bearing (ULS)			Tension (ULS)			Settlement (SLS)		
	F_v	F_m	F_t	F_v	F_m	F_t	F_v	F_m	F_t
	[kN/m²]			[kN/m²]			[kN/m²]		
Forces on pillar	29.04	43.25	72.3	-54.0	-43.25	-97.25	22.3	10.1	32.48
Own weight	284.0	-	284.0	196.6	-	196.6	218.5	-	218.5
Total	-	-	356.3	-	-	99.35	-	-	251.0

Table J- 20: Recalculated forces for the pillar

The pressure on the big lock is 356.3 kN/m^2. When placed in the middle of the pillar there could and placed 5 meters apart in the length of the pillar-wall there are 6 piles. In this situation each pile needs to take 1/6th of the area resulting in 50/6 = 8.32 m^2. The force on each pile becomes $8.32\ m^2 * 356.3 \frac{kN}{m^2} = 2969\ kN$. The bearing capacity of the piles earlier used is 3692 kN. The factor of safety in this case becomes: $FOS = \frac{3692\ kN}{2969\ kN} = 1.24$. Therefore these 6 piles are enough to take the forces on the wall. In a detailed design these pile can be placed more to the outside of the wall in order to reduce the forces even more.

Tension

The calculated tension in Table J- 20 shows that there is no tension force possible in the pillar and therefore no tension calculation is needed.

Shear forces

The total lateral force in the normative situation on the lock is 4914 kN. There are 6 piles to take this lateral force, leading to a lateral force of $\frac{4914}{6} = 819 \frac{kN}{pile}$. This is just below the stated lateral capacity of 1159 kN. The factor of safety in this case becomes: $FOS = \frac{1159\ kN}{819\ kN} = 1.41$. Therefore the foundation meets the ULS requirements. This shear forces might become normative if a more detailed design is made.

Settlements and displacements

Although it is shown that the settlements and displacements of before are not normative, the displacements in the foundation are calculated. The maximal horizontal and vertical forces that work on a pile are

$F_v = \frac{F_{own}+F_{ext}}{piles} = \frac{10925+1117}{6} = 2007\ kN$ and $F_h = \frac{1320}{6} = 220\ kN$. This results in a displacements of $u_v = 2007\ kN * 0.0093\ mm/kN = 18\ mm$ and $u_h = 220\ kN * 0.00085 = 0.18\ mm$.

Conclusion

Pile plan

The pile plans for both variants are presented below.

Variant A

Part	Length	Width	Pile type	Prestress	No of piles	Center to center distance width	Center to center distance length	Lowest FOS	Displacements	
									Vertical	Lateral
	[m]	[m]	[-]	[N/mm²]	[-]	[m]	[m]	[-]	[mm]	[mm]
Big lock	120	23	400*400	3.1	120	5.5	5	1.05	10	0.018
Small lock	85	11.6	400*400	3.1	45	5.15	5.66	1.37	12	0.034

Table J- 21: Pile plan variant A

The total number of piles is 165.

Variant B

Part	Length	Width	Pile type	Prestress	No of piles	Center to center distance width	Center to center distance length	Lowest FOS	Displacements	
									Vertical	Lateral
	[m]	[m]	[-]	[N/mm²]	[-]	[m]	[m]	[-]	[mm]	[mm]
Big lock	140	23	400*400	3.1	140	5.5	5	1.05	10	0.017
Pillar	25	2	400*400	3.1	6	-	5	1.24	18	0.18

Table J- 22: Pile plan variant B

The total number of piles is 146.
This pile plan meets the ULS and SLS requirements for tension, stress and moments and is checked on the 'clod criterion'.

Comments

Some comments have to be made about this design:

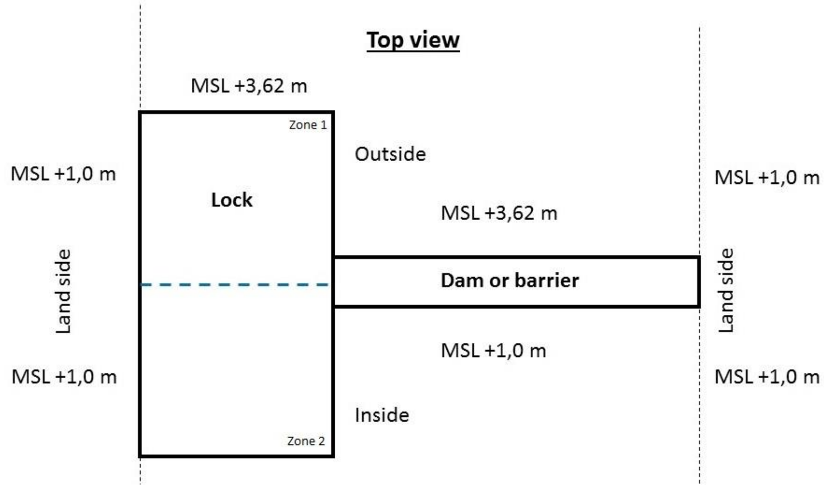
- The pile capacities are found by a literature study, but without experience with the local soil. Therefore the values taken in the calculations are always taken for the worst case scenario. This may lead to over-dimensioning of the foundation. On the other hand the uncertainty about the applicability of the literature may cause unexpected risks.
- The floor thickness is designed based on the worst case situation in navigation circumstances, but not with the empty lock chamber due to maintenance in mind. Therefore there a very high tension forces in a maintenance situation. If the floor is designed thicker these forces would be lower and a more balanced design could be found.
- In the future it is recommended to use CPT instead of SPT, this is also recommended by Shariatmadari N. et al. (2008).
- The piles chosen are driven prestressed concrete piles. It is assumed that the piles can be driven in this area, when this is not possible, screwed piles can be used. It must be taken into account that for most of the soil capacities of the pile the capacities of a screwed pile should be taken 0.9 for that of the driven piles. Also it is not possible to prestress screwed piles and therefore the reinforcement percentage should be increased.

Appendix K – Lock floor design

The function of the floor is described in the main report as are the dimensions of the floor and the aim of this paragraph. As a guideline in this paragraph the Hydraulic Structures manual ((Molenaar & Voorendt, 2016)) chapters 35, 40 and 46 are used. Also the Hydraulic Structures Lock manual ((CIE3330 Hydraulic Structures - locks, 2011)) chapter 4 is used.

Big Lock

This lock is designed in both alternatives; it has a large surface area that is divided into two equal zones. The weight difference of the vertical lift gate and mitre gate is neglectable as can be seen later in this paragraph, so there is one calculation conducted for both big locks. The length of the floor is 140m and the width is 23 meter. The maximum water level on the inside can be MSL +1,0 m, while on the outside the maximum water level can be MSL +3,62 m. It is assumed that in future the dikes are heightened to +3,62 so the water will not flow on the landside adjacent of the lock. The blue dotted line indicates the border of the two equal zones. The maximum water level adjacent to the lock is needed to calculate the buoyancy force, this water level is different for the two zones of the lock. The zones are simplified to a rectangle shape as can be seen **Appendix K - figure 1**.



Appendix K - figure 1: The pressure zones in the floor of the lock

Upward pressure

A water level of MSL+3.62 m, leads to a water height of 7.12 meter because the floor is located at MSL - 3.5m. A water level of MSL +1.0m on the Ho Chi Minh City side leads to floods, so this is the maximum water level possible and is equal to a water height of 4.5m. The upward pressure of the two zones is:

$P_1=\rho*g*h_1=1000*9.81*7.12 = 69847$ Pa = 70KN/m
$p_2=\rho*g*h_2=1000*9.81*4.5 = 44145$ Pa = 44KN/m

Thickness of the floor

The buoyancy force of the lock is for half the surface area 44KN/m and for the other half 70KN/m. This buoyancy force has to be lower than the vertical downward forces of the construction. The vertical forces downward of the construction are listed below. The maximum water level difference for inside the lock and outside the lock is taken into account so the construction will never be lifted up by the water pressure when $\sum V=0$ or $\sum V$ = downward in this calculation.
The weights are assumed to be equally spread throughout the surface area as is the upward pressure. This could be assumed because the two walls and four gates are constructed symmetrically so there is no moment that causes a turning to occur.

- Weight floor: 2300*9.81*x = 23 * thickness of the floor KN/m
- Weight water: 1000*9.81*3.5 = 34 KN/m
- Weight walls: 2(140 m * 1m * 7.5m) = 2100m^3 concrete, 9.45m^3 steel
  = (2100*2400) + (9.45*7800) = 5113710 kg = 16 KN/m
  The weight of the towers on the wall is negligible in comparison with the weight of the wall.
- Weight gate: 2*454 KN +2*161KN= 0.4 KN/m

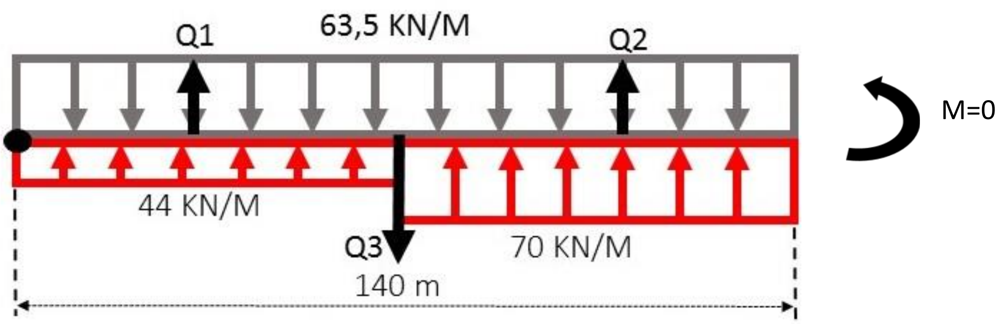
All these vertical downward forces together are summarised as 50.4KN/m+23KN/m*thickness of the floor and displayed in Appendix K - figure 2 as Q3. The two buoyancy forces are displayed as Q1 and Q2.

For the piles it is easier to resist a compressive force than a tensile force and it is difficult for the piles to resist a moment. Therefore two aspects are important to create in the floor of the lock:

- ∑V=0 or ∑V = downward.
- ∑M=0

When ∑V=0 than ∑M≠0, so the thickness of the floor is calculated in such a way that ∑M=0 and that no buoyancy occurs. A moment turning anti clockwise as shown in Appendix K - figure 2 is likely to occur due to a higher buoyancy force on the right. The pivot point is located on the left of the lock as shown in Appendix K - figure 2 with a black dot. The arms of Q1,Q2 and Q3 to the pivot point are known so the thickness of the floor can be calculated to create a ∑M=0.

Q1*70*35+Q2*70*105 = Q3*140*70 → 44*70*35+70*105*70 = Q3*140*70
The value of Q3 has to be equal to 63.5 KN/m, so 63.5=50.4+23*thickness of the floor→ the thickness of the floor is equal to 0.57 meter.



Appendix K - figure 2: Forces on the floor of the lock

A floor thickness of 0.57 meter does result in a ∑V downward of 63.5*140-(44*70+70*70)=6.5KN/m. This vertical force downward can be resisted by the piles, this occurs at a maximum water level difference for inside and outside the lock. The ∑V will become higher when the water level outside the lock is lower than the maximum water level.

Bending stress in concrete floor

Because the floor is constructed without reinforcement, the tensile stresses have to be lower than the tensile strength of the concrete. According the hydraulic structures manual chapter 35, the design value for concrete tensile strength is computed as follows:

$$f_{ctd} = \frac{\alpha_{ct} * f_{ctk,0.005}}{y_c} = \frac{1*2.2}{1.5} = 1.47\ MPa$$

The tensile stresses can be computed according the hydraulic structures manual chapter 40 and 46 as follows:

$$\sigma = \frac{M}{W} = \sigma = \frac{\frac{1}{10}*q*l^2}{\frac{1}{6}*b*h^2}$$

The value of the load in the limit state of collapse is equal to:

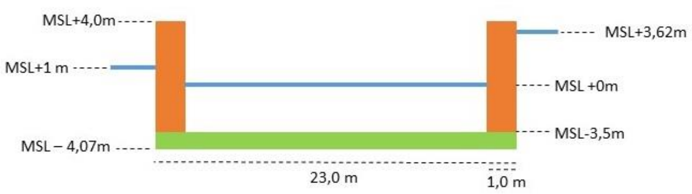
Q=y*(Q3-floor thickness*ρ(compact concrete)= 1.5*(63.5-0.57*23)= 76 KN/m

It is calculated per linear meter, so b=1m, the floor thickness is 0.57 meter so h=0.57m. The length of the floor is 140 meter, but the concrete floor can be constructed in several segments according to the (CIE3330 Hydraulic Structures - locks, 2011, p. chapter 4). The length of one segment determines the moment that occurs in such a segment, the tensile stress has to be lower than the tensile strength so the maximum length of one segment can be determined as follows:

$$1.47 * 1000 = \frac{\frac{1}{10}*76*l^2}{\frac{1}{6}*1*0.57^2} \rightarrow \text{l= 59 meter.}$$

This length for the segments leads to a tensile stress that is equal to the tensile strength, while the tensile stress should be lower than the tensile strength for safety to prevent bending.
This means that the length of the segments can be decreased although in practice the moment will be lower, because there is no reinforcement in the concrete which lowers the stiffness. As is stated in the Hydraulic Structures Lock manual (chapter 4) a typical length is 25 meter for the segments. In paragraph 6.3 and 7.3 it is proven that the minimum length between the piles is smaller than the 25 meter of the segments, so no bending stresses in the floor will occur.

A side view of the lock is provided in Appendix K - figure 3.



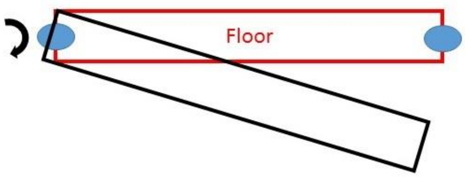
Appendix K - figure 3: Side view of the lock

Small lock

The dimensions of the floor of the small lock are 85m x 11.6m. The upward water pressure is the same as for the big lock, so p_1=70KN/m and p_2=44KN/m. Again it is aimed to have ∑M=0 so the same method is applied and the only unknown is the vertical force downward of the construction. To achieve that ∑M=0, the value of Q3 is again 63.5 KN/m. The vertical force downward consists of:

- Weight floor: 2300*9.81*x = 23 * x KN/m
- Weight water: 1000*9.81*3.5 = 34 KN/m
- Weight walls: 2(85 m * 0.8m * 7.5m) = 1275m^3 concrete, 5.5m^3 steel
  = (1275*2400) + (5.5*7800) = 31 KN/m
- Weight gate: 2*344 KN +2*71 KN= 0.3 KN/m
- TOTAL 65.3+23*thickness floor

The total weight of the water and walls makes clear that the floor is not needed to achieve a higher vertical force downward than upward. Of course a floor is constructed so locking can take place and the floor can transfer the forces to the piles. A floor thickness of 0.40 meter is assumed, so the floor thickness is smaller than for the big lock.



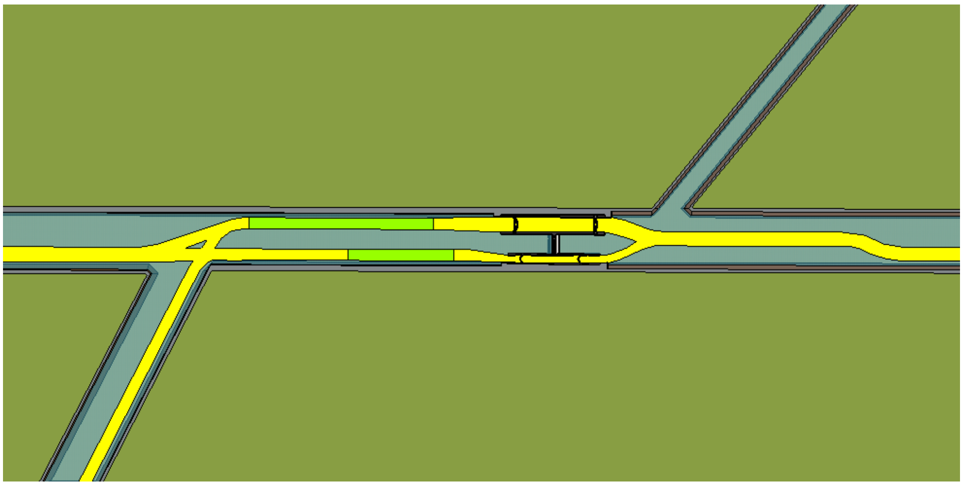
Appendix K - figure 4: Possible pivot points

The higher vertical load downward than upward can be resisted by the piles and is ((23*0.4) + 34 + 31 + 0.3) – 63.5= 11KN/m. The ∑V will become higher when the water level outside the lock is lower than the maximum water level. Because the downward force is 11KN/m higher than the upward force, there is a moment in the concrete floor which turns clockwise. There are only two possible pivot points on the floor, as is shown in Appendix K - figure 4. The two black lines show that only a compressive force occurs and that no tensile forces appear, due to the created moment. The compressive forces can be resisted by the piles so it is safe to assume a floor of 0.4m.

The tensile strength is the same for the small lock and the big lock, the same method is applied as for the big lock to determine the length of the segments. When the tensile stress equals the tensile strength, the length of the segments has to be 74 meter. As is stated before, the distance between the piles will be a lot shorter so the tensile stress will be a lot smaller than the tensile strength.

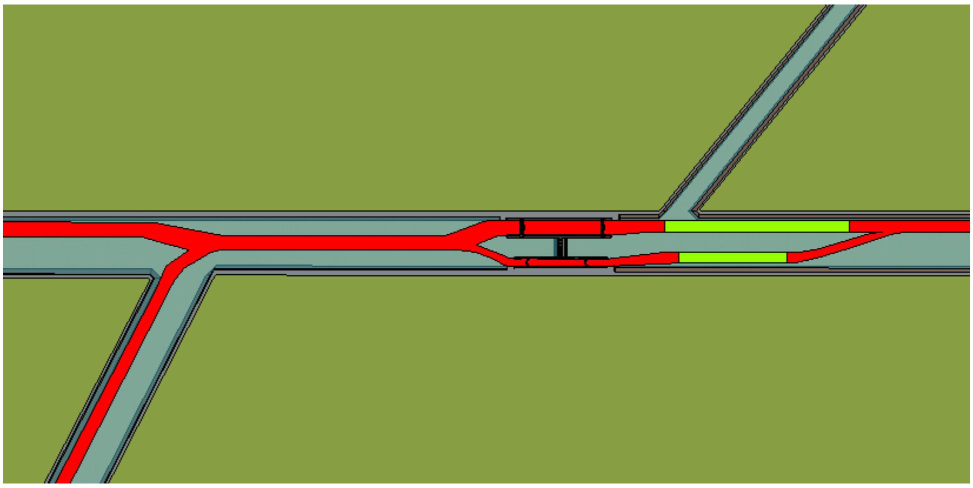
Appendix L – Ship paths near the structure at Phu Dinh

The following figures illustrate the paths of the boats in the main channels around the Phu Dinh project. The first picture displays the paths of the vessels navigating into Ho Chi Minh City in yellow, the green areas are the waiting areas for the two locks



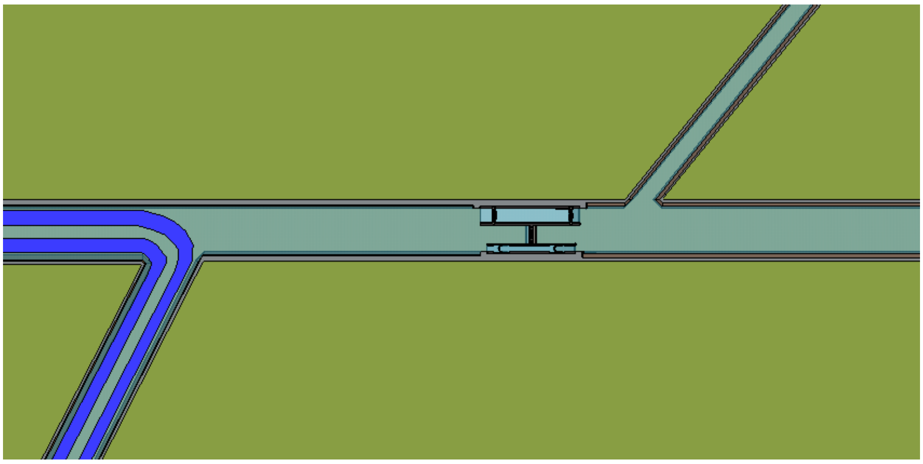
Appendix L - figure 1: Paths of ships entering HCMC.

The paths of the vessels leaving the city are illustrated in red in the next picture where the waiting areas are again green. Both of the paths have been developed in such a way that they have minimum interference with one another.



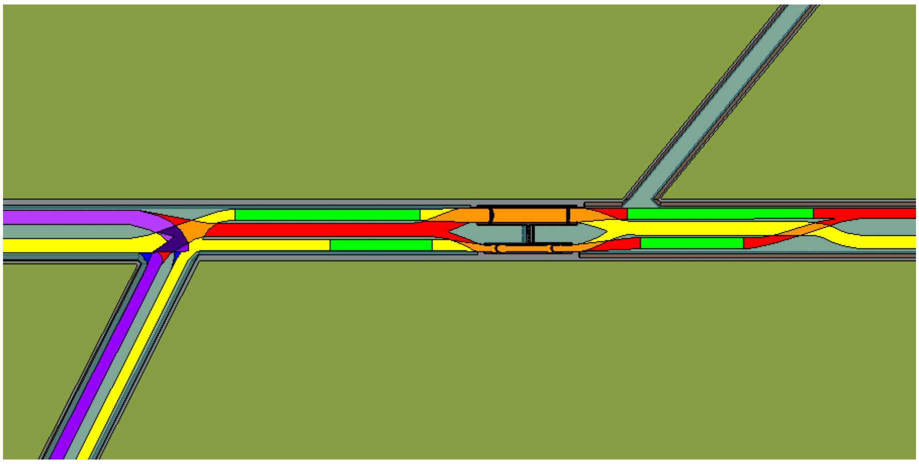
Appendix L - figure 2: Paths of ships leaving HCMC

Another possible navigation path may be of vessels not entering the city but moving from south to west or the other way around as illustrated by the blue paths.



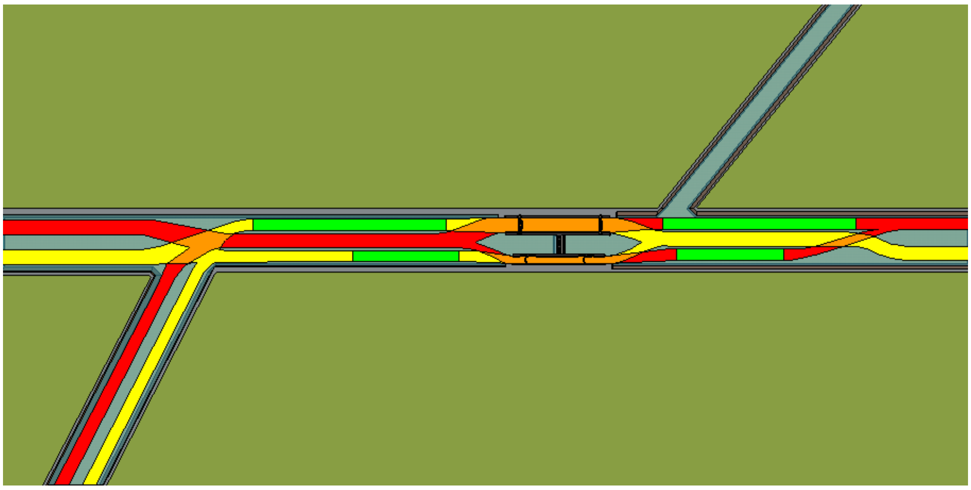
Appendix L - figure 3: Ships not navigating the Phu Dinh channel.

Combining all of the above paths leads to the following navigational overview. The same color has been applied for the paths here except for the areas where they interfere with one another. In this sense the interference between the yellow and the red line has been made orange for instance.



Appendix L - figure 4: Interchanging ship paths.

However if we neglect the vessels not going through the barrier we come up with the following simplified overview:



Appendix L - figure 5: All ships passing the barrier.

Appendix M – Forces on a sheet pile

This appendix is a screenshot of the excel file made to calculate the forces working on a sheet pile.

Sheet input								
Soil active	Water active	Soil passive	Water passive	Top sheet	Bottom sheet	Length sheet		Stempelhoogte
1.4	1.4	-4.1	1	4	-25.5	29.5		-4.5

Soil layers input												
Layer	From	To	Layer thickne:	Wet weight	Dry weight	theta	cohesion	K_a	K_p	γ/γw	K_a'	K_p'
	[m MSL]	[m MSL]	[m]	[kN/m^2]	[kN/m^2]	[degrees]	[kN/m^2]	[-]	[-]	[-]	[-]	[-]
1	2	-17.96	19.96	15	5	5	7.6	0.8397	1.191	0.6666667	0.9466	1.0565
2	-17.96	-23.06	5.1	18.5	8.5	15	31.3	0.5888	1.6984	0.5405405	0.8111	1.2329
3	-23.06	-35.46	12.4	19.6	9.6	29	4.2	0.347	2.8821	0.5102041	0.6802	1.4703
4a	-35.46	-39.06	3.6	20.2	10.2	32	4.5	0.3073	3.2546	0.4950495	0.6502	1.538
4b	-39.06	-50	10.94	20.2	10.2	32	4.1	0.3073	3.2546	0.4950495	0.6502	1.538
	Left	Right										
Diff top	0.6	6.1										

Calculation forces active side																	
			Active														
From	To	Layer tickness	avg pos	pos tov bottom	Ptop	Pbottom	Pavg	SumP	Force - c	C		P4	P3		M	Water	
[m MSL]	[m MSL]	[m]	[m MSL]	[m]	[kN/m^2]	[kN/m^2]	[kN/m^2]	[kN/m]	[kN/m]	[kN/m]	[kN/m]	[kN/m]	[kN/m]		[kNm]		
1.4	-17.96	19.36	-8.28	17.22	0	96.8	48.4	937.02	786.78	269.65061	517.13	-269.7	786.78		6366.4	From:	1.4
-17.96	-23.06	5.1	-20.51	4.99	96.8	140.15	118.475	604.22	355.76	244.97681	110.78	45.697	65.086		497.49	To:	-25.5
-23.06	-25.5	2.44	-24.28	1.22	140.15	163.574	151.862	370.54	128.57	12.073067	116.5	106.58	9.9156		138.09	Length	26.9
-25.5	-25.5	0	-25.5	0	163.574	163.574	163.574	0	0	0	0	0	0		0	Force	3618.1
-25.5	-25.5	0	-25.5	0	163.574	163.574	163.574	0	0	0	0	0	0		0	Place	8.9667
	Active side								1271.1	526.70049	744.41		744.41		7001.9	M	32442
Sum F [kN]	4362.46321	Sum M [kNm]	39443.7788														

Calculation forces passive side															
			Passive												
From	To	Layer tickness	avg pos	pos bottom	Ptop	Pbottom	Pavg	SumP	Force - c	P4	P3		M	Water	
[m MSL]	[m MSL]	[m]	[m MSL]	[m]	[kN/m^2]	[kN/m^2]	[kN/m^2]	[kN/m]	[kN/m]	[kN/m]	[kN/m]				
-4.1	-17.96	13.86	-11.03	14.47	0	69.3	34.65	480.25	571.95	0	571.95		6955	From:	1
-17.96	-23.06	5.1	-20.51	4.99	69.3	112.65	90.975	463.97	788.01	600.26423	187.74		3772.6	To:	-25.5
-23.06	-25.5	2.44	-24.28	1.22	112.65	136.074	124.362	303.44	874.54	792.18032	82.361		1033.4	Length	26.5
-25.5	-25.5	0	-25.5	0	136.074	136.074	136.074	0	0	0	0		0	Force	3511.3
-25.5	-25.5	0	-25.5	0	136.074	136.074	136.074	0	0	0	0		0	Place	8.8333
	Passive side								2234.5		2234.5		11761	M	31016
Sum F [kN]	5745.75556	Sum M [kNm]	42777.0394												

Results			
	Difference without struts		
Forces: [kN]	1383	Moments [kNm]	3333
FOS	1.32	FOS	1.08

Other forces in case of struts					
Stempels					
F stempel	415.848861	[kN/m]			
M passive:	51509.8655	[kNm]			
F passive	6161.60442	[kN]			
Passive	Active	Active	Passive	Water:	
[m MSL]	[m MSL]	[kNm]	[kNm]		
6.53	3.78	4493.455436	5056.07853	Place active	12.0333333
16.01	16.01	1828.97173	12775.6107	Place passive	12.1666667
19.78	19.78	2308.319831	17331.9297	Moment active	43537.2017
21	21	0	0	Moment passive	42720.2083
21	21	0	0	M active	M Passive
		8630.746997	35163.6189	52167.94866	77883.8272
					1.49
	1.41		1.31		

Results			
	Difference with struts		
Forces: [kN]	1799	Moments [kNm]	12066
FOS	1.41	FOS	1.31

Appendix N – Probability of failure

1) Fault tree

The three main possible errors are a series system, if one of these errors occurs the entire system fails and Ho Chi Minh City will be flooded. This is displayed in the fault tree by an 'or' gate. The probability of a failure mechanism consist of situations to occur. For each failure mechanism it is determined whether the situations are dependent or independent and if the situations are series or parallel. If those situations are independent and parallel, the probability of the failure mechanism to occur is the sum of all probabilities, like the failure mechanism overflow as shown in figure 1 below. The probabilities are calculated with the methods shown in figure 1 below.

system	gate	operator	components		
			mutually exclusive	independent	fully dependent
series	OR	$\cup$	$\sum_{i=1}^{n} P_i$ (upper bound)	$1-\prod_{i=1}^{n}(1-P_i)$	$\max\{P_i\}$ (lower bound)
parallel	AND	$\cap$	0 (lower bound)	$\prod_{i=1}^{n} P_i$	$\min\{P_i\}$ (upper bound)

Appendix N - Figure 1: fault tree method derived from Jonkman et al. (2015)

2) Bed protection locks emptying chambers

The bed protection of the lock heads is calculated and described using formulas from Rajaratnam from the book and Delft Hydraulics (1983).

The maximum velocity at maximum discharge by maximum head level difference through the opening sluices in the gate is:

$V = \sqrt{2gz} = \sqrt{2 * 9.81 * 3} = 7.67\ m/s$ According to groenveld, verheij

z = maximum head level difference

This velocity is used to calculate the velocity of the jet at the bottom

$$u_m = \frac{6.3u_0}{\left(\frac{x}{D}\right)} \qquad b = 0.1x \qquad u = u_m e^{\left(-0.693\left(\frac{R}{b}\right)^2\right)} \qquad \text{(Circular jet)}$$

u_m = velocity in the center of the jet (m/s)
u_0 = maximum outflow velocity at sluice openings(m/s) = 7.67
x = distance from orifice to bed protection (m) = 11
D = radius of the orifice (m) = 0.9
u = velocity at a specific location and distance from the orifice (m/s)
R = distance from axis of jet (m) = variable

Circular jets
Circular jets instead of plane jets are chosen because the velocity is reduced faster in the situation of circular jets compared with plane jets. The difference can be seen when for both formulas a distance of 11 m from the orifice to the bed is used:

Circular jet

$$u_m = \frac{6.3u_0}{\left(\frac{x}{D}\right)} = \frac{6.3*7.67}{\left(\frac{11}{0.9}\right)} = 3.95\ m/s$$

Plane jet

$$u_m = \frac{3.5u_0}{\sqrt{\frac{x}{B}}} = \frac{3.5*7.67}{\sqrt{\frac{11}{0.8}}} = 7.23\ m/s$$

Each set of doors has got filling openings with an area of 5 m^2. This means that in each door an opening of 2.5 m^2 is present which produces a jet. This will be a circular jet with a radius of 0.9 m in each door.

The distance x is the distance from the openings to the beginning of the bed protection. This distance is approximately 11 m, because the doors are 11 m in width and when they are open, they reach until the end of the concrete lock chamber and concrete floor. Only the first formula for u_m is used because the highest velocity in the center of the jet is leading for the bed protection.

$$u_m = \frac{6.3u_0}{\left(\frac{x}{D}\right)} = \frac{6.3*7.67}{\left(\frac{11}{0.9}\right)} = 3.95\ m/s$$

Turbulence in the jet is not yet included in the velocity. Using a relative turbulence intensity (r) of 20% (Delft Hydraulics (1983)) the final maximum velocity becomes: $v_{max} = (1+3r)v = (1+3*0.2)*3.95 = 6.32\ m/s$.

This velocity is used to determine the stone dimension needed for the bed protection with an Izbash-type formula. Izbash is used because the velocity does not depend on the equilibrium between flow force and bed friction in the case of water jets. Also big stones are considered in this situation. The following formula is used to determine the stone dimensions:

$u_c = 1.2\sqrt{2\Delta g d}$ or $\Delta d = 0.7\frac{u_c^2}{2g} = 0.7*\frac{6.32^2}{2*9.81} => d = \frac{1.43}{1.65} = 0.86\ m$ (assumed to be d_{n50})

Δ = specific density clay = 1.65 [-]
d = stone diameter [m]

- The diameter of the stones that should be applied to resist the emptying of the chamber at a maximum head level difference of 3 m (outside level > inside level) is: 0.86 m.
- The diameter of the stones that should be applied to resist the emptying of the chamber at a maximum head level difference of 1.4 m (inside level < outside level) is: 0.40 m.
  $\Delta d = 0.7\frac{u_c^2}{2g} = 0.7*\frac{4.32^2}{2*9.81} => d = \frac{0.67}{1.65} = 0.40\ m$

3) Bed protection lock propeller wash

The minimum stone size needed for the bed protection can be calculated with formulas from the book. The outflow velocity u_0 of the propeller of a ship can be calculated as follows:

$u_0 = 1.15\left(\frac{P/2}{\rho d^2}\right)^{0.33} = 1.15\left(\frac{640000/2}{1000*(0.7*2.0)^2}\right)^{0.33} = 6.18\ m/s$ In which

P[1] = power of the engine (W) (see below)
ρ = density of water (kg/m^3)
d = diameter of propeller: 70% of ship's unloaded draught (m)

The power of the engine of the largest vessel that will use the waiting area can be estimated as follows:

[1] It is assumed, as used in Delft Hydraulics (1983), that the ship uses half of its power during entering or leaving the lock

$$P = 0.66L * (2D + B) = 0.66 * 66 * ((2 * 2.5) + 9.7) = 640\ kW$$

This power is used in the formula for the outflow velocity of the propeller. When the location of the maximum or the distribution of the velocity on the bottom is not important, the maximum velocity on the bottom $u_{b,max}$ can be determined as follows:

$u_{b,max} = 0.3u_0 * \frac{d}{z_b} * \sqrt{n} = 0.3 * 6.18 * \frac{1.4}{1.23} * \sqrt{1} = 2.11\ m/s$ In which

z_b = vertical distance between the propeller axis and the bottom (m), it is assumed that the propeller is attached on 1/3th of the ship's hull. Minimum water level = -0.6 + 3.5 = 2.9 m. Draught is 2.5 m. Half of the propeller = 1.4 m → ((1/3)*2.5) + 0.4 = 1.23 m. n = number of propellers = 1

The formula is an Izbash-type relation since the transition from velocity to shear stress is very difficult and the propeller wash is a waterjet:

$\Delta d_{n50} = 0.47 \frac{(u_c(1+3r))^2}{2gK_s} = 0.47 \frac{(2.11(1+3*0.45))^2}{2*9.81*1} = 0.59$ In which

Δ = specific density (-) = 1.65
d_{n50} = nominal diameter (m) ≈ 0.84 d_{50}
g = gravitational constant = 9.81 m/s^2
K_s = slope correction factor = 1 for bottom protection

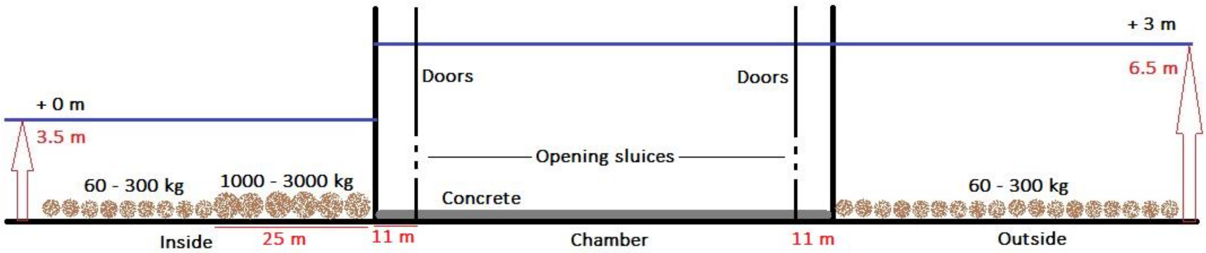
So $\Delta d_{n50} = 0.59$ which means that $d_{n50} = \frac{0.59}{1.65} = 0.36\ m$

4) Location of bed protection

The governing velocity, and therefore the corresponding bed protection, is coming from the jet when emptying the lock chamber at the highest head level difference (3 m). However, after some meters this jet will lose power, in such a way that the propeller wash of the ships in the proximity of the lock becomes governing. The length, at which the bed protection should be applied, is somewhat a matter of experience. Delft Hydraulics (1983) uses a practical length of 20 – 25 m. A simple calculation can be used to estimate the length of the bed protection needed for the emptying process of the chamber. When using 21 m for x in $u_m = \frac{6.3u_0}{\left(\frac{x}{D}\right)}$ (formula for emptying jet), a velocity of 2.07 m/s is the result. In this situation, the maximum velocity of the emptying jet is lower than the propeller wash from the ships which means the propeller wash is becoming normative. It is therefore advised to use 25 meters of bed protection in front of the doors to resist the emptying jet.

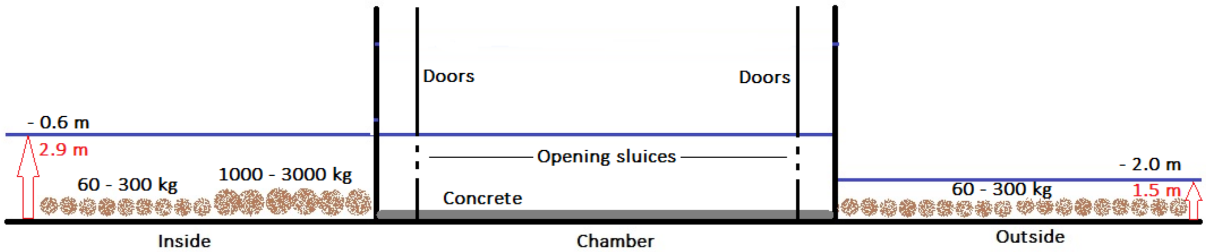
In figure 2 and 3 below, the governing situations and the needed bed protections can be found. Some notes and recommendations with respect to the bed protection:

- Figure 2: Shows the governing situation (outside level > inside level) with a head level difference of 3 m. Stones of 1000 – 3000 kg should resist the forces of the water jet due to emptying of the lock chamber over a length of 25 meter. The other stone dimension of 60 – 300 kg is used to resist the propeller wash of the ships.



Appendix N - Figure 2: Governing head level difference (outside level > inside level) and bed protection lay-out

- Figure 3 : the second governing situation (outside level < inside level) with a head level difference of 1.4 m. Still, stones of 1000 – 3000 kg should resist the forces of the water jet due to emptying.
- The stones to resist the water jet due to empyting of the lock chamber on the other side (right side in the figure) should be in the order of 0.40 m. This is almost the same size as needed for the bed protection due to the propeller wash of ships. Therefore it is advised to use the same stone class over the full length of the bed protection at the right side.
- The water depth at the right side is only 1.5 m which implies that the propeller wash has a stronger impact on the bottom. However, because only small ships are able to navigate with a depth of 1.5 m, the propeller wash is not that strong that a larger stone dimension is needed.



Appendix N - Figure 3: Governing head level difference (outside level < inside level) and bed protection lay-out

5) Scour depth formula

Explanation of the variables in the scour depth formula

$h_s(t) = \frac{(\alpha\bar{u}-\bar{u}_c)^{1.7} * h_o{}^{0,2}}{10\Delta^{0,7}} t^{0,4}$ (Eq. 4.13)

$h_s(t)$ = the maximum depth in the scour hole as a function of time
α = among others, turbulence and increases the effective velocity
$\bar{u}$= the vertically averaged velocity at the end of the protection
$\bar{u}_c$= critical, depth-averaged velocity
h_o= original waterdepth
Δ= relative density
t= time in hours
Derivation of the values for the several variables in the scour depth formula:

H_o = 5,5 meter. (M.S.L. -5.5 m until M.S.l. +0 m.)

U top sill = The average flow velocity at Phu Dinh is 1.8 m/s as explained in chapter 2. This average is used because the flow distribution is unknown and all other variables in this formula s are taken as a maximum, otherwise there would be a to high safety margin. There is a horizontal constriction at the barrier and dam, the width decreases from 66 meter to 40 meter. This increases the flow velocity with: $\frac{66}{40}$=1.65. This gives a flow velocity on the top of the sill of 2,97 m/s.

Ū = (eq 3.15) $= \frac{u\ top\ sill*(h_0 - height\ of\ sill)}{h_0} = \frac{2.97*(5,5-2)}{5,5}$ = 1,89 m/s

Dn50= The top layer of the bed is made out of soft, saturated clay. The grainsize of clay is smaller than $2*10^{-6}$ m. The dn_{50} in the river is higher than the grainsize of the bed material, so the dn_{50} is assumed to be $2*10^{-5}$ m.

K_r= Accordinng Breusers (Delft Hydraulics 1969) 7 transport stages were discerned, to develop a scour depth, at least number 5 must occur: frequent movement at all locations. In figure 3-3 of the 'introduction to bed bank and shore' book it is displayed that this fifth stage equals a shields parameter value of 0.05. In figure 3-6 in the same book it can be interpolated that a shield parameter of 0.05 leads to a k_r value of $9*dn_{50}$.

C=(eq 2.10)= 18 $\log\frac{12\ R}{k_r}$ = C= 18 $\log\frac{12*4,3}{9*0,00002}$ =98 √m/s. With R= $\frac{b*h}{b+2h} = \frac{40*5,5}{40+11}$= 4,3.
This Chezy value seems quite high, this is because the dn_{50} has a low value. Though, the sensitivity of the dn_{50} is low, when multiplying the dn_{50} by a factor of 10, the Chezy value decreaes by a factor of 1.2. When dividing the Chezy value by a factor of 2, which is still a reliable value for Chezy, the scour depth increases by a factor of only 1.03. So although the Chezy value is uncertain, the sensitivity for the final scour depth is low.

$\bar{u}_c$=(paragraph 3.2.4)= C*$\sqrt{d_{n50} * \psi_c * \Delta}$ = used for a non-cohesive soil. The porosity of the clay soil is equal to 67% as can be seen in chapter 2. According to Ven Te Chow (1959) a porosity of 67% in the clay results in a critical velocity of 0.15 m/s.

Δ = $\frac{\rho_s - \rho_w}{\rho_w} = \frac{2490-1000}{1000}$=1,5. The density of saturated, clean, weak clay is 15KN/m^3, as can be seen in table 31-4 of the Hydraulic structure manual (TU Delft, 2016). This is equal to 1500Kg/m^3, this consist of:

- 67% water, which weights in total 670kg.
- 33% clay particles, which weight in total 1500-670=830 kg. So 100% of clay particles weights 2490 kg. This provides a ρ_s of 2490 kg/m^3.

t= 15 hours, this is the time that the barrier is open for a maximum period. It occurs in winter time because there is less rainfall in Ho Chi Minh City.

α= 1.5+5*r_o = 2.2, with r_o= 0.144. $r_{o}= \sqrt{0.5\ k_o[1-\frac{D}{h}]^{-2} * [\frac{X}{\lambda}+1]^{-1.08} + 1.45\frac{g}{C^2}}$ =
$\sqrt{0.5 * 0,045[1-\frac{2}{6,5}]^{-2} * [\frac{(55-6*2)}{6,67*5,5}+1]^{-1.08} + 1.45\frac{9,81}{98^2}}$

The α formula of Hoffmans/Booij 1993 is used to determine α. Hoffmans (1992) includes the roughness of the bed, this value is uncertain so using this formula would make the answer less reliable. Though there is a smooth bed present with high velocities, on the other hand more turbulence will be present which decreases the velocity. The α formula of Trinh (1993) is not used because it includes the horizontal constriction, this is already taken into account at the flow velocity above the sill.

6) Scour formula Van (1998)

$$Hm = \left(1 + \left(\frac{L_{gc}}{H_h}\right)^{-0.33} * \frac{U}{U_{kx}} - 1\right) * H_h$$

In which:

Hm Maximum scour depth (m)

L_{gc} the length of bottom protection = m*Hm, in which m is the slope (1:3), so m =3.

H_h Flow depth in downstream section of the canal (5,5m)

U Average flow velocity (1.8 m/s)

U_{kx} Critical flow velocity (erode velocity) depending on soil condition (0.35 m/s)

According to Trinh Cong Van, for soil condition in the Mekong delta:

$$U_{kx} = (1.94 + 0.581 \log H_h).\ C^{0.39}$$

In which: C is shear stress of cohesion soil (0.0076 N/m2)

7) Scour depth in Excel

The variable of the scour depth is on both sides of the equation, using the iterative formula function of Excel, Hm is computed after 1000 iterations to be 11.64 meter.

8) Prob2B

It is tried to create a limit state function in the software Prob2B and to calculate the probability of failure for a bed protection length of 41 meter. It worked out that it is impossible to rewrite the formula of Van(1998) in such a way that the scour depth is only on the left side of the equation.

9) Bed protection

The assumed Chezy value of the bed protection is 50 √m/s. The dn_{50} is determined with the following formula:

$$dn_{50} = \frac{U_c{}^2}{\Delta * \psi * C^2}$$

In which:

U_c= Critical depth averaged velocity in the river, 1.8m/s.
Ψ = shields parameter, occasional movement at some locations is undesired for the bed protection so the value is determined to be 0.03.
Δ= stones of the bed protection will have a relative density of 1.65.

The iterative values are:
C=50 √m/s → dn_{50}=0.026 m → C=54√m/s
C=54√m/s → dn_{50}=0.022 m → C= 55.8√m/s
C=55.8√m/s → dn_{50}=0.021 m → C=56.3 √m/s
C=56.3√m/s → dn_{50}=0.021 m→C=56.4 √m/s

The dn_{50} is 0.021 meter, there has to be included a velocity turbulence factor, indicating a load deviating from uniform flow. There is a vertical constriction of the sill, this is a K_v value of 1.4 has can be seen in figure 3-13 in the book. The horizontal constriction leads to a K_v value of 1.5, so the total Kv value is: 1.5*1.5=2.25. To achieve the dn_{50} of the bed protection the calculated dn_{50} is multiplied by K_v^2, this leads to: 2.25^2 * 0.021 = 0.011 meter.

In appendix A of the book a table is provided that relates the dn_{50} to the stone class range. The stones on the inside(HCMC) of the barrier and dam should be 90-250 mm.