Technische Universiteit Delft

Iv-Infra

The Conceptual Design of the Bolivar Roads Navigational Surge Barrier

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Professional Doctorate of Engineering (PDEng)

in

Comprehensive Design in Civil Engineering

Preface

This research project is part of the fulfillment of Professional Doctorate of Engineering (PDEng) Degree in Comprehensive Design in Civil Engineering (CDCE) program. The project is realized at the faculty of Civil Engineering and Geosciences of Delft University of Technology and with cooperation of Iv-Infra company.

I would like to thank my supervisors who helped me to perform this research, Prof. Dr. Frank Sanders, Prof. Dr. Bas Jonkman, Ir. Ad van der Toorn, Ir. Arno Willems and Ir. Jarda van Spengen.

If you have any questions or suggestions regarding this project, please do not hesitate to contact me.

Iman Karimi,

June 2013,

Delft, the Netherlands

Summary

Hurricane Ike made landfall on 13th of September 2008 on the Galveston Island in the Texas area. This hurricane is the third-costliest hurricane that ever has made landfall in the United States and the costliest hurricane in the history of the Texas. The possible future disasters should be prevented by using the best practices and existing technologies used in the Netherlands and New Orleans to protect the region.

The Ike Dike is a coastal barrier which would protect the Houston/Galveston region from the possible future hurricane storm surges. If we consider the Ike Dike as system level 1, in system level 2 the Ike Dike will consist of a system of levees, a coastal barrier and a storm surge barrier in the Bolivar Roads Pass. This storm surge barrier can be further divided to the environmental barrier and the navigational barrier in system level 3. The focus of this report is the conceptual design of the navigational storm surge barrier of the Bolivar Roads Pass as an important part of the Ike Dike concept.

In this report, firstly the Ike Dike is defined as a system with special requirements. The focus is then on the navigational barrier as part of this system and particularly the moveable gate. The requirements are defined for this barrier and an integral design method is chosen as the design approach. This integral design includes different steps.

In the first design step, different options for the barrier gate are investigated and evaluated using the Multi Criteria Analysis on the basis of boundary conditions and program of requirements. The floating barge gate is chosen as the best option for the moveable gate of the navigational barrier. The conceptual design of the barrier is the second design step which defines the system of the barrier and the operational phases. Concrete is chosen as the construction material and the barrier is designed for the full surge retaining height (MSL+5.5 m).

In design step 3, the barge gate is geometrically designed as a caisson structure. A gate with the dimensions of 230 m* 36 m* 22.5 m is the final design result with the weight of 70,779 tons. The complementary structures including bed protection, berthing system and articulation system are depicted and designed in the design step 4.

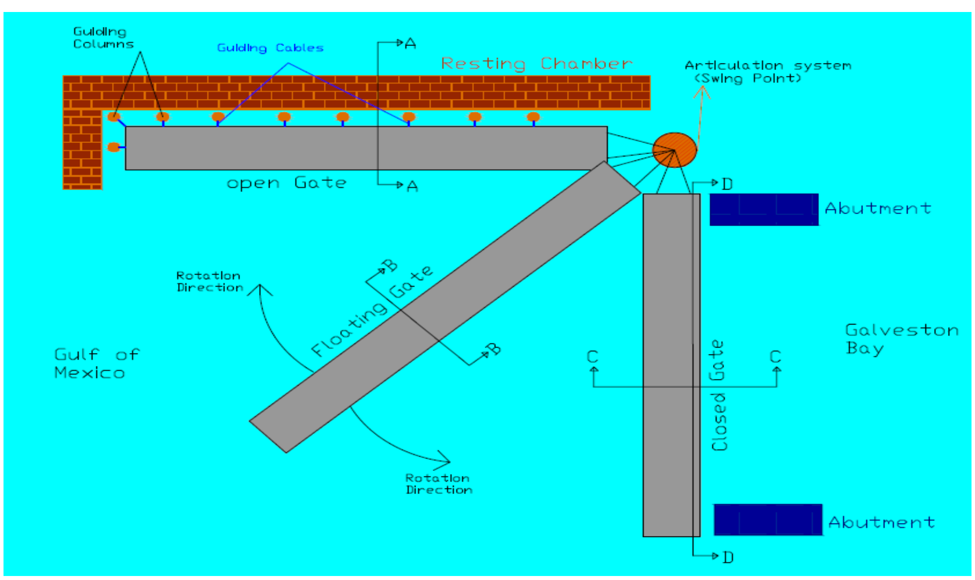
Design evaluations and reflections and risk analysis in design step 5 control the design and provide an overview of the design results and important design parameters. As part of the design iterations, in design step 6, some design revisions are realized including the choice of material (concrete or steel), retaining height selection, design with prestressed concrete and initial hydrodynamic analysis of the structure. Comparison of the steel barge gate with the concrete barge gate shows that the steel barge gate has less weight while it is more expensive.

Furthermore, the design with the prestressed concrete reduces the weight of the structure to 63,724 tons while increases the construction costs of the barge gate.

The preliminary design and overview of the supporting structures including abutments and foundations is the next part of the report. The abutments are designed as the piers in prestressed concrete with the dimensions of 24 m* 7 m* 5 m each. Due to the weak subsoil of the project site, deep foundations in the type of steel tubular piles filled with concrete are proposed as the suitable foundation type for the structure.

The project management aspects such as construction methods, maintenance aspects and cost calculations are the other discussed issues. The barge gate can be constructed in the dry dock and be transported to the site afloat. It is expected that the navigational barrier with the concrete barge gate costs 303 million USD while with the steel gate costs 538 million USD. The total cost of the barrier of the Bolivar Roads Pass (including the environmental barrier and the navigational barrier) is estimated in the range of 2.3 billion USD to 4 billion USD.

The choice of the concrete barge gate for the navigational barrier is expected to be an economical and realizable solution for the Bolivar Roads Pass storm surge barrier. However, there are some issues which should be investigated more. The articulation system (swing point) of the barge gate is an important aspect of the project which needs special attention. The barge gate in this size has not been realized yet and the operational phases should be investigated more using hydrodynamic analysis or by using the laboratory tests. The construction of the barrier and installations require offshore works which may in reality increase the costs compared to what is calculated in this research project. The optimizations should be done to reach the best configuration for the barge gate in the future.



Plan of the navigational barrier

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1 Introduction

This chapter includes a short background about the Galveston Bay area, the hurricane Ike, general information about the current report and the main purpose of it. By reviewing the previous studies, the main problem will be pointed out and the research questions will be written down. The desired design approach for the project will be also depicted. Finally, the structure of the report will be descried to offer a clear view for the readers.

1.1 Background

Hurricane Ike made landfall on 13th of September 2008 on the Galveston Island in the Texas area. This hurricane is the third-costliest hurricane that ever has made landfall in the United States [1] and the costliest hurricane in the history of the Texas. The damage from the storm surge and wind resulted by this hurricane is estimated around 24.9 billion USD [1]. The city of Galveston was inundated and the Bolivar Peninsula was hardly damaged due to the storm surge.

The Galveston Bay is located near the Greater Houston Metropolitan Area (GHMA), along the upper coast of Texas in the United States. The GHMA is one of the fastest growing metropolitan areas in the United States. Also, in the Galveston Bay, the population density especially on the west side of the Bay is high and has the high growing rate [4]. The largest and most important concentrations of petroleum refining and petrochemical processing plants in the United States are located in the Houston/Galveston area and the Port of Houston is the second-busiest port in the nation [2]. Before the occurrence of the hurricane Ike, a 25 foot (7.62 m) storm surges up the Galveston Bay was predicted [3]. This hurricane could have killed hundreds, made thousands homeless and jobless, destroy the nation's largest petrochemical complex, crippled its busiest port and caused 100 billion USD damage.

The economic importance of the area combined with the dense population and strong rate of population growth, local geography and the climate situation have made the area very vulnerable to the possible coastal flooding. The possible future disaster can be prevented by using the best practices and existing technologies used in the Netherlands and New Orleans to protect the region [3]. The coastal spine concept, which is the approach that was used in the Netherlands after flooding in 1953, might be the most attractive solution. The Ike Dike is a coastal barrier which would protect the Houston/Galveston region from the possible future hurricane storm surges.

The idea of the Ike dike has been assessed by different organizations and researches recently. In the next sections, the previous studies in this regard are being described and then the objective of the current study is going to be depicted on the basis of the problem statement.

1.2 Previous Studies

Several studies have been performed regarding the flood risk and the preventing methods in the Galveston Bay Area especially after the hurricane Ike. In this section an overview of these studies, particularly related to the current project, are summarized.

The structural design of sector gates proposed for storm surge barrier of the Bolivar Roads Pass in the Galveston Bay Area is studied by Davis et al (2010), under the project name of “The design of the Bolivar Roads Surge Barrier” by supervision from Dr. M. Miller [5]. In this project, it is proposed that the Bolivar Roads Pass can be closed by using the combination of vertical lifting gates and sector gates. The idea is similar to the Eastern Scheldt Barrier and Maeslantkering (Maeslant barrier) in the Netherlands. The barrier is designed for a lifetime of 200 years with the storm conditions of 1000 yr^{-1} (or 16 ft. / 4.9 m surge) while the channel depth is 65 ft. (19.8 m) which satisfies the needs for future shipping navigations. The study has the part about the environmental conditions and implies that the proposed solution is acceptable because enough flow circulations in the Bay is still possible. This report is started with the design of a sector gate and vertical lifting gate combination. There is not a comprehensive qualitative or quantitative method for choosing the best gate type.

The Master thesis by Ruijs (2011) is the other study under the name of “The effects of the Ike Dike barriers on Galveston Bay” which studies the effects of the barrier regarding the hydrodynamics, morphology and water quality on the Galveston Bay [6]. The impact of the barrier on the Galveston Bay’s hydrodynamics has been assessed by using the 2D model. Also, the impacts on the morphology and water quality have been evaluated by qualitative way. The 2D modeling concluded that the tidal range and tidal prism decreases with 10-19% due to the construction of 40-60% at the Bolivar Roads Pass, 20-39% including the extra energy losses. The closures of the Rollover Pass and San Luis Pass have the negligible effects. Also, the current speeds within the bay will decrease 19-37% for a 40-60% constriction while the current speeds at the Bolivar Roads barrier place increase to 1.3-1.6 m/s (which is currently 1 m/s).

“Applying best practices from the Delta Works and New Orleans to Galveston Bay” is the title of the internship report at the Dutch Ministry of Foreign Affairs and Texas A&M University by Stoeten (2012) [7]. In this report, the comparison of the Dutch Delta Works, New Orleans and the Ike Dike concept is provided. In fact, the “Best Dutch Practices” was the main basis for the protection of the New Orleans and Greater Houston Metropolitan. In all the systems, shortening the perimeter, keeping the surge out of the internal waters and ensuring passage by means of the navigational gates barriers were the main aspects of the work. The Galveston Bay is located above the sea level and doesn’t have the significant outflow of fresh water like the Netherlands and New Orleans. The report concludes that the Ike dike can effectively reduce the flood risk by keeping the surge out of the Galveston Bay. The cost estimation of the Ike Dike is about 6 billion USD in this report.

The report “Galveston Bay; Bolivar Roads flood risk reduction barrier: Sketch design” made by Jonkman et al (2013) provides suitable candidate solutions for a storm surge barrier in the Bolivar Roads as part of the Ike Dike system and provides recommendations for further research [8]. The report concludes that the optimal alignment of the storm surge barrier is a subject of further discussions. Also, the proposed barrier consists of two sections including the navigational section around 200 m wide and the environmental section around 2800 m wide. For the navigational part the barge and sector gates are proposed as feasible solutions or leaving the navigational part completely open. However, these proposals are just on the basis of the short workshop and related discussions.

The recent Master thesis studied by Stoeten (2013) under the title of “Hurricane surge risk reduction for Galveston Bay” is the recent work which includes the probabilistic behavior-oriented storm surge model for Galveston Bay [9]. Simulations performed in this project show that local wind set-up contributes up to 50% to the surge within the Bay depending on the landfall location and storm intensity. In addition, a preliminary flood risk assessment has been performed which compares the flood inundation effects for the 100 yr^{-1} and 10,000 yr^{-1} patterns. Some strategies have been also proposed for the flood risk reduction including the Coastal Spine, Ship Channel Gate and Texas City Levee Upgrade. The preliminary cost benefit analysis for these strategies are also included in this report.

Figure 1-1: Risk reduction strategies, Stoeten (2013)

Cox et al. (2013) has performed a study entitled “Sector gates in Bolivar Roads”. This report has a comparison between four alternative types of gate [10]. Finally, the flap gates and sector gates are chosen because of wide span of the navigational part of the barrier. The study has been continued by structural analysis of the sector gates by using the Finite Element Method (FEM). The comprehensive comparison of the alternative gate types is however missing in this study.

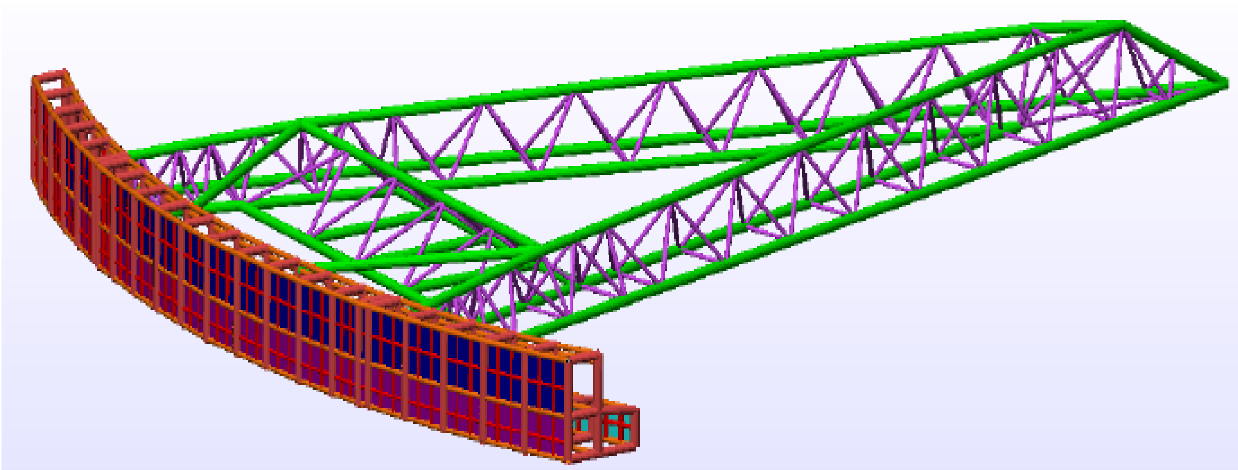


Figure 1-2: Structural modeling of the sector gate, Cox et al 2013

Van Breukelen (2013) has studied the applicability of inflatable dams as the storm surge barrier for the Bolivar Roads [53]. In this study, the barrier of Ramspol (near Kampen) in the Netherlands has been taken as a reference point in order to improve the inflatable barrier concept. Two designs with the inflatable barrier have been proposed for the Bolivar Roads Pass. Multiple smaller length inflatable barriers instead of one big barrier have been considered. The cost analysis has been also done for the design and it has been derived that the inflatable barrier for the Bolivar Roads (including the maintenance costs) costs 0.95 million euros per meter of barrier length which has been estimated that is cheaper than traditional barriers.

The other recent study is “The design of the Bolivar Roads surge barrier” by de Vries (2014) [11]. In this study the most cost-effective approach for design of the storm surge barrier in Bolivar Roads is investigated. The barrier in the Bolivar Roads has been divided to two parts of environmental part and the navigational part. The focus of the project is on the environmental part. The design of the environmental barrier has been done by using the caissons structures. The foundation design of the barrier has been investigated with different options. The navigational barrier design is out of the focus of this project.

1.3 Problem Statement

As it is shortly discussed in the previous sections, there are various studies about the Galveston Bay and especially the concept of the Ike Dike. Choosing the best alternative for the barrier type has the high importance for the project. As it is mentioned before, the barrier is suggested to be divided to two parts of the environmental part and the navigational part.

Some researchers like de Vries (2014) have studied the environmental part and have done the analysis about different alternatives. On the other hand, the studies about the navigational barrier is just focused on the design of the sector gate types previously.

A comprehensive Multi Criteria Analysis (MCA) for choosing the most suitable gate type and the conceptual design of the navigational barrier system are still missing. Different aspects in this regard such as barrier type, barrier operational systems, conceptual design of the moveable gate and the supporting elements, execution and construction considerations, maintenance and cost calculations should be investigated in more details.

1.4 Objective of the Study

The main purpose of this study is:

"Choosing the best alternative for the navigational barrier of the Bolivar Roads Pass, defining the system of this barrier and make a conceptual design for the moveable gate and some of the supporting elements."

The study will give an overview of the possible gate options for the barrier with some project examples and advantages and disadvantages of each gate type. Then the boundary conditions and requirements for the navigational barrier should be investigated. The MCA is going to be used for selection of the best gate option. The selected gate is going to be further designed by considering different aspects such as barrier system, conceptual design, design of some supporting elements, design evaluations and reflections, risk analysis and some project management considerations such as construction issues, maintenance and cost calculations of the project. The conclusions and recommendations for the future research will make the report complete.

1.5 Research Questions

The main research question of this study is as follow:

"What is the most suitable, realizable, reliable and economical option and its conceptual design for the navigational storm surge barrier in the Bolivar Roads Pass?"

The sub-questions can be formulated as follow:

1- *What type of navigational gate can be considered for the design?*
2- *Which method is used for the selection of alternatives and how they would be scored?*
3- *What would be the system of the moveable gate and supporting structures?*
4- *How can the barrier be designed and how much it will cost?*

5- *What are the risks in the design of the barrier and what are the rooms for improvements and optimizations?*

1.6 Design Approach

The integral design approach is going to be used for designing the navigational barrier. Different system levels can be considered for the design. Figure 1-3 shows the main system of the Ike Dike project.

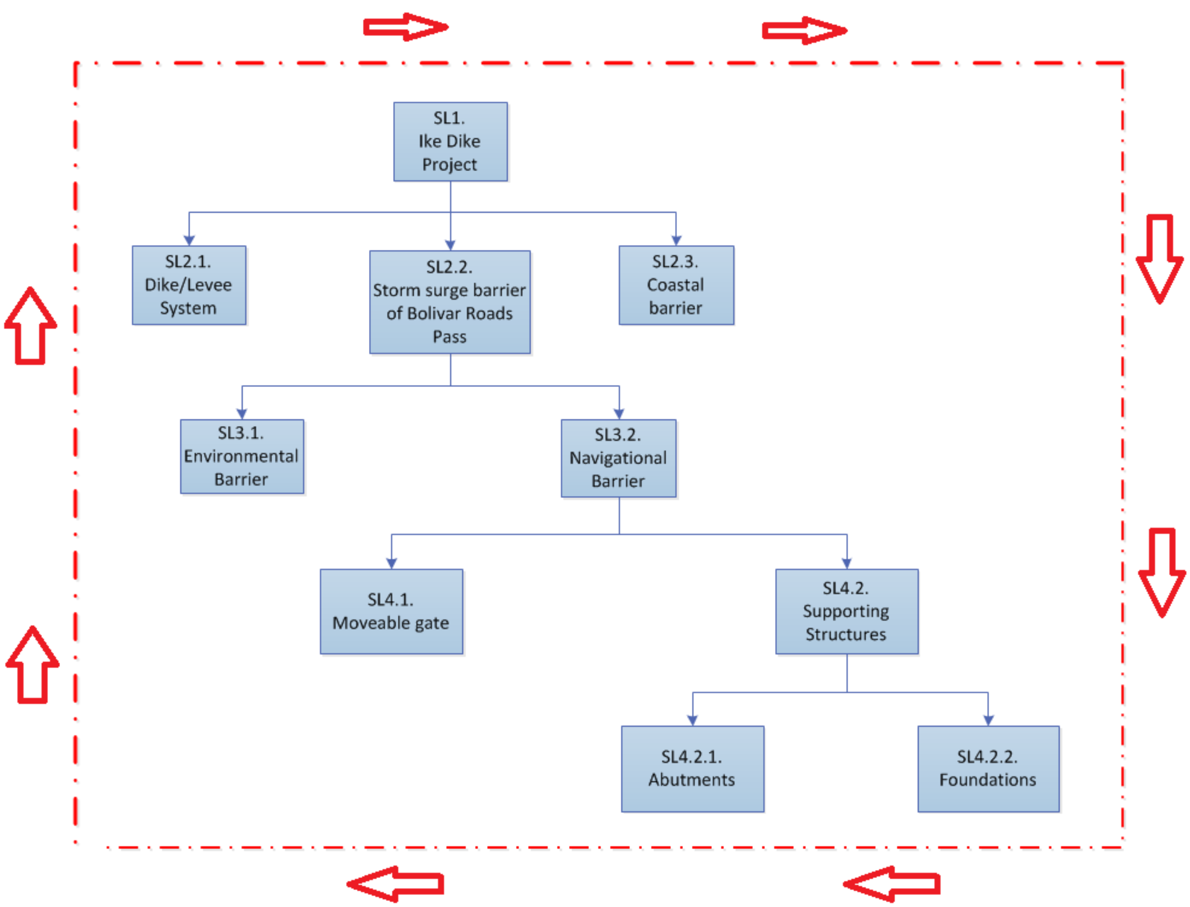


Figure 1-3: The main system of the Ike Dike project

The focus of the current report is on the navigational barrier or system level 3.2. (SL3.2.). The navigational barrier can further divided to some sub-systems which are SL4.1. moveable gate and SL4.2. Supporting structures. The red arrows in the figure show the iterative process and interconnections between different system levels.

The design procedure is an iterative procedure. It means that in each system level after the design, the requirements of the previous step can be checked and the design can be further adjusted to meet all the needs of that system. In the other words, there is a relation between the systems from top to bottom and also bottom to top.

In the current report, the main focus is on the moveable gate (SL4.1.). The design of moveable gate consists of different Design Steps (DS) (Figure 1-4). These design steps have been shown using the DS and the appropriate numbers in the rest of this report. In the following chapters, the design will go on according to these steps. The program of requirements for the Ike Dike project can be seen in Figure 1-5 and for the moveable gate in Figure 1-6.

The design of the moveable gate starts with a MCA for selection of the best gate alternative. Then the conceptual design of the gate should be performed in DS2. When the conceptual design of the gate is determined, geometry of the gate should be derived using structural and stability checks in DS3. Then the complementary structures designs are treated in DS4. DS5 considers the design evaluation and reflection including the risk analysis. On the basis of the information gained and as part of the iterative process of the design, in DS6 some design revisions will be performed. The design of the abutments, an overview about the foundations and some project management aspects such as construction considerations and cost calculations are going to be depicted later in the report.

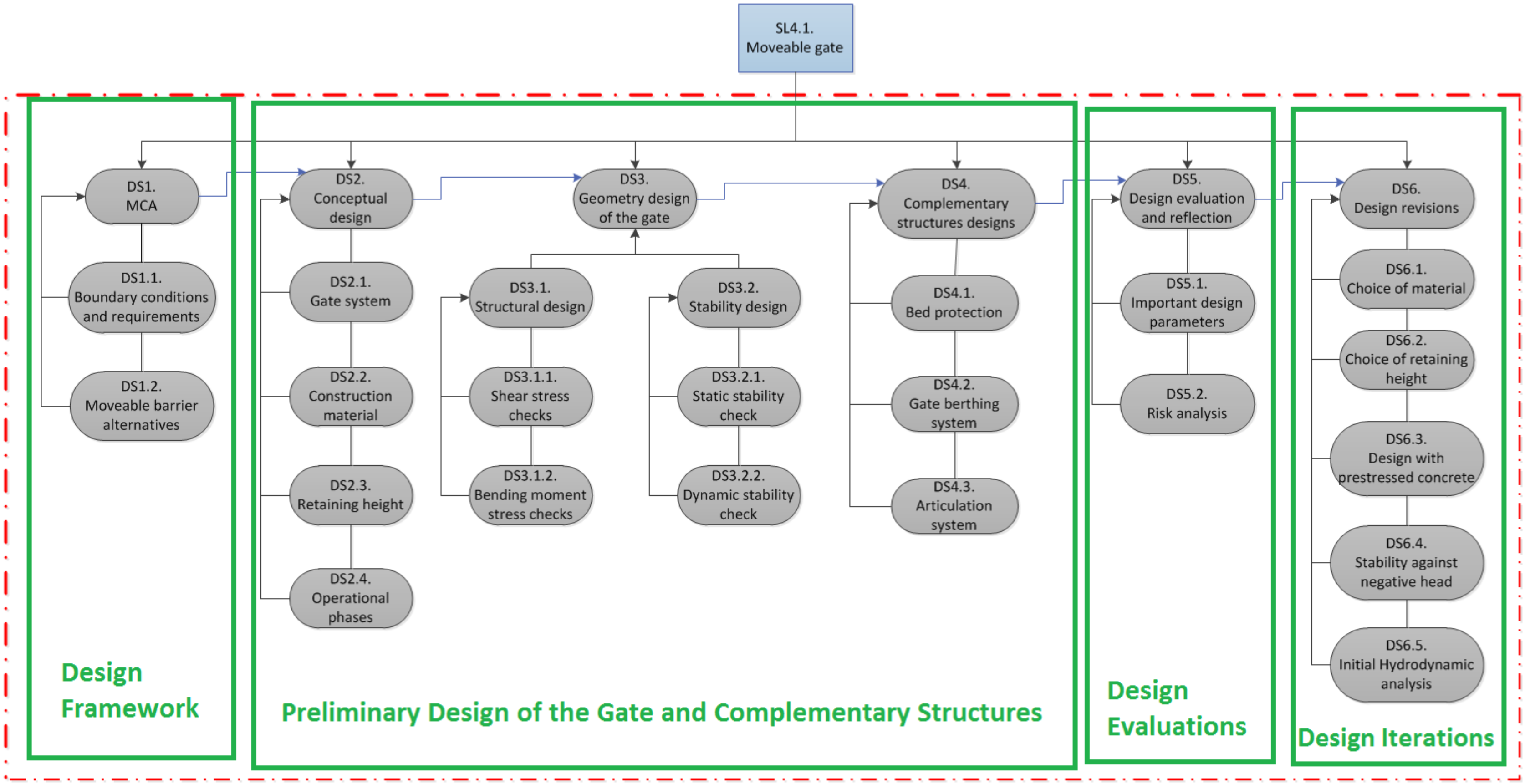


Figure 1-4: Moveable gate integral design and process

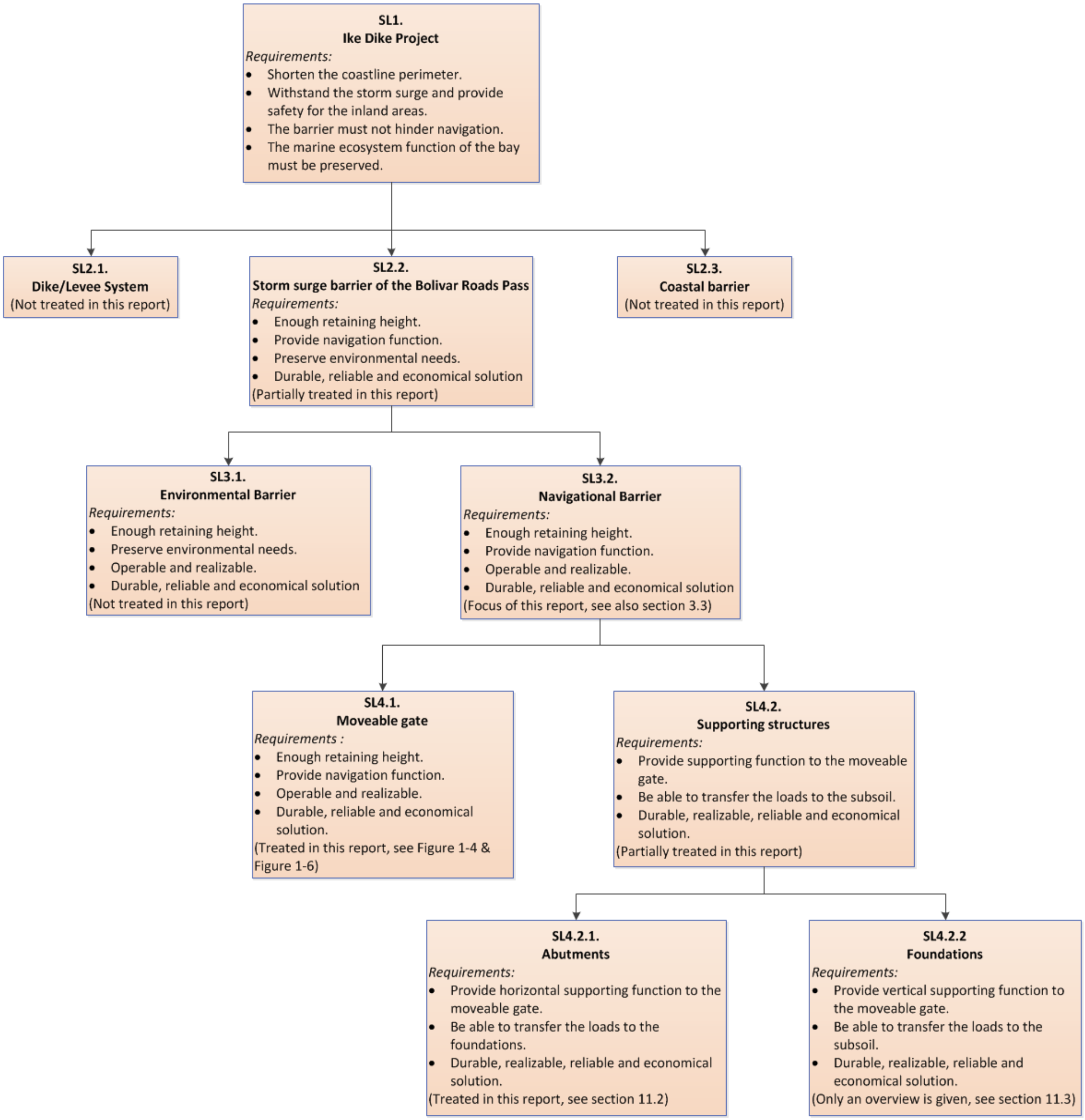


Figure 1-5: Program of requirements in the system level of the Ike Dike project

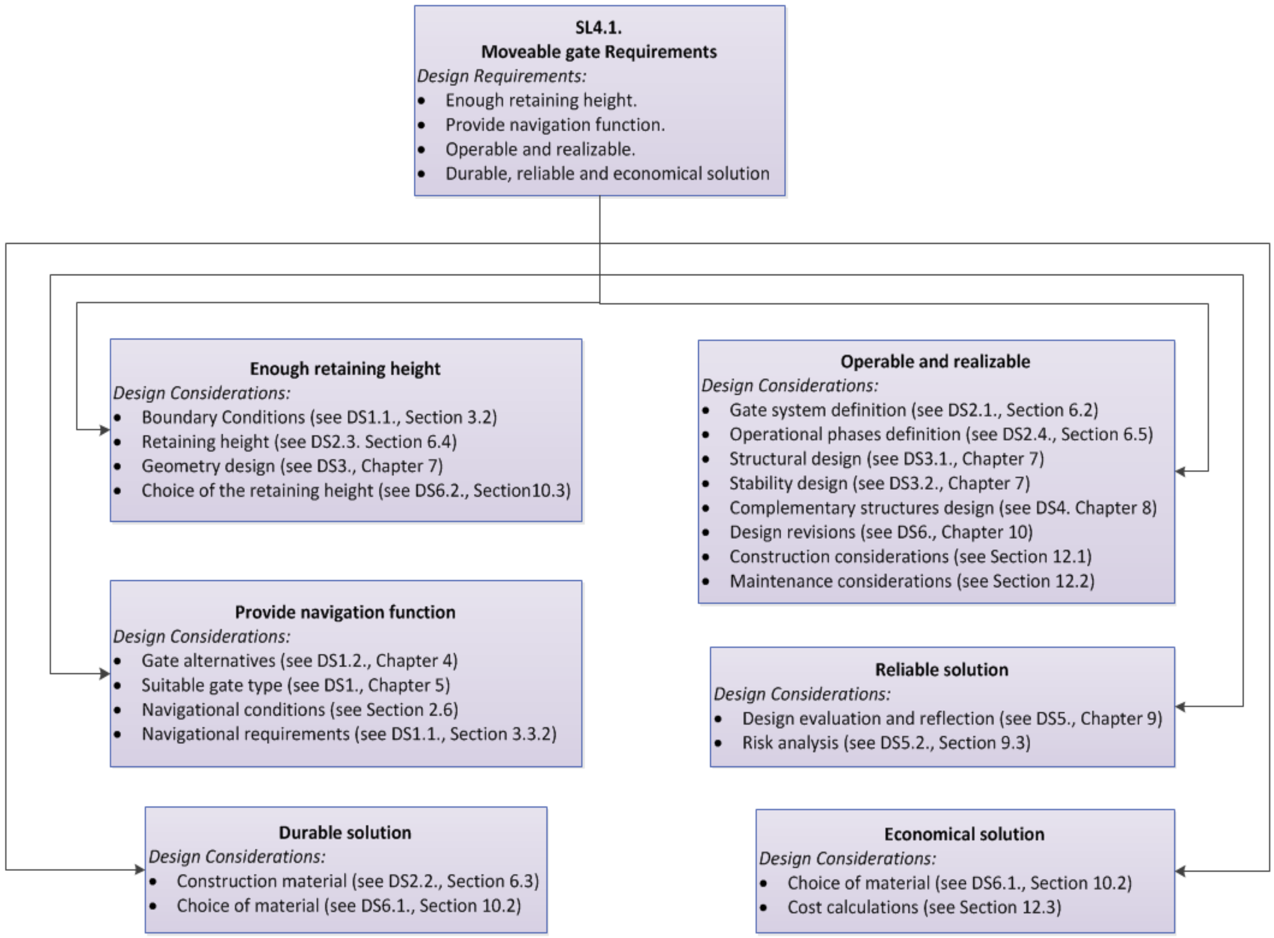


Figure 1-6: Program of the requirements of the moveable gate and related sections and design steps

1.7 Structure of the Report

The main purpose of this report is to provide a reader with a simple reading structure regarding the project design. On the basis of the design approach which is described in the previous section, the report is divided to different chapters. The list of the chapters and the most important contents of them are listed below.

Chapter 1: Introduction

- Introduction to project
- Introduction to report

Chapter 2: Galvestion Bay Description

- Project location and characteristics
- Ike Dike concept

Chapter 3: Boundary Conditions and Requirements (DS1.1.)

- Boundary conditions
- Requirements

Chapter 4: Gates and Moveable Storm Surge Barriers (DS1.2.)

- Overview of gate alternatives
- Advantages and disadvantages of the alternatives

Chapter 5: Multi Criteria Analysis (MCA) (DS1.)

- Multi Criteria Analysis (MCA)
- Selection of the best gate option for the navigational barrier

Chapter 6: Conceptual Design (DS2.)

- Barrier system definition
- Construction material selection
- Retaining height selection
- Operational phases descriptions

Chapter 7: Geometry Design of the Gate (DS3.)

- Structural checks and design
- Stability analysis
- Determination of the gate dimensions

Chapter 8: Complementary structures Designs (DS4.)

- Bed protection design
- Berthing system design
- Articulation system conceptual design

Chapter 9: Design Evaluation and Reflection (DS5.)

- Important design parameters explanation
- Risk analysis
- Risk register tables and fault trees
- Design Challenges and following tasks

Chapter 10: Design Revisions (DS6.)

- Design revision check for choice of material
- Design revision check for choice of retaining height

- Design revision check for design with prestressed concrete
- Stability calculations against negative head
- Initial hydrodynamic analysis

Chapter 11: Supporting structures

- Design of the abutments
- Overview about the foundations

Chapter 12: Project Management Considerations

- Construction and execution considerations
- Maintenance considerations
- Cost calculations

Chapter 13: Final Conclusions and Recommendations

- Conclusions
- Recommendations

References

Chapter 14: Appendices

- Details of calculations for different designs and analyses

2 Galveston Bay Description

2.1 Introduction

In this section the general description of the Galveston Bay is presented. The purpose is to make the reader familiar with the condition and characteristics of the project area. This chapter provides the information regarding the SL1. to SL3. of the project (Figure 2-1).

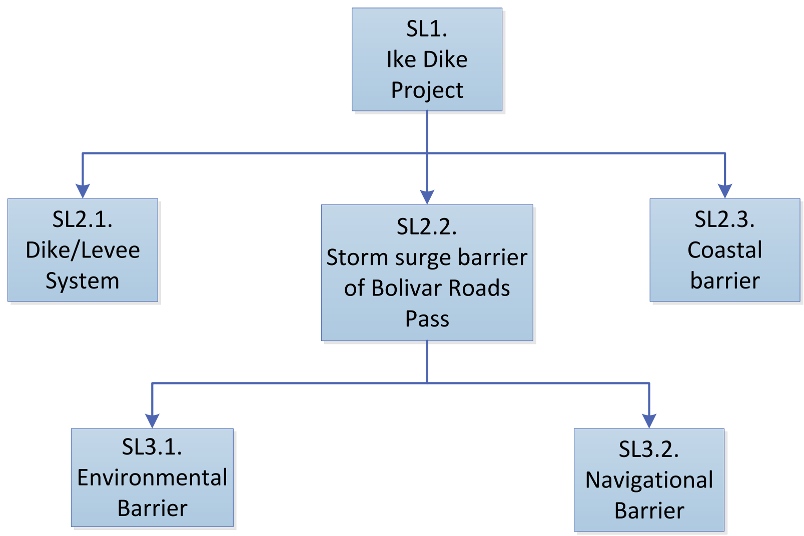


Figure 2-1: Project system level 1 to 3

2.2 Location and Characteristics

Galveston Bay is a large semi-enclosed estuary system which is located on the Upper Texas Coast, in southeast of Texas and near the Houston Metropolitan Area (Figure 2-2). It is the largest estuary in the Texas Coast and consists of five sub-bays including Trinity Bay, East Bay, West Bay and upper and lower Galveston Bay.

Galveston Bay is a high valuable resource for the country. It is home to variable estuary habitats such as marshes, oyster reefs and sea grass beds. Also, it provides recreational opportunities, ecological services, natural resources and transportation links.

The Bay has the estimated surface area of 1,554 square kilometers and the shoreline length of about 374 kilometers [12]. The average depth of the Bay is estimated as 3 meters [8]. Also, Galveston Bay is separated from the Gulf of Mexico by Galveston Island and Bolivar Peninsula. Both of these barrier Islands are relatively low with around 1 to 3 meters above MSL height, narrow with 1 to 2 kilometers width and straight. Figure 2-3 shows the Digital Elevation model of the Galveston Bay.

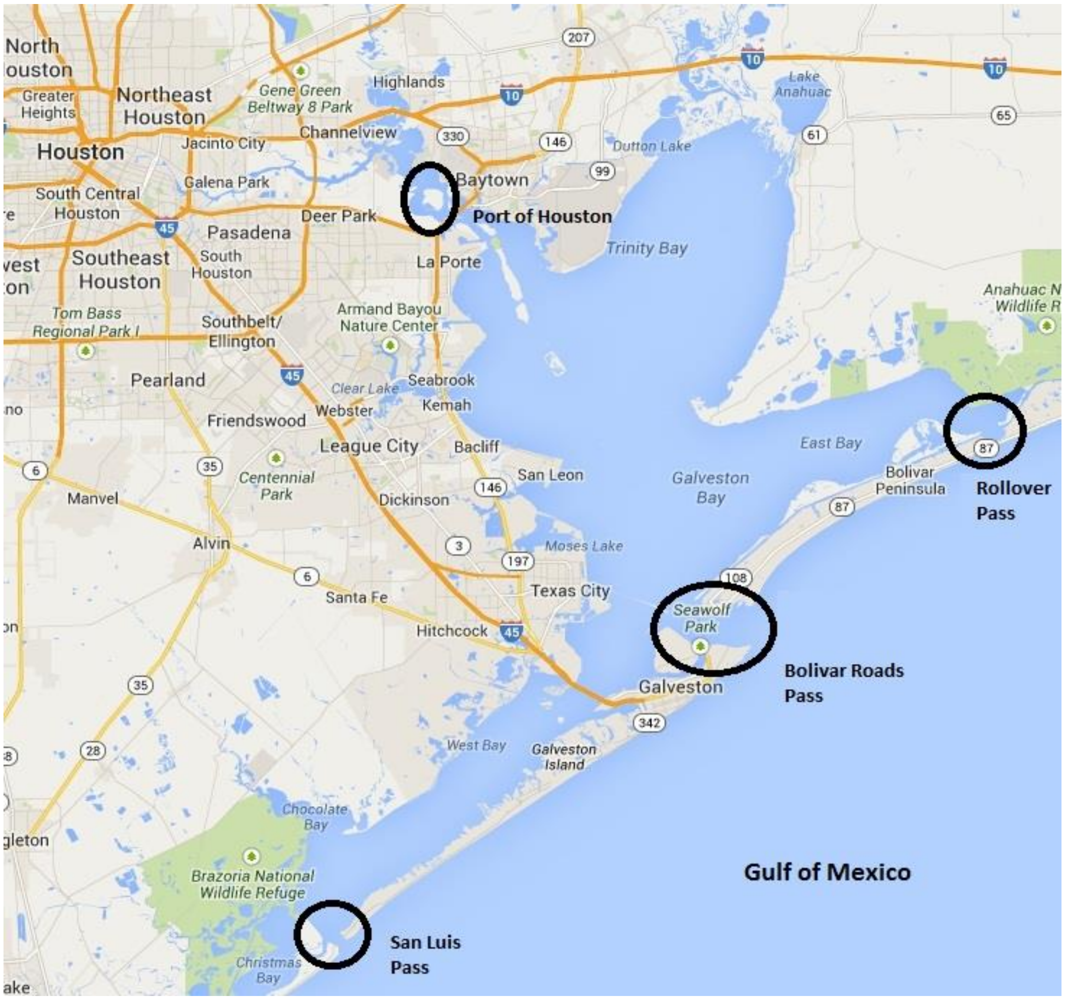


Figure 2-2: Galveston Bay map

As it has been shown in the Figure 2-2 three channels connect the Galveston Bay to the Gulf of Mexico, namely Bolivar Roads Pass, San Luis Pass, Rollover Pass where the Bolivar Roads Pass is the largest channel. The flow velocities within these channels have been estimated as below 2 m/s in the normal conditions [8].

The Bolivar Roads Pass has the width of around 2.8 km (Figure 2-4). This pass is responsible for 80% of the tidal exchange between the Galveston Bay and Gulf of Mexico. It is also an important shipping passage from the gulf of Mexico to the Port of Houston. It is actually part of the Houston Shipping Channel (HSC) and the Gulf International WaterWay (GIWW). In the future, the new Panamax vessels should be able to navigate in this channel.

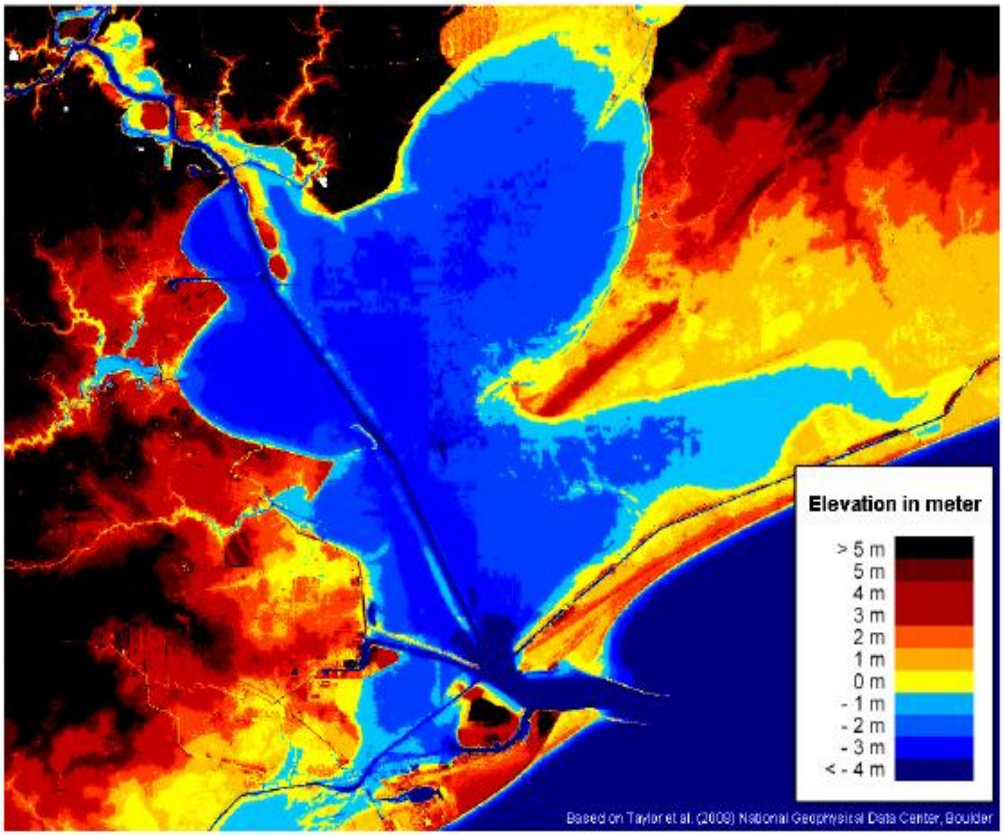


Figure 2-3: Digital Elevation model of the Galveston Bay [13]

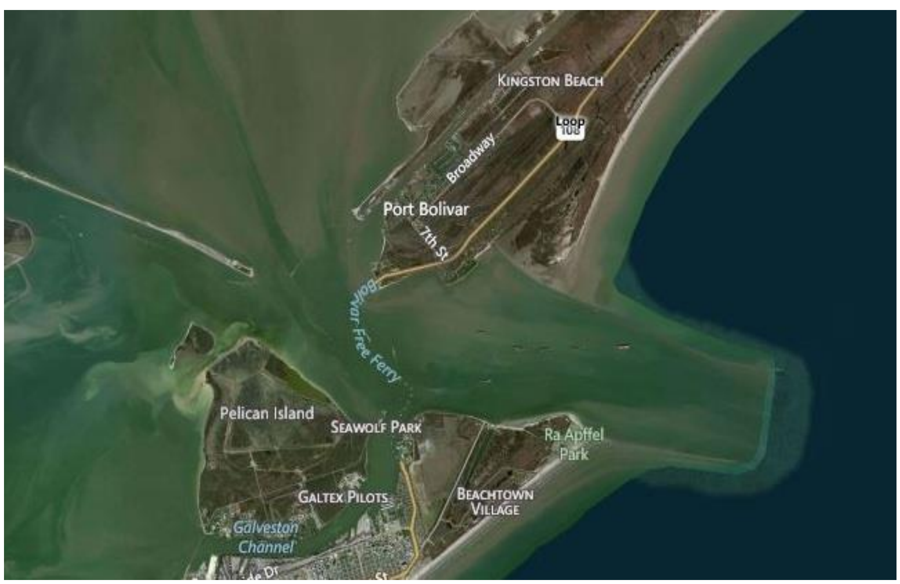


Figure 2-4: Bolivar Roads Pass [14]

The San Luis Pass (Figure 2-5) is the other inlet which connects the West Bay to Gulf of Mexico and with its about 900 meter wide and average about 2 meter depth is responsible for 20% of the tidal exchange between the Galveston Bay and Gulf of Mexico.

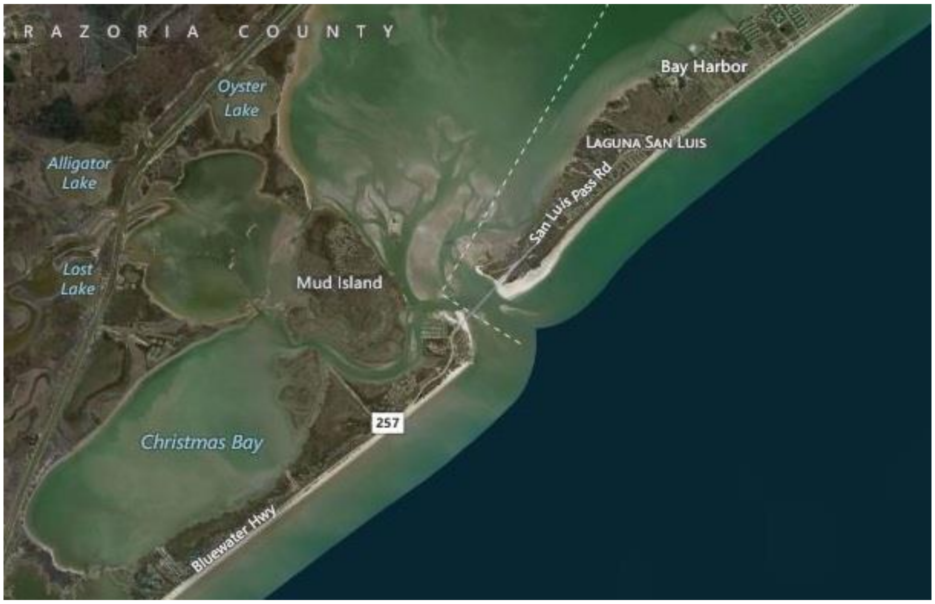


Figure 2-5: San Luis Pass [14]

The other inlet is Rollover Pass (Figure 2-6) which is a 60 m wide artificial inlet and it connects the East Bay with Gulf of Mexico. The Pass is initially constructed for the purpose of improvement of fishing in the Bay in 1956. The contribution of this pass to the tidal exchange between the Galveston Bay and Gulf of Mexico can be neglected [11]. There are plans from the Texas General Land Office (GLO) to close this pass permanently because of the sediment problems.

Figure 2-6: Rollover Pass [14]

2.3 Hurricane Ike

Hurricane Ike was a strong category 2 hurricane which made landfall near the city of Galveston on 13th of September 2008. The navigational functions should be maintained with a storm surge barrier. The surge values along the Gulf Coast were estimated as 3 to 4 meters along the Galveston Island and as 4 to 5.2 meters along the Bolivar Peninsula. The hurricane Ike was the costliest hurricane in the history of the Texas and made 112 casualties.

In advance of landfall of the hurricane Ike, unpredicted water level increase happened along the Texas coats which is named as forerunners. This forerunner effects should also be considered in the design of the storm surge barrier.

2.4 Ike Dike Concept

The Ike Dike is a coastal barrier which is going to be constructed in the future to protect the Houston-Galveston region and Galveston Bay from hurricane surge storm [3]. The project is initially proposed by Professor Bill Merrel from Texas A & M university at Galveston. The idea is to extend the protection afforded by the existing Galveston Seawall along the rest of the Galveston Island and along the Bolivar Peninsula with a 17 ft (5.2 m) high revetment near the beach or raising the coastal highways [3]. The Galveston Seawall has the height of 5.2 meter and it is designed for 100 yr^{-1} storm event [15]. This seawall is the concrete structure which is founded on timber piles and is protected by sheet pilling and a layer of riprap from undermining.

The proposed Ike dike consists of around 100 km massive levee system including the gate storm surge barriers in the bolivar Roads Pass and San Luis Pass.

The main elements of the Ike Dike would be as follow (Figure 2-7):

- A dike or levee system around 29 km from existing seawall to the San Luis Pass
- A coastal barrier around 56 km as high as existing seawall along the bolivar Peninsula
- A Storm Surge Barrier, including environmental part and navigational part in the Bolivar Roads Pass

The main important requirements of the Ike Dike are [16]:

- It should shorten the coastline perimeter as much as possible.
- The system may overflow.
- The barrier must not hinder navigation.
- The marine ecosystem function of the bay must be preserved.

Figure 2-7: Ike Dike concept

The cost estimation for the Ike dike is different and it is in the range of 3 to 10 billion USD. Whatever the future cost of the Ike dike would be, the beneficial results (in terms of risks and damages reductions) from the prevention of the inundation of the Galveston Bay and important industrial and oil and gas facilities in this area would be higher [17].

An important aspect of the Ike Dike is to keep the storm surge outside of the Galveston Bay (Figure 2-8) [7]. In this way the water levels remain within acceptable levels preventing from widespread flooding of the area. The maximum wind induced setup for Galveston Bay is estimated as 2.13 meter.

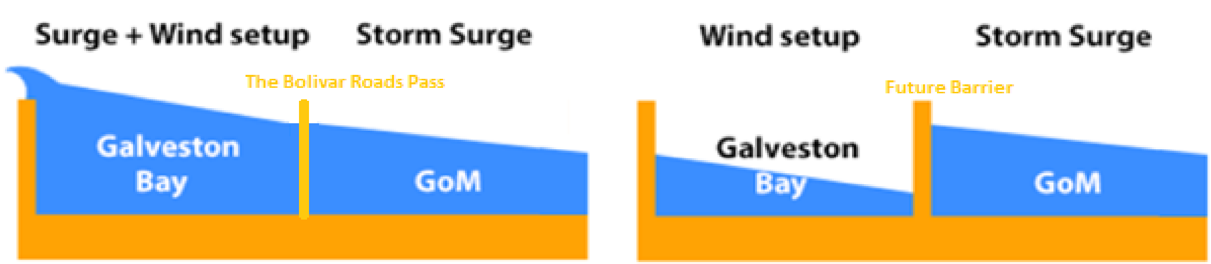


Figure 2-8: Schematized cross section of storm surge in Galveston Bay and Gulf of Mexico; Left: without barrier, Right: with barrier [7]

2.5 System Layout

The main system of the storm surge barrier in the Bolivar Roads Pass includes the environmental section and navigational section. The focus in this project is on the navigational section as it is mentioned before. The location of the barrier should be chosen with respect to the optimization of the investment cost.

The investment cost of the storm surge barrier can be estimated by the following formula [52]:

$$C_b = U_b\,(L_b \,.\, H_b \,.\, RH_b) \qquad \text{Eq. 2-1}$$

Where:

- C_b = Cost of the barrier ($)
- U_b = Unit cost of one meter barrier ($/m^3$)
- L_b = Length of the barrier (m)
- H_b = Height of the barrier (m)
- RH_b = Retaining height of the barrier (m)

The estimated unit cost of the barrier is U_b = 40,000 (USD/m^3) for a navigational barrier and U_b = 30,000 (USD/m^3) for an environmental barrier [52].

The optimization regarding the location has been done by de Vries (2014) where the shortest way from the Bolivar Peninsula to Galveston Island with the length of 2,757 m has been proposed to be the most cost effective option [11] which is logical. In this project, the navigational barrier is going to be designed on the basis of this alignment. Figure 2-9 shows the navigational section and environmental section in this alignment next to the other structures of the Ike Dike. The cross section of the layout can be seen in Figure 2-10.

The Bolivar Roads Pass is an important inlet for the Galveston Bay in terms of ecological and environmental aspects. The water circulation after the construction of the barrier should be in a way that does not affect the ecosystem. In the study of Ruijs (2011), it has been found out that the constrictions more than 40% will have a certain negative effects on the Bay's ecosystem [6]. That is why the environmental section has been considered which should support this condition by providing the opportunity for sufficient water exchange between the Galveston Bay and the Gulf.

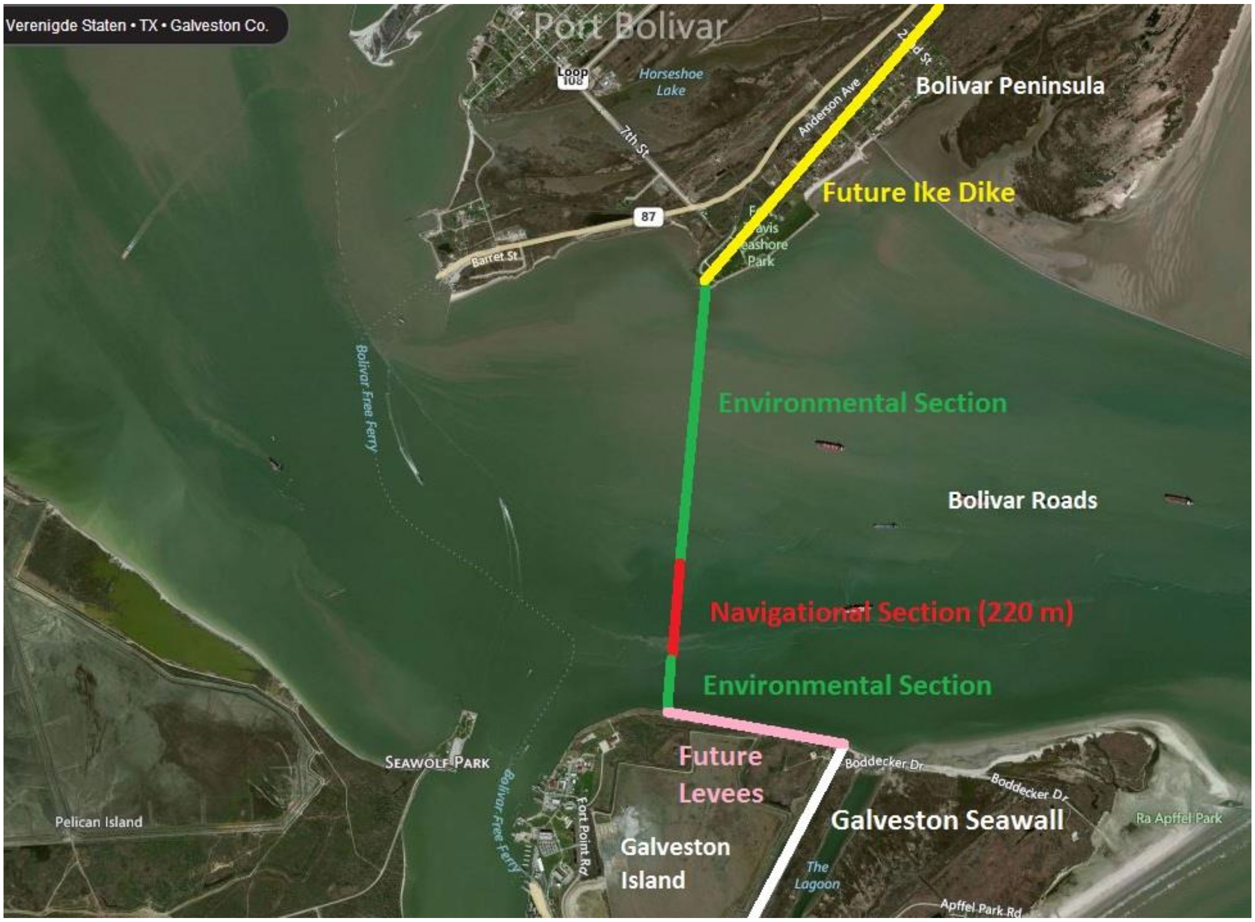


Figure 2-9: Storm surge barrier system layout

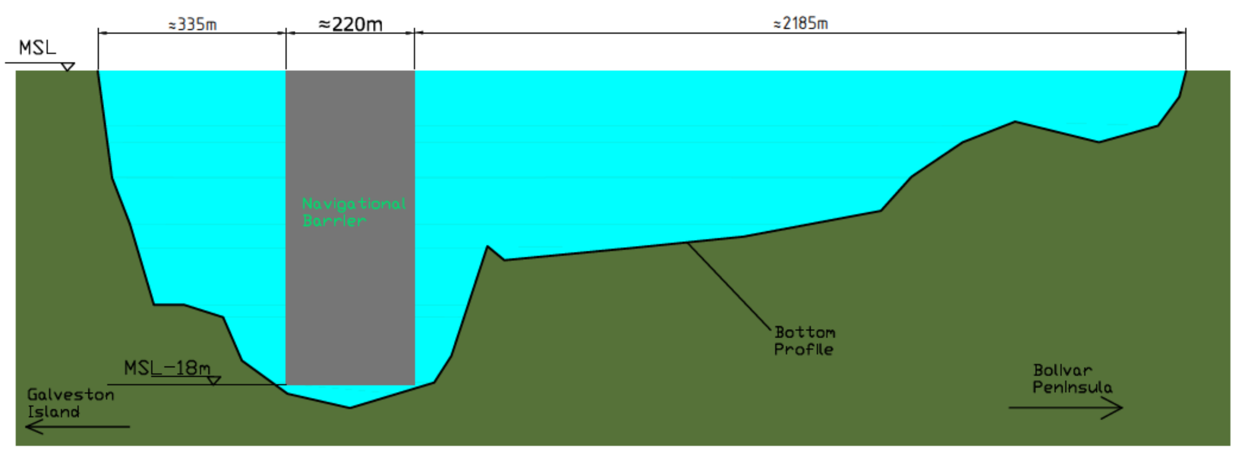


Figure 2-10: Cross section of the layout of the storm surge barrier

2.6 Navigational Conditions

The port of Houston is the most important port in the area of the Texas. The Houston shipping Channel (HSC) provides the passage for ocean-going vessels to reach this port from Gulf of Mexico. From around 1836 is this channel a place for the transportation of goods. The

Galveston Bay and Buffalo Bayou have been dredged to be able to accommodate larger ships on the basis of the market needs for growing ship sizes.

The current dimensions of the channel are 13.7 m depth and 161.5 m width which provides navigation for not fully loaded Suezmax tankers with the dimensions of 13.7 m draft, 50 m width and 270 m length [11]. In the future the channel should provide passage for New Panamax vessels with the dimensions of 15.2 m draft, 49 m width and 366 m length.

	Length (m)	Beam (m)	Draft (m)	Air Draft (m)
Old Panamax	289.6	32.31	12.04	57.91
New Panamax	366.0	49.0	15.2	57.91

Table 2-1: Summary of Old and New Panamax dimensions [48]

The Bolivar Roads Pass is currently used for shipping and the traffic intensity is about 7,000 ships per year [8]. The traffic is expected to be increased because of the Panama Canal expansion. The annual increase of 10% is mentioned by Vessel Traffic Service (VTS) since 2004 [18].

The schematic cross section of the HSC in the current situation is shown in Figure 2-11.

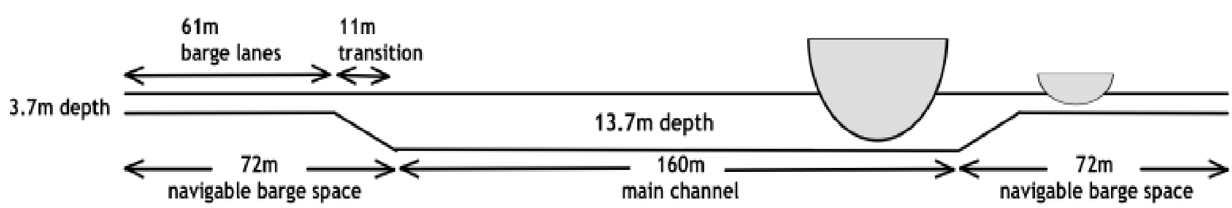


Figure 2-11: The schematic cross section of the HSC in the current situation, modified from [18]

On the other hand, the Gulf Intracoastal Waterway (GIWW) as part of the Intracoastal Waterway (IWW) with the length of around 1,700 km is located along the Gulf Coats of the United States and includes the parts in Brownsville, Texas, Fort Meyers and Florida. The Texas part of this waterway is about 686 km. It provides the navigational route along the coast without the possible hazards of the open sea [11]. The depth of the channel is about 3.7 m and the initial purpose is for transporting of the goods. The dimensions of the barges are limited in this channel to 360 m length and 16.8 m width [19]. Enlarging the canal to 4.9 m by 46 m had been proposed. The situation of the navigational routes are shown on the Figure 2-12.

Figure 2-12: Navigational routes in the Galveston Bay Area [11]

2.7 Conclusions

In this section the concept of Ike Dike has been depicted. The Ike Dike will protect the area against the storm surge. The storm surge barrier in the Bolivar Roads Pass consists of the environmental barrier and the navigational barrier. Due to the importance of the Bolivar Roads in terms of the navigation and economic aspects, the design of the navigational barrier should be investigated more to reach the best option for construction and realization of the storm surge barrier. The main purpose of this report is to evaluate the options for the navigational gate and reach to the most beneficial design and construction method.

3 Boundary Conditions and Requirements (DS1.1.)

3.1 Introduction

In this section boundary conditions and requirements for the navigational gate of the Bolivar Roads Pass are going to be described. This information is going to be used as the input in the later phases for the design of the barrier. This chapter includes the information of DS1.1. (Figure 3-1).

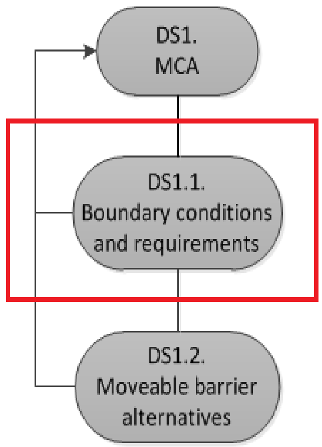


Figure 3-1: Design Step 1.1.

3.2 Boundary Conditions

3.2.1 Bathymetry Boundary Conditions

The bathymetry map of the Bolivar Roads Pass is shown in Figure 3-2.

3.2.2 Design Storm

The design storm is an important element of the boundary conditions. In the study of Stoeten [9] the storm of 10,000 year was considered and the peak surge level without including the forerunner surge was calculated. The forerunner effects can be added through extrapolating the forerunner surge of the Hurricane Ike [11]. In this way, the design storm of the barrier can be derived. The combination of the forerunner and maximum surge can be considered as the conservative 10,000 year storm for the design of the barrier.

It should be mentioned that this design storm is derived with simple analysis. For the time being this design storm is going to be used for the design calculations in the following chapters. The focus of the current project is not in determining the design storm and it is recommended for future researches.

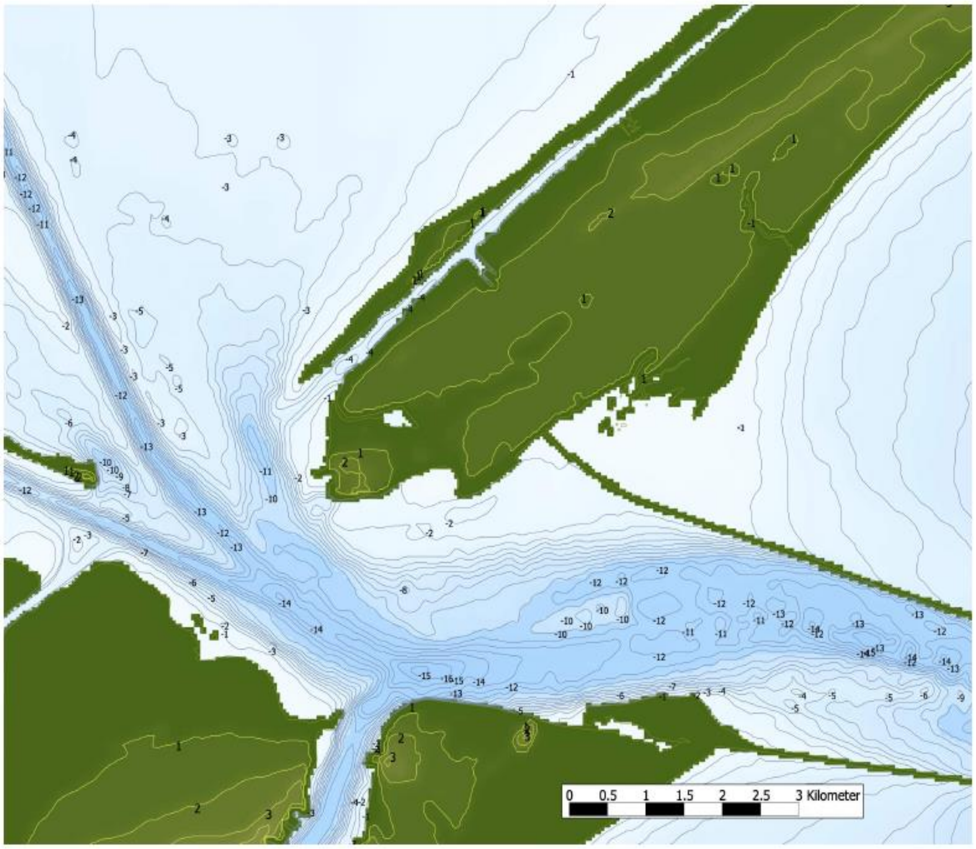


Figure 3-2: Bathymetry map of Bolivar Roads [49]

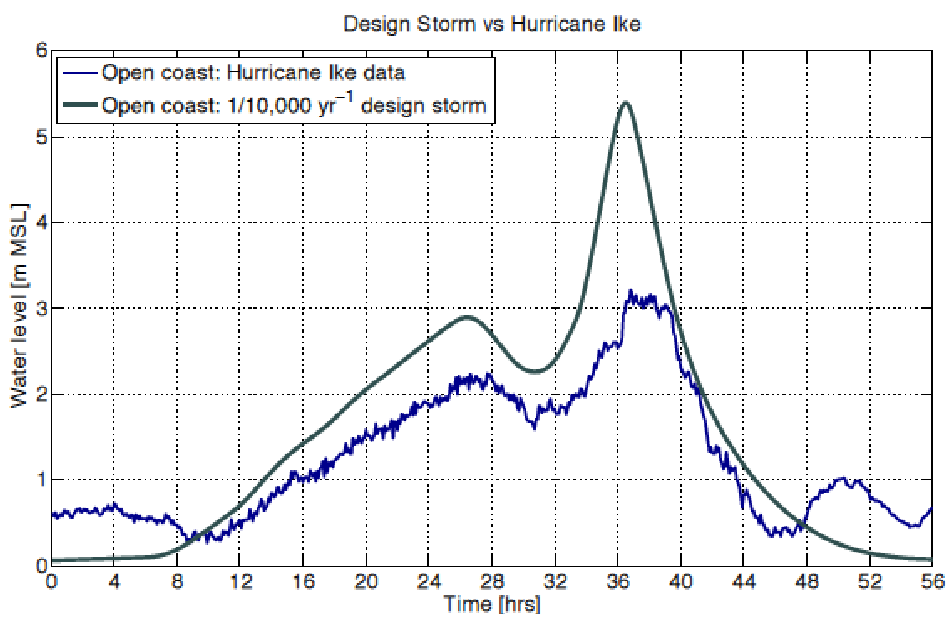


Figure 3-3: Design storm of the barrier [11]

3.2.3 Hydraulic Boundary Conditions

In the Galveston Bay, tides are from mixed types (diurnal and semi-diurnal) with a tidal range of approximately 0.35 m [8].

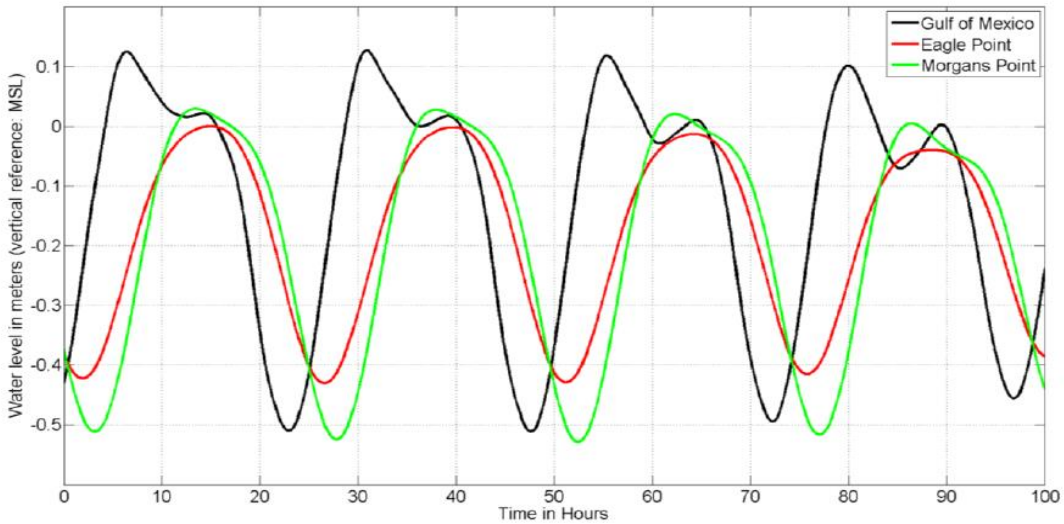


Figure 3-4: Tidal propagation in Galveston Bay [8]

The hydraulic boundary conditions can be considered in two separate situations; regular conditions and hurricane conditions. In the regular conditions the normal situation is considered while in the hurricane conditions the surge levels of the 10,000 year storm is assumed. The assumption is made also for the Seal Level Rise in the next 100 years. Table 3-1 summarizes the hydraulic boundary conditions in the Bolivar Roads Pass [11].

3.2.4 Geotechnical Boundary Conditions

The available data about the geotechnical conditions of the project site is limited. The soil in the Bolivar Roads mainly consists of the soft clay at the upper layers. The exact characteristics of the clay layers are unknown. The strong sand layer begins from -40 m [130 ft.] of Mean Sea Level (MSL). Table 3-2 shows the available data about the soil layer classification and strength conditions [11].

3.2.5 Meteorological Boundary Conditions

The main meteorological boundary conditions can be summarized as below [11]:

- Galveston’s climate has the humid subtropical situation.
- Annual rainfall in Galveston is about 1,104 mm (43.46 in).
- Rainfall does not increase the design surge.

Hydraulic Boundary Conditions in Bolivar Roads Pass			
Conditions		**in m**	**in ft**
Regular conditions			
	Max. current Velocity	1,0 (m/s)	3,3 (ft/s)
	Average river discharge	540 (m^3/s)	1,9 x 10^4 (ft^3/s)
	Tidal difference	0,35 (m)	1,16 (ft)
	Low tidal prism	0,85 x 10^8 (m^3)	3,0 x 10^9 (ft^3)
	High tidal prism	2,8 x 10^8 (m^3)	9,9 x 10^9 (ft^3)
	Seal level rise in 100 years	1,0 (m)	3,3 (ft)
Hurricane conditions			
1/10,000 yr storm	Max. surge level (h_{surge})	5,4 (m)	17,7 (ft)
	Max. wave height (H_{max})	5,9 (m)	19,4 (ft)
	Significant wave height (H_s)	3,3 (m)	10,8 (ft)
	Peak wave period (T_p)	7,9 (s)	7,9 (s)

Table 3-1: Hydraulic boundary conditions

Layer	Classification	Depth		Relative Density	Undrained shear Strength	
		MSL - m	MSL -ft		KN/m^2	kips/sqft
0	Very soft clay	+1.5 - 0.0	+4.9 - 0.0	-	12	0,25
1	Interlayered very soft clay	0 - 3	0 - 10	-	12	0,25
2	Loose to dense recent sands	3 - 15	10 - 50	50%		
3	Soft to firm clay	15 - 20	50 - 66	-	24	0,5
4	Laminated firm clay and silt	20 - 32	66 - 105	-	36	0,75
5	Firm to stiff clay	32 - 40	105 - 131	-	48	1
6	Very dense sand	40 - 50	131 - 164	> 85%		

Table 3-2: Soil layers and strength properties

3.3 Requirements

The requirements of the navigational barrier for the Bolivar Roads are discussed in this section. The final design of the barrier should fulfill these requirements.

3.3.1 General Requirements

The navigational barrier obviously should be open in the normal conditions. In the storm and hurricane conditions the barrier should be able to withstand the design storm. The barrier should be realizable with acceptable cost expenses.

3.3.2 Navigational Dimensions Requirements

As it is mentioned before, the barrier should be able to provide the navigational conditions for the New Panamax tankers with the following dimensions:

- Length of the design ship = L_s = 366 m
- Width of the design ship = W_s = 49 m
- Draft of the design ship = D_s = 15.2 m
- Air draft of the design ship = AD_s = 57.91 m

On the basis of these information and the formulas provided by PIANC the navigational channel depth and width can be calculated as below [50]:

Navigational Channel Width:

The width of the navigational channel (unidirectional) should fulfill the following requirement (PIANC):

$$W_{min} = W_{BM} + \sum W_i + 2W_B \qquad \text{Eq. 3-1}$$

In which:

Width component	**Condition**	**Width implication**
Basic width W_{BM}	Good Maneuverability	$1.6W_s$
Additional widths W_i		
– Prevailing cross-winds	25 kts	$0.4W_s$
– Prevailing cross-current	0.4 kts	$0.2W_s$
– Prevailing wave height	<1m	0
– Aids to navigation	VTS	$0.1W_s$
– Seabed characteristics	Soft	$0.1W_s$
– Cargo hazard	High	$1.0W_s$
Bank clearance W_B	Sloping edge	$0.5W_s$
Total		$4.4W_s$

Which results in:

W_{min} = 4.4 x (49) = 215.6 m or → *W_{min} = 220 m [722 ft]*

Navigational Channel Depth:

The following formula for the channel depth is used:

$$d_{nav} = D_s - \zeta_{tide} + s_{max} + \zeta_m + s_s \qquad \text{Eq. 3-2}$$

in which:

- d_{nav} = Depth of navigation channel (m)
- D_s = Draft of design ship (m)
- ζ_{tide} = Tidal elevation above reference level below which no entrance is allowed (m)
- s_{max} = Maximum sinkage due to squat and trim (m)
- ζ_m = Vertical motion due to wave response (m)

- s_s = Remaining safety margin or net under keel clearance (m)

By considering the D_s = 15.2 m, ζ_{tide} = 0 m, s_{max} = 0.75 m, ζ_m = 0.5 m, s_s = 0.5 m then:

d_{nav} = 16.95 m → *d_{nav} = 17 m [56 ft]*

3.3.3 Current Velocity Requirements

PIANC has proposed that the longitudinal current velocity more than 1.5 m/s in navigational channel is considered as strong [51]. This amount should be the maximum current velocity for the navigational channel of Bolivar Roads in normal conditions.

3.3.4 Safety Level Requirements

According to Stoeten (2013), the protection with the safety level of 1/10,000 yr^{-1} would provide the highest rate of return [9]. In this project the same safety level is assumed which the related wave and surge data can be seen in the boundary conditions.

3.3.5 Lifetime Requirements

The navigational barrier should be designed for the lifetime of 100 years like the other large storm surge barriers. The construction material and the design should be able to provide such a lifetime for the structure.

3.3.6 Environmental Requirements

As it has been mentioned in the previous parts of the report, the Bolivar Roads is the main inlet of water for the Galveston Bay. Galveston Bay has the great importance for the environmental issues. The study from Ruijs [6] has revealed that the maximum constriction of the water exchange between the Bay and the Gulf due to the barrier is limited to 40% where the less constriction is of course more desirable.

The barrier in the Bolivar Roads is considered as two different parts of the environmental part and the navigational part. The environmental part should be designed in a way to fulfill the above mentioned requirements by considering enough openings in the structure. Then the navigational barrier, which is open in the normal conditions, would have no sever effect on environmental conditions of the Galveston Bay. Further investigations regarding the interaction between the environmental part and the navigational part is proposed.

3.4 Conclusions

In this chapter the requirements and boundary conditions are described. The navigational barrier should be designed for the storm of 1/10,000 yr^{-1} where the maximum storm surge level of 5.4 m should be considered for the design. The lifetime of the structure is 100 years and the navigational barrier has the dimensions of 220 m width and 17 m depth which provides the desired navigational conditions for the passing vessels without any hindrance.

By considering the requirements mentioned in this section, the MCA can be done in the next step to provide the best gate option which meets the needs of the project.

The summary of the main aspects of the requirements for the navigational barrier can be seen in Table 3-3. For sure this list contains only the main requirements and the sub-requirements are not included.

Program of Requirements for the Navigational Barrier	
Type	**Criteria**
General	Open in normal conditions
	Closed during the hurricane conditions
	Realizeable solution
	Operable solution
	Reliable solution
	Economical solution
Navigational	Enough width
	Enough depth
	Enough air draft
Current velocity	Acceptable current velocity in navigational channel
Safety level	Enough retaining height
Lifetime	Enough lifetime
	Durable solution
Environmental	Environmentaly friendly solution

Table 3-3: Program of requirements for the navigational barrier

In different design steps, the outcome of the design can be checked regarding fulfillment of the program of requirements. (see Figure 1-5 and Figure 1-6 for more information regarding the related sections and design steps to the requirements)

4 Gates and Moveable Storm Surge Barriers (DS1.2.)

4.1 Introduction

Before starting the design process, in this chapter different types of the navigational gates and moveable storm surge barrier are described. The purpose is to make an overview about the possible solutions for the navigational barrier in the Bolivar Roads Pass. Next to the main description of the gate type, some project examples are depicted to make the functions of the gate more clear. Then the advantages and disadvantages of the gates are described. This chapter deals with the DS1.2. of the integral design (Figure 4-1).

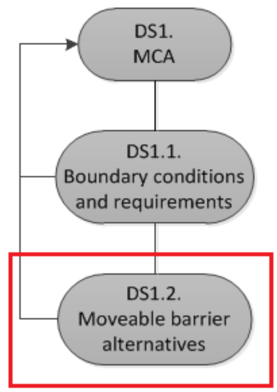


Figure 4-1: Design Step 1.2.

4.2 Gate and Navigational Barrier Types

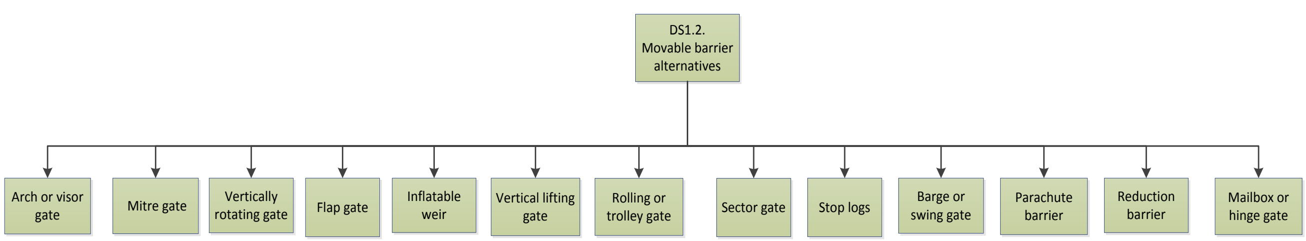


Figure 4-2: Movable barrier alternatives

4.2.1 Arch or Visor Gate

An arch or visor gate is a three-hinged arch that pivoted on horizontal pins and spans from abutment to abutment across the waterway [20]. It rotates upward for storage and downward to close the channel. It limits the allowing air draft of the vessels. The gate closure can be done by using the gravity forces of the leaf while the opening of the gate can be done by mechanical hoists with wire ropes placed on concrete structures built on the piers [21].

Project examples

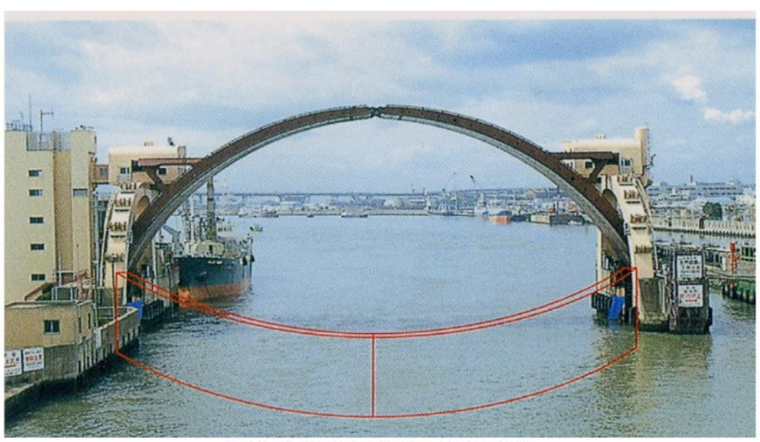
Rhine Visor Weirs:

Constructed during 1956-1966, these double visor gates with the span of 54 m and the gate weight of 200 tons are used to control flow for water generation and navigation. It is located in Hagestein, the Netherlands [28].

Figure 4-3: Rhine visor weirs, Hagestein, the Netherlands [28]

Aji River Barrier:

This arch gate, consists of steel, with the span of 57 meter and 530 tons weight is the storm surge barrier which protects the city of Osaka in Japan [29]. It is constructed from 1966 to 1970 and it has a side gate with the span of 17 m and weight of 107 tons. The height of the main gate is 11.9 m and the height of the side gate is 11.55 m.



Red line indicate the situation of closed gate

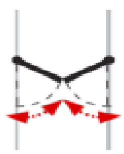
Figure 4-4: Aji river visor barrier, Osaka, Japan [29]

Advantages and Disadvantages

Arch or Visor Gate	
Advantages	**Disadvantages**
Proven concept	Limmited clearance height for shipping
Material efficient	Destruction of the landscape
	Large opening limitations
	Maintenance and inspection

Table 4-1: Advantages and Disadvantages Arch or Visor Gates

4.2.2 Mitre Gate



Mitre gates can be considered as the invention of Leonardo da Vinci and they are common from the 16 and 17 century [22]. They are more common for the shipping locks rather than flood control [20]. Mitre gates are double-leafed gates while the leaves make an angle positioning upstream when the gates are in a closed position. When the gate is open, the leaves are positioned in locked wall recesses. When the gate should be closed, the leaves are turned around 60 degrees and they meet in the centre line of the lock. The significant thrust to the abutments are forced in this situation. Mitre gates are operable only if the water levels on both sides of the gates are equal. The maximum width of the single gate already built is 25 m [22].

Mitre gates are favorable when the waves and currents are limited such as the locks cases [11]. At the locations with severe conditions these gates are less attractive. Also, they are sensitive to reversed water heads which makes them not favorable for storm surge barriers [22].

Project examples

Goole Caisson:

In this project the mitre gates are designed to prevent a loss of water in Goole docks should the canal wall be breached and to prevent the canal flooding if the sea locks fail [30]. It is constructed from 2001 to 2002 located in Great Britain.

IJmuiden Locks:

This well-known locks complex is the biggest locks located in the Netherlands and it connects the port of Amsterdam to the North Sea [22]. The complex includes four locks where the biggest lock with the width of 50 m has straight rolling gate and the other three locks, with the biggest width of more than 20 m, have the mitre gates.

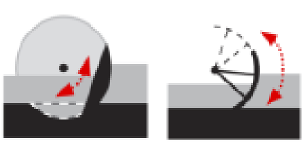
Figure 4-5: Example of Mitre Gate in The Netherlands [22]

Advantages and Disadvantages

Mitre Gate	
Advantages	**Disadvantages**
Proven concept	Little or no controlled operation under flow and waves
Little space required	Sensible to vibration, reverse head and waves
Unlimmited clearance height for shipping	Large opening limitations
Not subjected to wind	
Horizontal closure	
Discharge of excess water through gate	

Table 4-2: Advantages and Disadvantages Mitre Gates

4.2.3 Vertically Rotating Gates



Vertically rotating gates have two types in general, Segment gates with circular side disks that are stored in a bottom recess and radial gates or Tainter gates which are the conventional and most useable gates that are rotated above the water level and leave space for small ships to pass underneath them [22]. A radial gate has a skin plate that is mounted on an open structural frame and is supported by strut arms at each side of the gate; The strut arms extend to trunnion bearings mounted on abutment walls on either side of the gate opening [20]. The gates can be stored submerged and raise to close the opening or stored overhead and lowered to close flow. The vertically rotating gates in bottom recesses (namely segment gates) may be vulnerable to silting. The abutments can be built together with the driving systems within the cofferdams. The sill can be constructed in site or prefabricated and floated to site [22].

Project examples

Thames Storm Surge Barrier:

Located in London, United Kingdom, this storm barrier consists of four main navigable openings with a breadth of 61 m and two smaller navigable openings with a breadth of 31.5 m which are closed by segment gates (rising gates) and four non-navigable openings with a breadth of 31.5 m are closed by radial gates (falling gates). The radius of the segment gates are 12.2 m and are stored in bottom recesses in the concrete sill in the open position [22]. Maximum height (design water level) is 17 m.

Figure 4-6: Thames barrier, London, UK

Ems Barrier:

Located in Germany, this barrier provides flood protection and supports navigation. It has a length of 476 m with 7 openings [31]. The main shipping opening with a width of 60 m has the segment gate which is stored in a recess in the sill in the open position. This barrier has been constructed in the period of 1998 to 2002. The barrier structure is built underwater with the reinforced concrete. The five lateral openings beside the main opening are equipped with vertical lift gates. All the gates are operated by using the double-sided oil-hydraulic drives which can be controlled from the control room situated at the northern end of the barrier [31].

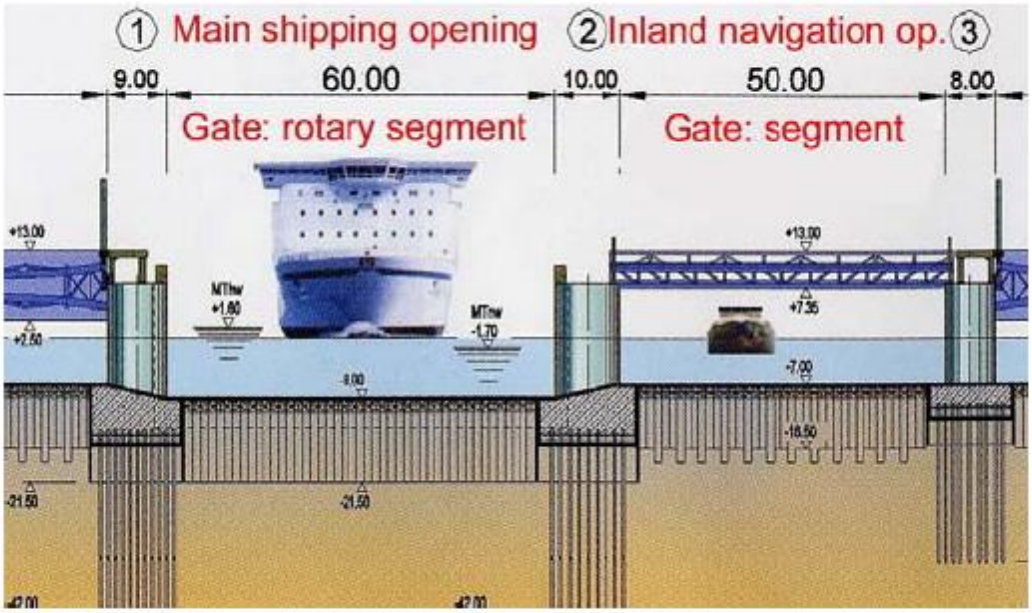


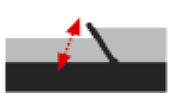
Figure 4-7: View of the main shipping opening and inland navigation opening, EMS barrier, Germany [31]

Advantages and Disadvantages

Vertically Rotating Gates	
Advantages	**Disadvantages**
Proven concept	Limmited clearance height for shipping (radial gate)
Large gate opening possible	Load transfer and concentration
Immidiately ready for operation	High sill tolerance demands (segment gate)
Controlled operation flow and wave	Access and maintenace (segment gate)
Little space required	Vulnerable to silting, objects and corrosion (segment gate)
No shipping clearance (segment gate)	Sensitive to oscillation in case of overflow (segment gate)
Inspection and maintenance	Open gates subject to down-pull forces
Limited horizontal flow contraction	Open gates subject to wave loads
Sutiable for reverse head and flow	
Radiad gate kept free of sill	
Excess water through gate	

Table 4-3: Advantages and Disadvantages Vertically Rotating Gates

4.2.4 Flap Gates



Flap gates are not visible when the flood protection barrier is not in uses [22]. They are hinged along the upstream edge of the gate and attached to a sill foundation. They are submerged in the stored position to the bottom recess. To close the flow, the downstream edge is rotated upward [20]. They are favorable types of gates in terms of the wide openings where a maximum gate span can be up to 100 m [11]. In general, the flap gates can be driven by hydraulic cylinders or they can be pneumatic gates which are operated by air injection into floating tanks. This type of gates have not been applied a lot in the flood protection projects up to now.

Project examples

MOSE Buoyant Flap Gate:

The Module Sperimentale Eletromeccanico (MOSE) project is a flap barrier to protect the city of Venice and the neighboring areas along the Venice Lagoon against floods from the Adriatic Sea. It consists of 78 flap flood gates which will be provided at 3 locations. They will vary in width from 3.6 m to 5 m and the length will vary from 18 m to 28 m. When the water level becomes more than 1.1 m, air is pumped into the metal box structure and the gate will rise up and block the tidal flow. The estimated final cost of the barrier is 2.3 billion euro. The duration of works was estimated initially as 8 years [32].

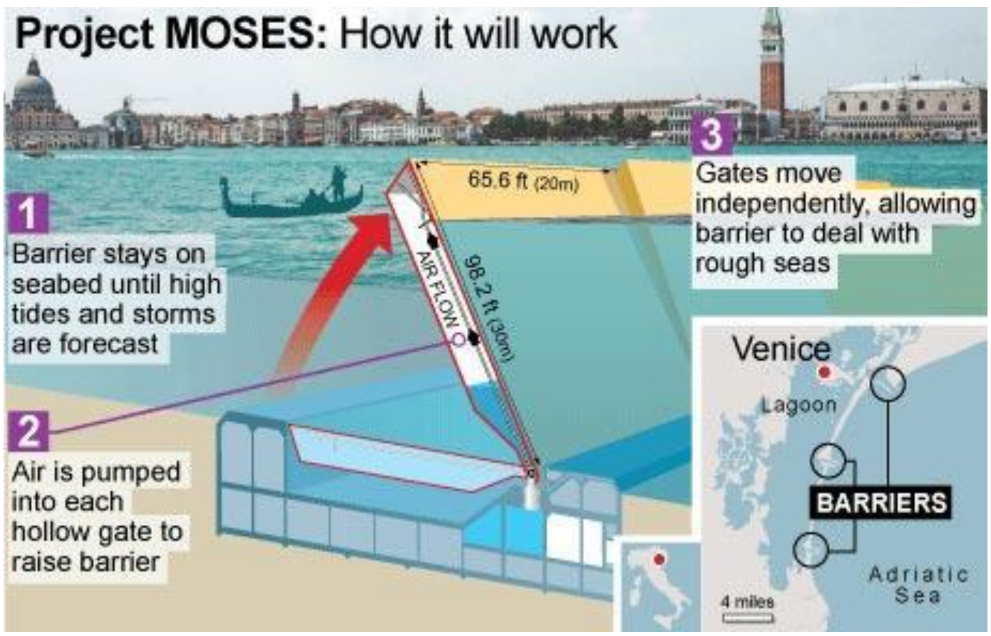


Figure 4-8: MOSE barrier, Venice, Italy [23]

Stamford Hurricane Barrier:

The East Branch Barrier at Stamford which is constructed in 1968 is a barrier consists of a earth-and-rock dike with the length of 866 m with the top elevation of more than 54 m and a 28 m opening channel with a single steel flap gate protection [22]. In this flap gate, the hollow steel gates rests on the bottom of the channel and it is raised to close the opening by means of the hydraulic cylinder. The gate lifted in 20 minutes.

Figure 4-9: Stamford Hurricane Barrier, US [22]

Sauer Closure Gate:

This barrier protects the cities and islands by the river Rhine against flooding. There is a single flap gate with 7 m height and 60 m length. It is constructed from 1990 to 1993 in France. The structure is the prefabricated steel structure and the total weight of the flap gate is 280 tons. The operation is made by two hydraulic cylinders which are in the abutment and fed by same hydraulic generator [33].

Figure 4-10: Saner Closure Flap Gate, France [33]

Advantages and Disadvantages

Flap Gates	
Advantages	**Disadvantages**
Large gate openning possible	Sensitivity to vibration
Suitable for deep water	Not fully controlled (Pneumatic case)
Unlimited shipping clearance	Concentration cylinders (Hydraulic case)
Limited failure risks beacuse of separated flaps	Corrosion and marine growth under water
Little space required	Hinges may wear out in sand
Controlled operation flow and wave	Maintenace and inspection
Not subjected to wind	Small stiffness during operation
No destruction of the landscape	Subject to down-pull flow forces
Vertical closure, single flap	Subject to wave loads
Excess water through one flap or by lowring the gate crest	Natural frequencies low; small stiffeness, great mass
High closure and opening speed	Not functioning in both ways (Pneumatic case)
Functioning in both ways (Hydraulic case)	Accumulation of sand behind the gate in close state
Not strong confinement of horizontal flow (gates used alternately)	

Table 4-4: Advantages and Disadvantages Flap Gates

4.2.5 Inflatable Weirs

Inflatable barriers (rubber dams) are composed of bladders (fabrics) secured to a bottom foundation and the barrier is raised by inflating the bladders with air or water [20]. In fact, the fabric is fixed to a reinforced concrete sill using clamp plates and anchor bolts. This type of barrier can be used even for long spans of 100 m. Rubber dams are widely used in the world but mainly in river engineering and water control applications and for creation of water reservoirs. The application of this type of barrier is restricted to the shallow water currently maybe because of the difficulty in fabrication of reinforced rubber sheet with large dimensions [22].

Project examples

Ramspol Barrier:

The Ramspol barrier located in the Netherlands, is in operation since 2002 and can be mentioned as the only major flood protection barrier in the world of this type. The barrier consists of 3 identical rubber dams with a width of 60 m and provides 2.7 m of flood protection from inland river flood waters. The dams can be inflated around 8.2 m above the sill. One of the barriers is the shipping channel. When the barrier is not in used the rubber sheets are stored in the bottom recesses. In the open situation, the dams are filled with water and air. The air is blown to the dams through the openings in the abutments while at the same time water from the upstream side flows into dams through pipes in the base. For deflating the dams, the air escapes through vents in the abutments and the water is pumped out. Then the sheet can be retracted into the recesses by using the guiding rollers. The barrier can be closed within an hour. This barrier is constructed from 1997 to 2002 [34].

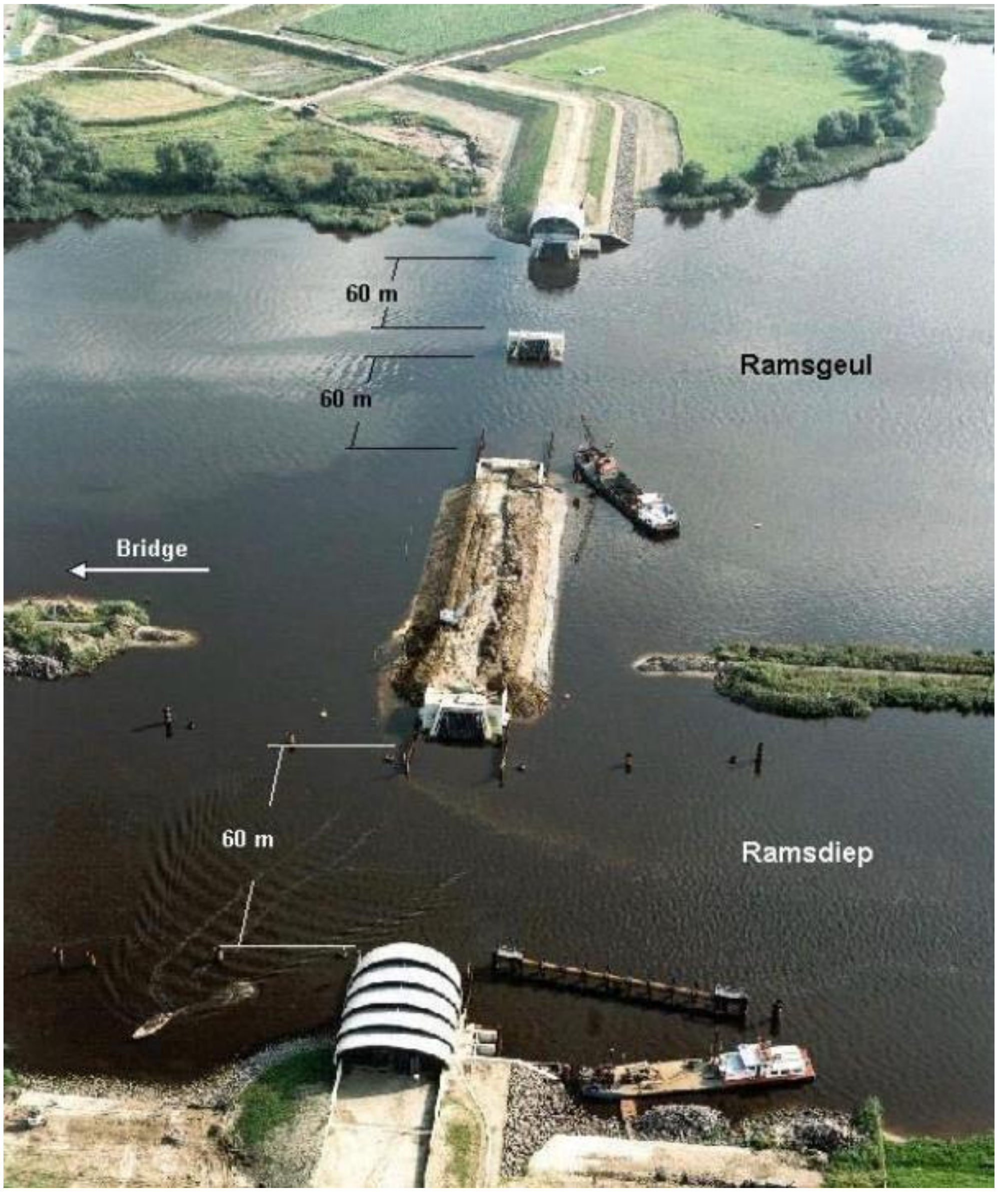


Figure 4-11: Ramspol Barrier, the Netherlands [34]

Rubber Dam at River Lech:

This rubber dam is for the purpose of hydropower generation. There are four sections, one with the width of 26.65 m and height of 3.35 m and the other three ones with the width of 46.67 m and the 1.25 m height. The damming element is from the rubber-textile bag which is filled with water and fixed to concrete structure of the lower frame of the dam. The material is actually the special rubberized fabric which has the long life [35].

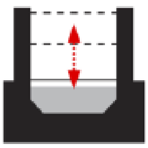
Figure 4-12: Rubber dam at river Lech, Germany [35]

Advantages and Disadvantages

Inflatable Weirs	
Advantages	**Disadvantages**
Maintenance and inspection	Unproven technology in large scales
Functioning in both directions	Replacement of the rubber sheet is difficult
No limitation of opening	Flexible structure, low frequencies, small stiffeness and great mass
Little space required	Not suitable for deep water
Direct transfer of hydraulic loads	Ships or objects collision
No destruction of the landscape	Strong flow contraction in last stage
No subject to wind	Control of immersion and storage of rubber sheet is hard
Unlimited shipping clearance	Internal pressure determines stability
No need for hinges and driving system	Considerable response to wave loads
Vertical closure of the flow opening	No spill of excess water
Not sensitive to silting of sill	Overflow vibrations

Table 4-5: Advantages and Disadvantages Inflatable Weirs

4.2.6 Vertical Lifting Gates



Vertical lifting gates are raised and lowered vertically. They have been widely used and have a records of satisfactory operations [22]. Because of their widely use, one can find lots of documentations and technical information about the construction, operation, functioning and behavior under flow and wave conditions. They may be stored under the water and raised to close the opening when it is needed or stored above a channel on towers and lowered to close the flow [20]. They are applicable for the large spans such as 100 m. Cofferdams can be used for construction of the hoisting towers and the tower foundations. The concrete sill can be floated in and immersed on a gravel base or pile foundation which has been constructed already under the water [22]. The riprap is sometimes used for the protection of the bed adjustment to the sill and towers. Vertical lifting gates are usually designed as wheeled gates instead of sliding gates because then they can be closed easily by gravity forces. Using the steel doors and concrete towers is the most common form. The new techniques for the doors such as using the FRP material is also possible.

Project examples

Eastern Scheldt (Oosterschelde) Storm Surge Barrier:

Maybe the most famous vertical lifting gate barrier in the world is Eastern Scheldt Storm Surge Barrier in the Netherlands. This barrier closes off the Eastern Scheldt Estuary from the North Sea in case of storm surge while it remains open to maintain the environmental conditions in the normal situation. The gates can be closed in case of water level more than NAP+3 m. With the length of almost 9 km it is the largest project of the Delta Works.

The barrier has been constructed by pre-constructed elements which includes concrete piers, concrete sill beams, concrete upper beams, concrete road bridge elements and steel lifting gates. The sliding gates are operated by using the hydraulic cylinders. In the fully lowered situation they close off the opening between the upper and lower beam while the horizontal forces are transferred to the supports in the gate recesses of the piers [22]. Several layers of riprap in combination with asphalt layers protects the sill and adjustment bed. With the lifetime of 200 years, this barrier was the most difficult and most expensive part of the Delta Works which the construction took a decade to be completed (1976 to 1986).

In total there are 62 openings where each opening has the width of 40 m and the steel gates have the width of 42 m. The height of the gates are different between 5.9 m to 11.9 m. The concrete pillars have the height between 35 m and 38.75 m depending on their location. Regarding the design wave condition, wave height up to 5.8 m and wave period up to 10 seconds have been considered. It is worthwhile to mention that the barrier do not allow the shipping but there is a separate shipping lock next to it for the shipping purpose.

Figure 4-13: Eastern Scheldt (Oosterschelde) Storm Surge Barrier, the Netherlands

Hartel Canal Barrier:

Hartel Canal storm surge barrier consists of two lens-shaped vertical lifting gates with spans of 98 m and 49.3 m and the height of 9.3 m. The gate never fully closes to facilitate water storage while at high floods the gates are overtopped [36]. The sliding gates are driven by hydraulic cylinders with a long piston which are hinged to the side towers [22]. The clearance between the mean water level and the gate underside in the open situation is about 14 m while the gates have the distance of around 0.65 m with the concrete sill in the closed situation. This barrier is constructed from 1993 till 1996. There is a shipping lock beside the barrier which is used when the barrier is closed.

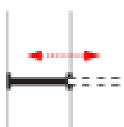
Figure 4-14: Hartel Canal Barrier, The Netherlands [36]

Advantages and Disadvantages

Vertical Lifting Gates	
Advantages	**Disadvantages**
Proven technology	Sensitivity to vibrations
Large opening feasible	Water depth versus gate heights
Little space required	Limmited clearance height for shipping
Raised gates accessible for maintenance	Destruction of the landscape
Controlled operation under flow and wave	Raised gates subject to wind load
Vertical closure	Smooth slide required versus growth underwater
Discharge of excess water	Small stiffness during operation
Overflow and reverse flow acceptable	Subject to down-pull forces and wave loads
Underside free of sill	Wheel gates weak spot, wearing
Limited vertical flow forces and wave loads	Subject to down-pull flow forces and wave loads

Table 4-6: Advantages and Disadvantages Vertical Lifting Gates

4.2.7 Rolling or Trolley Gates



These gates are closure panels which are stored adjacent to the waterway and in case of flood anticipation are rolled into the position. Rolling gates are bottom supported while trolley gates are top supported.

Project examples

Selby Lock Rolling Gate:

As the flood control gate, this gate is stored in a slot at the side of the waterway and is winched across the canal [37]. The flood gate is a fabricated steel structure and has the dimensions of 6.4 m wide, 3.85 m height and 0.35 m depth. The gate is partially buoyant and seals to a timber sill.

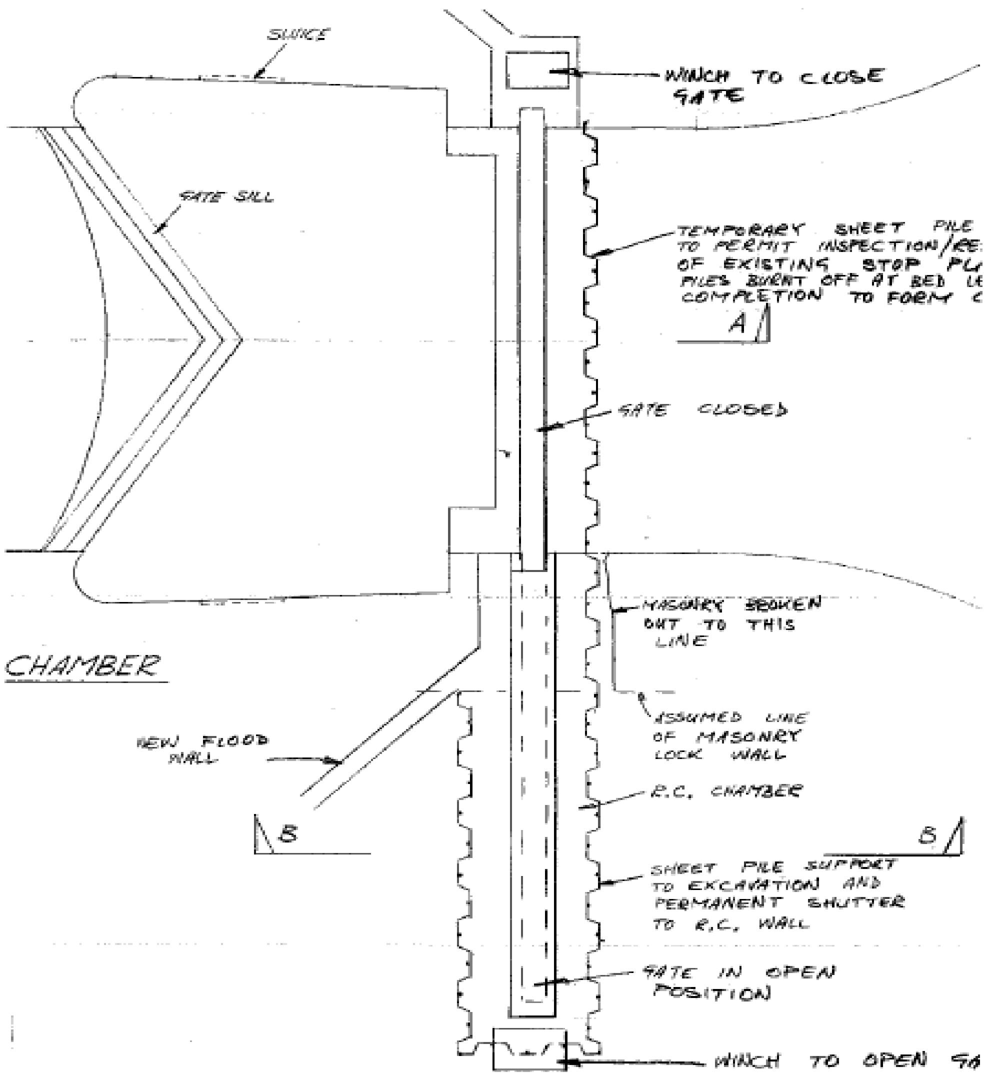


Figure 4-15: Selby lock rolling gate, manufacture's drawing [37]

Berendrecht Flood Control Rolling Gate:

Located near the Antwerp, Belgium, the purpose of these rolling gates is to provide navigation access through a flood control barrier. The four gates are identical and interchangeable with the dimensions of 69.69 m length, around 22.60 m height and the average width of 9 m. The total weight of each gate is about 1,500 tons. The gates can resist water pressure in both directions and are from the wheel barrow type [38]. Each gate rests on a wagon under the water at the front end. At the rear end the door hangs on a wagon that moves on rails which are fixed above the water level on the gate chamber walls. The gates can be moved by the cables fixed on the back wagon.

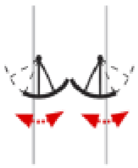
Figure 4-16: Lock doors during construction [38]

Advantages and Disadvantages

Rolling or Trolley Gates	
Advantages	**Disadvantages**
Proven technology	Large space required
Not subjected to wind	Deep excavation required for chambers
Suitable for deep water	Flat and smooth slide way required
Immidiately ready for operation	Silting may hamper operation
Free of sill, reduced load on sill	Large part of construction under water
Stable structure with no load concentration	Limited shipping clearance
Dry dock, maintenance and inspection easy	Sluice gates may be required
Limited differential head and horizontal flow	
Excess water through sluie opening possible	
Suitable for reverse head and flow	
Not sensitive to flow vibrations	
Large opening feasible	
No destruction of the landscape	

Table 4-7: Advantages and Disadvantages Rolling or Trolley Gates

4.2.8 Sector Gates



Sector gates are circular sections supported on a vertical hinge at the centre of a circular arc [20]. There is only the skin plat on the face of the circular arc. There is very little unbalanced load because the hydraulic thrust is directed radially inward toward the vertical axis and thus the gates can be opened and closed with differential head across the gate. In general, sector gates can be divided in two types of non-floating or floating gates.

In the non-floating gates, the gates move over a carriageway or slide away on a sill on the river bed [22]. Side chambers are needed for gates storage when they are not in used. These chambers and abutments with gate supports can be built within cofferdams. Sill can be floated to the site and immersed on gravel base or pile foundation that has been already constructed underwater or built within the cofferdam.

In the floating sector gates, the gates can be stored in a shallow dry dock in the abutments when not in used. The gates can be immersed on the sill even when the sill is covered with silt. The dry docks and the abutments with ball hinges can be built within the cofferdams while the sill can be constructed underwater [22]. The riprap protection might be used for bed adjacent to the sill.

Project examples

Maeslant Storm Surge Barrier (Maeslantkering):

Maeslant Storm Surge Barrier is a floating sector gate in Rotterdam, the Netherlands. It is in operation since 1997 and in November 2007 had its only successful closure under storm conditions. This barrier is also a part of the Delta Works. This flood barrier spans 306 m. The barrier consists of two floating sector gates, each with a radius of 246 m and arch length of 208 m and a gate height of 22 m [22]. The gate arm is connected to a single ball hinge on the abutments. When the forecasted water level for Rotterdam exceeds NAP+3 m, the barrier in the New Waterway has to be closed [39]. The design lifetime of the barrier is 200 years.

When the barrier is not in used, the gates are stored in relatively shallow side docks with a high elevated floor, which can be closed by using the gates and then can be pumped dry. Then the inspection and maintenance is possible in the dry situation. The locomotive engine, which is connected to the abutment by means of a rod that can move along a vertical pile, is used as the driving device for gates. There are floatation boxes which are filled with water after the gates have been floated into the water and then the gates can sink down onto the sill [22].

Figure 4-17: Maeslant Storm Surge Barrier (Maeslantkering), New Waterway, the Netherlands [39]

New Bedford Hurricane Barrier:

Located in Massachusetts, US, this barrier consists of a around 2.8 km dam with a crest level of more than 6.1 m and a navigation opening of around 4.8 m wide which is protected by two sector gates. The sector gates with the height of around 18.3 m are housed in side chambers in the abutments [22]. The gates can be rolled by using the steel wheels on a concrete sill. The closing time of the gate is around 12 minutes.

Figure 4-18: New Bedford Hurricane Barrier, Massachusetts, US

Advantages and Disadvantages

Sector Gates	
Advantages	**Disadvantages**
Large opening possible	Large space required
Unlimited shipping clearance	Complicated operation
Maintenance and inspection in dry dock (Floating case)	A negative differential head may cause problems (Floating case)
Can be immersed if sill is covered with sill (Floating case)	Objects on sill can cause damage (Floating case)
No flatness of sill required (Floating case)	Load concentrations and transfer of forces to hinges
No strong horizontal flow contraction	Ship collision, siltation in open chambers (Non-floating case)
No subjected to wind	Flat and smooth slide way reuired (Non-floating case)
suitable for deep water	Mobilization time; filling of dry docks (Floating case)
	Sensitivity to flow-induced oscillations (Floating case)
	Sensitive to dynamic wave forces (Floating case)

Table 4-8: Advantages and Disadvantages Sector Gates

4.2.9 Stop Logs and Bulkheads

Stop logs and maintenance bulkheads are commonly built with a pair of horizontal trusses supporting a vertical skin place on one face [20]. The gates are stored separately from the opening and lifted into the place by use of an overhead or mobile crane. The gate can consist of one unit from the sill to the top or several smaller units which may be stacked and sealed against one another to close the opening.

Project examples

Kentucky Lock Floating Caisson:

This floating gate is used to dewater lock chambers for the purpose of maintenance [40]. The bulkhead, with the dimensions of 34.3 m width, 9 m height and depth of 3.2 m, is towed from one site to another as a barge. Then it is filled with water in a sequence to rotate it vertically, move into the position and lower it to the final position. It is designed to resist a maximum design head of 9 m and weighs 190 tons as outfitted without water ballast, it has 79.5 tons of permanent concrete ballast.

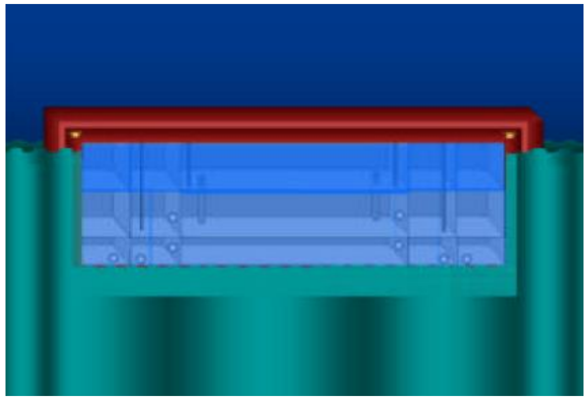


Figure 4-19: Kentucky Lock Floating Caisson, Nashville, Tennessee, US [40]

Olmsted Maintenance Bulkheads

Constructed in 2003 for the purpose of maintenance and dewatering of the locks and radial gates, the four bulkhead sections have been built. Two of the bulkheads have the height of 11.6 m, one 5.5 m and the other one 3.4 m. The bulkheads are framed with open web steel trusses designed to span 34.42 m from center to center of bearing in the bulkhead slots [41].

Figure 4-20: Bulkhead ready for shipment to the site [41]

Advantages and Disadvantages

Stop Logs and Bulkheads	
Advantages	**Disadvantages**
Little space required	Large opening not feasible
Unlimmited clearance height for shipping	Not proven concept for large spans
Not subjected to wind	Not sutiable for deep water
Maintenace and inspection	Little or no controlled operation under flow and waves
	Sensible to vibration, reverse head and waves

Table 4-9: Advantages and Disadvantages Stop Logs and Bulkheads

4.2.10 Swing (Barge) Gates

A swing gate or a barge gate is stored on one side of a waterway and pivots about the vertical axis to close against abutments on either side of the waterway [20]. The gate might be floated to reduce hinge and opening forces. The gate might have the wall openings with valves to keep it permeable when it is closed. The permeability makes the control over the barrier easier during the rotation to the position. In the closure position and when the gate is immersed the valves are closed to make it water retained [11].

Project examples

Bayou Dularge Barge Gate:

This flood barrier is made buoyant and can be floated into the position of opening by winches in advance of a flood. The steel barge gate has the dimension of 7.3 m wide and 19.5 m long while when in position has a height of 6.25 m. It floats with a draft of 0.76 m when empty and is stored at the side of the Channel when not required for flood protection [42]. When the gate is stored at the side of the channel, it will be ballasted to 1.8 m of draft to reduce the ballasting time required when it is swung into place and fully ballasted. The gate is composed of eight compartments for ballasting.

Figure 4-21: Bayou Dularge Barge Gate, Lockport, Louisiana, US [42]

Bayou Lafourche Barge Gate:

This flood barrier has the gate opening width of 22.9 m and the barge gate length of 25 m. The barge gate also has the width of 7.3 m and height of 3 m with a 1.5 m parapet wall on top [43]. The swing barge gate consists of a steel hull with a pivoting swing arm attached to a pivot piling. The barge gate itself will consist of two basic parts: a decked lower hull, 3.05 m deep and a 1.5 m high watertight barrier wall. The lower hull is built with 8 watertight compartments while all of the compartments are accessible from the barge deck via watertight hatches. The swing arm connects the barge gate to the pivot piling and it consists of the pivot assembly and the support arm.

Figure 4-22: Bayou Lafourche Barge Gate in open position, Lockport, Louisiana, US [43]

Advantages and Disadvantages

Swing (Barge) Gates	
Advantages	**Disadvantages**
Little space required	A negative differential head may cause problems
Unlimmited clearance height for shipping	Objects on sill can cause damage
Not subjected to wind	Load concentrations and transfer of forces to hinges
Maintenace and inspection	Sensitivity to flow-induced oscillations (Floating case)
Large opening feasible	Sensitive to dynamic wave forces (Floating case)
Suitable for deep waters	Limited resistance to negative differential head (Floating case)
Can be immersed if sill is covered with sill	

Table 4-10: Advantages and Disadvantages Swing (Barge) Gate

4.2.11 Parachute Barrier

The parachute barrier is in fact an open fabric moveable barrier which unfolds like a parachute in horizontal direction. It is called open fabric because contrary to rubber dams which are classified as closed fabric, only one side of the rubber material is constrained [11]. The barrier consists of a floating body (such as long steel pipe) and an open fabric attached to the bottom. The main principle of this kind of barrier is that it is got open by the water flow and kept open by hydraulic pressure. The presence of the revered head can affect the applicability of this barrier inversely. Parachute barrier has not been constructed yet in the world in the large scale. There is a Master's Thesis by van der Ziel (2009) which has the structural design for a parachute barrier [24].

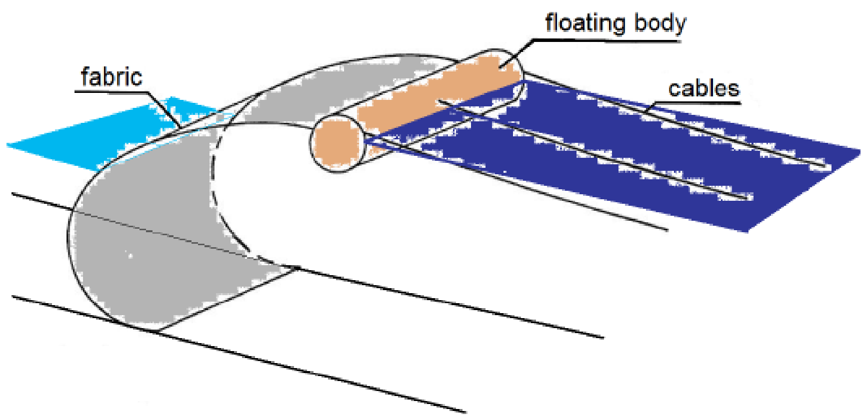


Figure 4-23: Possible open fabric flood barrier [25]

Project examples

Curtain Barriers (Temporary):

The laboratory tests have been done in Canada for temporary barrier structure which is designed for installation in river to provide flood protection [44]. A system feasibility study was carried out between 2000 and 2004 and two relevant projects have recently been executed. The barrier is designed to generate a blockage to the movement of water so it can generate a rise in water level upstream of the structure's location. The barrier consists of several units which are connected together with chains and act to resist the water and generate the desired blockage. Each of the units has two steel pipes joined with three chains, one in the centre and one at each end. The diameter of the top pontoon is dependent on the required water level rise. The ends of the top pontoon are capped to provide buoyancy. A rubber sheet is fixed to the pipes while supported by the chains. A series of these units can be deployed in the river for the project. They are all connected together by using chains and cables. The top pontoon will submerge and the flow will move over top of the barrier.

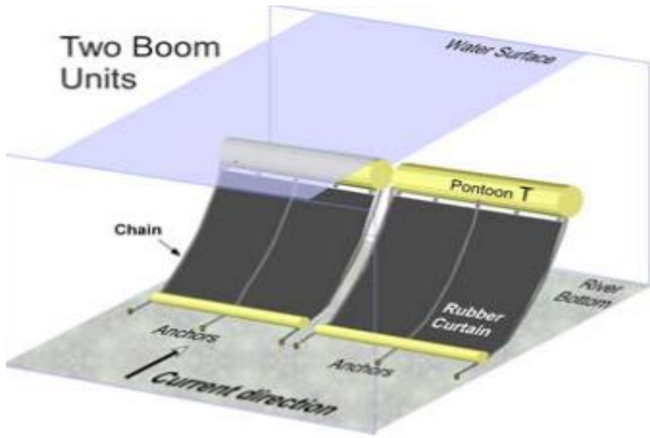


Figure 4-24: Three-dimensional view of two units of the temporary curtain barrier [44]

Advantages and Disadvantages

Parachute Barrier	
Advantages	**Disadvantages**
Little space required	Maintenance and inspection
Large opening feasible	Flexible structure
Unlimmited clearance height for shipping	Not suitable for deep water
Not subjected to wind	Considerable response to wave loads
No destruction of the landscape	No spill of excess water
	Not proven concept
	Sensible to vibration, reverse head and waves

Table 4-11: Advantages and Disadvantages Parachute Barriers

4.2.12 Reduction Barrier

Reduction barrier can offer safety by making additional resistance in the estuary by reducing the amplitude of the tide. The reduction barrier can be described as a dam with openings in it. The water discharge is still possible for inside and outside of the estuary which is beneficial in terms of the environmental issues and shipping traffic. However, the flow velocities through the shipping openings must be acceptable for the passage of vessels [26]. The reduction barrier can be constructed as concrete caissons or rubble mound.

Advantages and Disadvantages

This concept means that no gate should be used for the navigational section. Indeed, the advantages and disadvantages of this type is not discussed here.

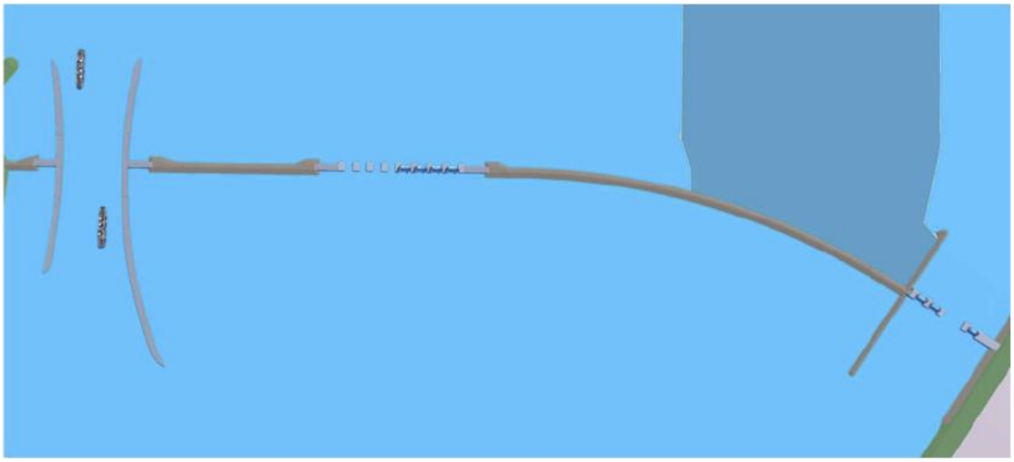


Figure 4-25: Reduction barrier proposed for Western Scheldt, the Netherlands [26]

4.2.13 Mailbox (Hinge) Gate

The mailbox or hinge gate is an innovative concept which consists of a heavy concrete flap gate of 20 to 30 m length hanging on two yokes [8]. The yokes are founded on inclined foundation piles. In this way the soft clay layer with poor bearing capacities is avoided. In normal conditions the flap is positioned horizontally and the flow goes from underneath it. While in high tide situation the floating flap reached the end of the slotted holes and it creates a moment taking the flap to tilt vertically.

This barrier is in fact a leaky system because it is the combination of a top and bottom spillways. High water pressures due to surge or waves push the flaps and open the gate. As it is mentioned, this is a new concept and it is not implemented anywhere yet.

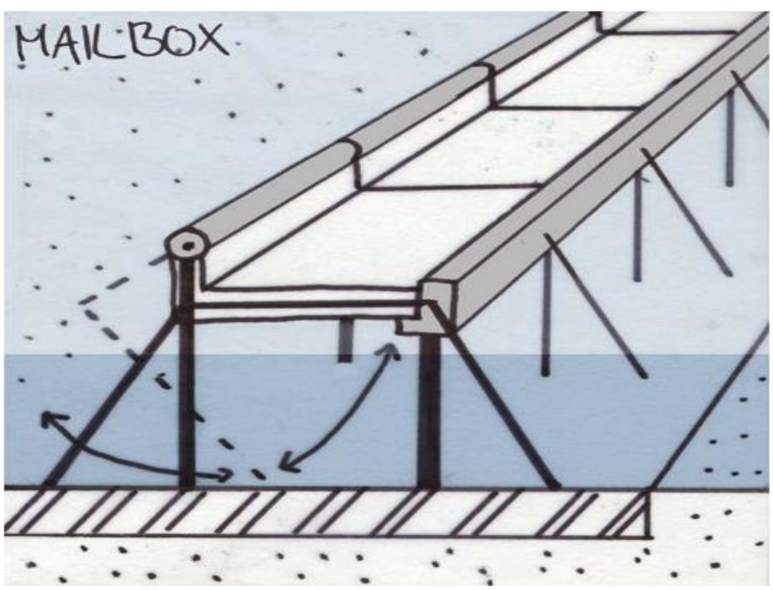


Figure 4-26: Mailbox (Hinge) Gate [27]

Advantages and Disadvantages

Mailbox (Hinge) Gate	
Advantages	**Disadvantages**
Little space required	Maintenance and inspection
Large opening feasible	Flexible structure
Unlimmited clearance height for shipping	Not suitable for deep water
Not subjected to wind	Considerable response to wave loads
	Not proven concept
	Sensible to vibration, reverse head and waves

Table 4-12: Advantages and Disadvantages Mailbox (Hinge) Gates

4.3 Conclusions

In this section different types of navigational surge barriers have been introduced. The characteristics, advantages and disadvantages and some project examples of the gate types have been described. By using the information provided in this section, in the next section the Multi Criteria Analysis (MCA) is going to be used for selection of the best alternative for the Bolivar Roads navigational surge barrier. The investigated gates in this section are summarized in Table 4-13.

Gate Types	
1	Arch or Visor Gate
2	Mitre Gate
3	Vertically Rotating Gates
4	Flap Gates
5	Inflatable Weirs
6	Vertical Lifting Gates
7	Rolling or Trolley Gates
8	Sector Gates
9	Stop Logs and Bulkheads
10	Swing (Barge) Gates
11	Parachute Barrier
12	Reduction Barrier
13	Mailbox (Hinge) Gate

Table 4-13: Summary of gate types

5 Multi Criteria Analysis (MCA) (DS1.)

5.1 Introduction

Selection of the navigational surge barriers has great economic, environmental and other effects on the project regions. That is why the selection procedure is an important aspect of the project. Many people can be affected by the selection of a specific type of barrier in terms of safety of their properties or even nature of their means of income [20].

Next to the general requirements for the selection of a particular gate type, some other aspects play important roles too, such as gate location, waterway navigability, flooding risks, adaptability, water flows, bottom or shore erosion, water ecosystems, local economy and local energy balance. Indeed, the gate type selection can be considered as a combination of engineering, economy, politics or maybe other relevant disciplines. Multi Criteria Analysis (MCA) helps the project team to select the most appropriate gate type by considering different interests and aspects.

In this chapter, firstly different methods of MCA are going to be described. Then the selected method for the current project is going to be depicted in details and finally the best gate alternative for the Bolivar Roads Navigational Surge Barrier is going to be selected for further design. This chapter deals with the design step 1. (Figure 5-1).

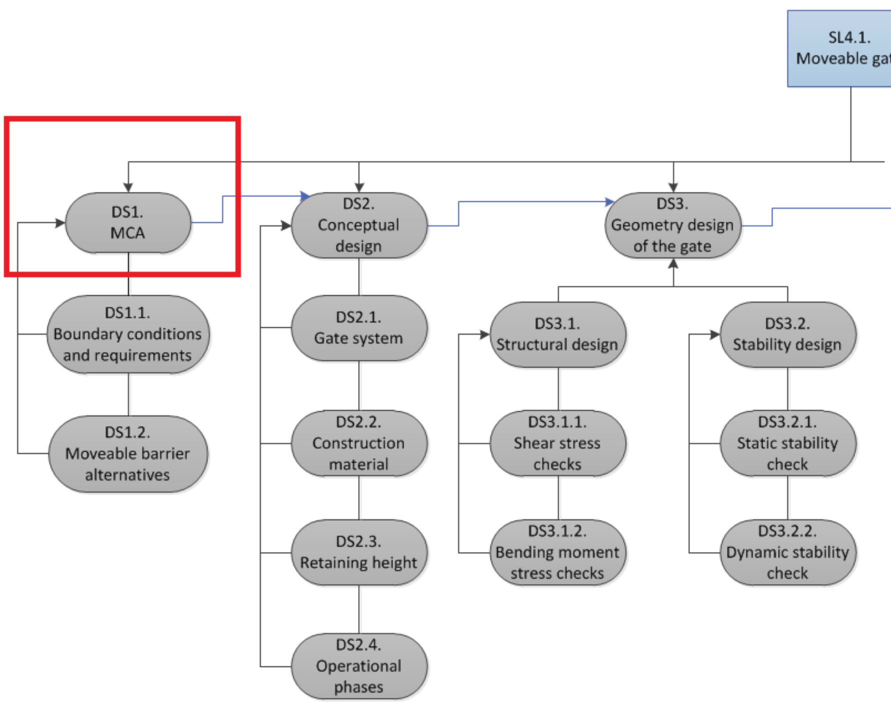


Figure 5-1: Design Step 1.

5.2 Multi Criteria Analysis (MCA) Methods

The main outcome of MCA is a matrix where different options are evaluated by using different criteria (Table 5-1). The main issue is to give values to the criteria within the matrix.

MCA for Gate Type Selection		Gate type options				
		Gate type 1	Gate type 2	Gate type 3	→	Gate type m
Criteria	Criterion 1	Values				
	Criterion 2					
	Criterion 3					
	↓					
	Criterion n					
Total						

Table 5-1: Sample of MCA table

The main questions regarding the MCA for gate type selection are as follow [20]:

1. How and in which units to measure the scores of gate type in each criterion?
2. How to convert these scores to the same units in order to make a total evaluation?

5.2.1 Qualitative Assessment

In this method the simplest solution for answering to the above mentioned questions is chosen which is ignore the questions and using the qualitative descriptions and no quantitative values [20]. This kind of analysis is totally based on subjective judgments of a person or a project team. The considered performance assessment of the gate types is not possible in this method. This method is useful when there is a time or budget pressure, simple projects or the need for educated justification of the already chosen type.

Advantages and disadvantages of this method are summarized in Table 5-2 [47].

MCA with Qualitative Assessment Method	
Advantages	**Disadvantages**
Quick and simple assessment	Arbitrary, entirely dependent on assessor's view Performance assessments unclear, no answers to "why?" and "how much?"
Good for small numbers of options and criteria, pre-selections etc.	Little or not verifiable
Surveyable for deciders, other interest groups, non-professionals etc.	Inconsistent, criteria incomparable
Can be good to present conclusions	Vulnerable to manipulations, lobbying, disinformation etc.

Table 5-2: Qualitative Assessment MCA Advantages and Disadvantages

An example of this method is the gate type assessment for the Dutch navigable waterways, the Meuse-Waal Canal in Heumen. In this project three gate types were considered including two mitre gates and one vertical lift gate.

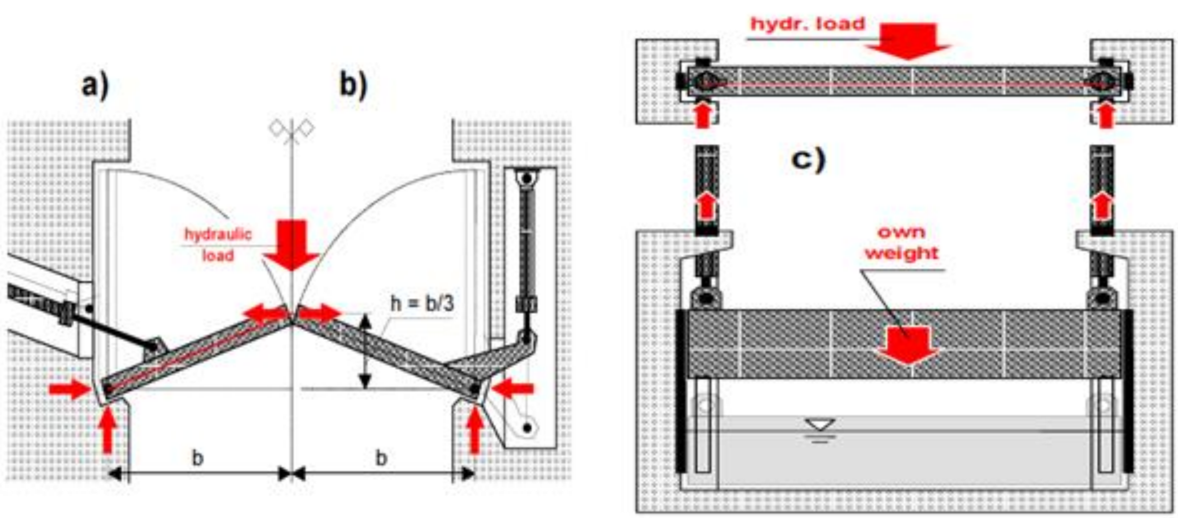


Figure 5-2: Gate types considered as flood barrier on the Meuse-Waal Canal in Heumen, the Netherlands [47]

The MCA matrix for this gate evaluation is shown in figure below. As it can be seen from this matrix, the vertical lift gate has the better evaluation compared to the other gate types. The mitre gates are evaluated to be more expensive and also less reliable in case of the need for emergency closure. The aesthetics and navigation advantages of the mitre gates are not enough to compensate the disadvantages in the other aspects.

Criterion / Option	Mitre gate (a)	Mitre gate (b)	Vert. lift gate (c)
Total costs	-	--	+
Operation	+/-	+/-	++
Navigation	+/-	+	-
Maintenance	+/-	+	+
Environment	+	+	++
Aesthetics	+	+	-
Total	+/-	+/-	+

Herein:
++ very good;
+ good;
+/- fair;
- poor;
-- bad

Table 5-3: Qualitative MCA for Gate type in Heumen, the Netherlands [20]

5.2.2 Quantitative Assessment

For the MCA by using the quantitative assessment two methods of assessments in terms of costs and performance rating with weighting factors are possible.

5.2.2.1 Assessment in Terms of Costs

The gate selection criteria can be quantified in some cases such as for costs and for some other cases such as navigation they are less clearly quantifiable. For some criteria such as aesthetics or environment, it is almost impossible to make them quantifiable. One method for the quantitative assessment is expressing everything in terms of costs (in currency units) [20]. The reason is that the project costs are always one of the most important criteria in gate selection. Also, this criteria is the best quantifiable.

In this method, the values in currency units are given to gate performances in all the other criteria as well as the costs criteria. In this way the both questions from the beginning of this chapter are answered reasonably. Some criteria such as maintenance and operation costs can be easily describe by this way. Advantages and disadvantages of this method are summarized in Table 5-4 [47].

Assessment for a new weir on the Meuse in Sambeek in the Netherlands is an example of this method. For this project vertical life gate with a flap section, sector gate of Thames Barrier type, top-hinged (suspended) flap gate and bottom-hinged flap gate were considered as different possible options (Figure 5-3) [20].

MCA with Quantitative Assessment in Terms of Costs Method	
Advantages	**Disadvantages**
Gate type performances quantifiable, little place for arbitrariness	Not all criteria can be quantified in money (e.g. environment, aesthetics)
Clearly determined measure (money)	Tendency to skip the not-quantifiable criteria
Measure directly related to one of the most substantial criteria - costs	Quantifiable criteria privileged, method not well-balanced
Measure applicable (more or less) to a number of other criteria	Money measure insufficient e.g. for maintenance and operation criteria
Consistency of the method, criteria mutually comparable	Money to be paid by a stronger party (e.g. site owner) counts more
A “no-nonsense” approach, little space for controversial arguments	Defensive, little place for innovations
Prefers well proved technology	Tendency to give everything a price
Assessments to great extend verifiable,little space for manipulations	Vulnerable to instabilities of monetary systems

Table 5-4: Quantitative Assessment in Terms of Costs Method MCA Advantages and Disadvantages

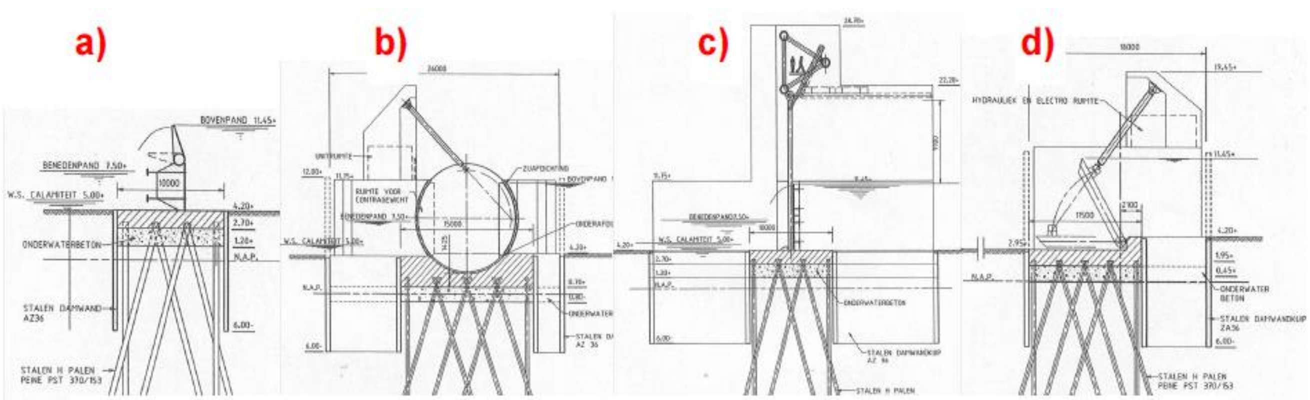


Figure 5-3: Considered gate types for the Meuse weir in Sambeek, the Netherlands [47]

Table 5-5 shows the resulted matrix of the MCA for this project.

Costs (€) / Option	Vertical lift gate (a)		Sector gate (b)		Suspended flap gate (c)		Bottom flap gate (d)	
Construction		36,000,000		37,000,000		34,000,000		32,000,000
Maintenance: - per year - capitalized	340,000	7,596,000	447,000	9,987,000	365,000	8,155,000	421,000	9,406,000
Operation: - per year - capitalized	246,000	5,496,000	246,000	5,496,000	246,000	5,496,000	246,000	5,496,000
Totally (€)		**49,092,000**		**52,483,000**		**47,651,000**		**46,902,000**

Table 5-5: Analysis in terms of costs for the Meuse weir in Sambeek in the Netherlands [20]

For the maintenance and operation costs, a period of n = 50 years was considered with the rate of interest and inflation as i = 4%. The capitalized maintenance and operation costs C_c were calculated from the estimated yearly costs C_y by using the formula below:

$$C_c = C_y.\frac{1-(1+i)^{-n}}{1-(1+i)^{-1}} \qquad \text{Eq. 5-1}$$

5.2.2.2 Performance Rating with Weighting Factors

In this method the measure units from any single criterion is not used and a measuring system is introduced which is applicable to all the criteria [20]. A Rating scale from 0 to 10 can be assumed to quantify gate performances in each single criterion while higher marks represent better performances (the higher the worse is also possible).

For using this method, the following steps should be followed:

- For quantifiable criteria, the gate performances in quality units of a criterion (for example in money for the costs criterion) should be measured. A rating range covering the performances range should be chosen and the measured values should be converted to the rating systems.
- For non-quantifiable criteria, a group of representative of specialists should rate the gate performances subjectively and make a consensus or mean score for them.

In order to produce the total scores, the relative importance of each of the criteria should be assessed by using the weighting factors. In fact, a weighting factor represents the importance of a particular criterion in the analysis in relation to the total of all the criteria. The range of weighting factors from 0 to 1 is the most convenient method while the sum of the factors is equal to 1. The weighting factors is advised to be chosen by a team representing the project initiator (such as local authorities and other parties involved) who act independently from the team of professionals which actually rates the gate performances [20]. Maybe asking a multi-disciplinary team to set up criteria and their weighting factor and a team of specialists to generate solution ideas and to do the rating would be the best method.

Advantages and disadvantages of this method are summarized in figure below [47].

MCA with Quantitative Assessment by Performance Rating with Weighting Factors	
Advantages	**Disadvantages**
Gate type performances quantified in all criteria	Still much space for arbitrariness in not-quantifiable criteria
Weighting factors open a way to a well-balanced solution	Much space for arbitrariness in choosing weighting factors
Method in principle open to any thinkable criteria	Danger of “multiplying” criteria, criteria of little significance etc.
Project-orientated selection of criteria and weighting factors, flexible method	Method flexibility gives way to manipulations
Potentially open to innovations, new materials, technologies etc.	Technical feasibility not emphasized, “surprises” during the project
Works well in democratic societies with emancipated social groups	Little effective in autocratic societies, tensed local relations etc.
Assessments to large extend verifiable, little space for manipulations	Some space for manipulations still present

Table 5-6: Quantitative Assessment by Performance Rating with Weighting Factors advantages and disadvantages

Gate type selection for a storm surge barrier in the Hartel Canal (one of the two waterways to the harbor of Rotterdam) is an example of this method [20]. The project where completed in 1996 where the gate selection took place 4 years earlier. Initially, 40 diverse gate types were proposed in a brainstorming session. Six of these gates were selected finally on the basis of the project requirements and feasibility studies for MCA.

These gate types were (Figure 5-4):

- Vertical lift gate,
- Visor(arc) gate,
- Single rolling gate,
- Double rolling gate,
- Suspended flap gate,
- Turn-over gate.

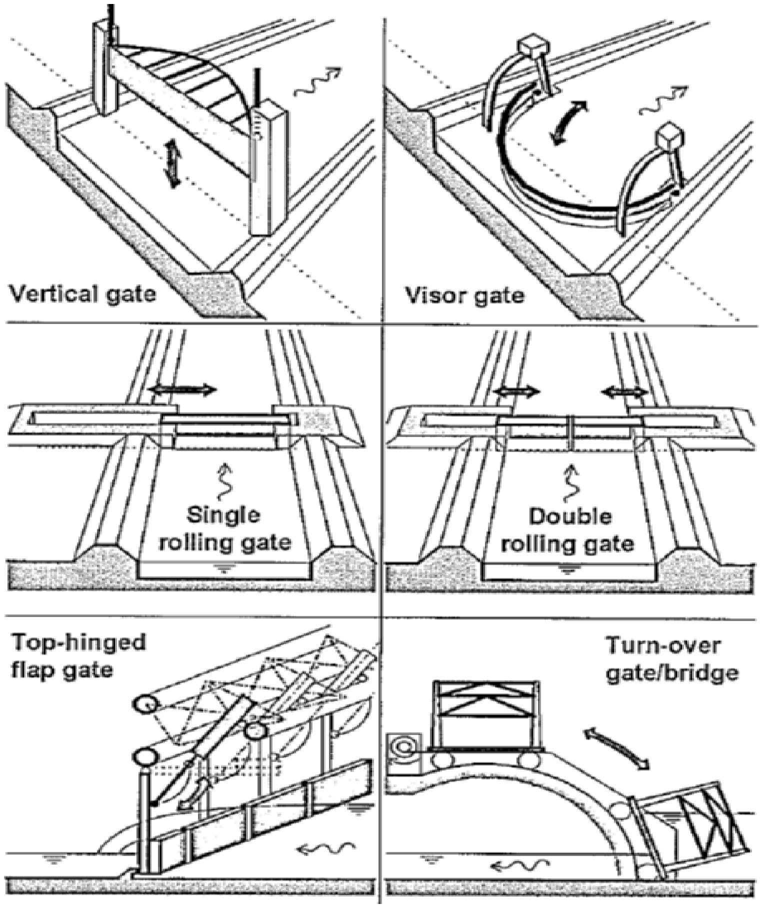


Figure 5-4: Gate options for the Hartel Canal Barrier [20]

The criteria to be consider in the MCA were selected in the brainstorming session including reliability, project control, navigation, local constraints, total costs and realization time. Twenty people of different disciplines were in the team of specialists for MCA. The mean values for the weighting factors were selected because the team couldn't reach a consensus. The MCA matrix of this project can be seen in Table 5-7. The vertical lift gate with highest score in this matrix is operated since 1997.

		Gate type					
Option Criterion	Weighting factor	Vertical lift gate	Visor (arch) gate	Single rolling gate	Double rolling gate	Top-hinged flap	Turn-over gate
1. Reliability	0.27	9.0	8.5	8.0	7.0	6.0	8.0
2. Project control & operation	0.20	8.5	6.0	7.0	6.0	6.0	6.0
3. Navigation	0.19	8.0	7.0	8.0	8.0	8.0	7.0
4. Local constrains	0.12	7.0	8.0	7.0	7.0	6.5	7.0
5. Costs (total)	0.11	9.0	8.0	6.0	6.0	7.5	5.0
6. Realization time	0.11	8.0	6.0	6.0	6.0	6.0	6.0
Total score	1.00	**8.36**	**7.33**	**7.24**	**6.77**	**6.61**	**6.74**

Table 5-7: Gate type assessment by performance rating for the Hartel Canal Barrier [20]

In general the performance rating method is rather vulnerable to arbitrary opinions. It is quite difficult to set up an objective and traceable marking system maybe except the cost criteria and also the choice of weighting factors. Sensitivity Analysis can be used to estimate the influence of this issue in final results. The focus should be on the sensitivity to different assumptions of the criteria weighting factors which is a crucial, final numeric decision to be made [20].

An example of sensitivity analysis is in case of the gate type selection for Naviduct Enkhuizen. In this project four gate types were considered suitable for the project:

- Mite gates
- Single leaf gates
- Rolling (or slide) gates
- Sector gates

The MCA matrix for this project is shown in Table 5-8.

		Gate type			
Criterion / Option	Weighting factor	Mitre gates	Single leaf gates	Rolling Gates	Sector gates
1. Total costs	0.40	8	9	6	6
2. Operation	0.35	9	8	8	7
3. Local constrains	0.10	8	7	8	7
4. Navigation	0.10	8	7	8	6
5. Environment	0.05	7	7	6	7
Total score	1.00	**8.30**	**8.15**	**7.10**	**6.50**

Table 5-8: Gate type assessment by performance rating for the Naviduct Enkhuizen, the Netherlands [20]

Considering the cost criterion, if we are not sure about the current weighting factor (0.40) and want to see what will happen if we change it to 0.10, and divide the difference proportionally between the remaining criteria, the result would be as follow:

- Total Costs: 0.10
- Operation: 0.525
- Local constraints: 0.15
- Navigation: 0.15
- Environment: 0.075

Using the above mentioned weighting factors, other total scores are derived. For each gate type two points coordinates (f_w, s) is present where f_w is the weighting factor for cost criterion and *s* is the total gate score. The linear function defined by these points can be presented in a graph as Figure 5-5.

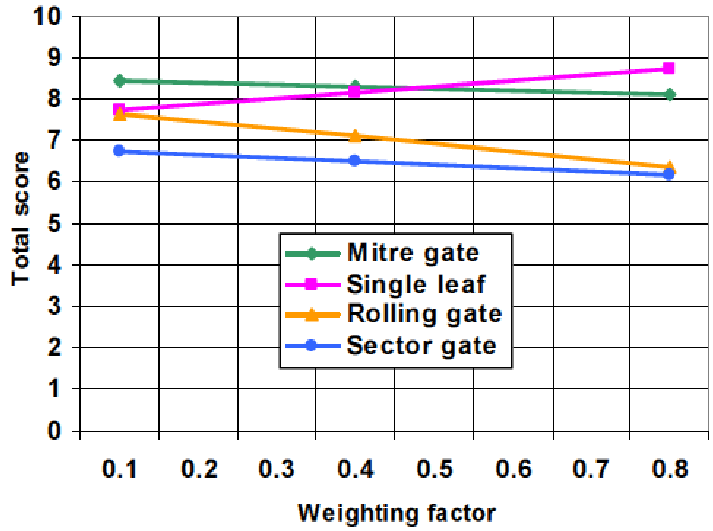


Figure 5-5: Naviduct Enkhuizen, sensitivity analysis for the costs criterion [47]

Mitre gate has the highest score up to the weighting factor of about 0.46 for the costs criterion and then the single leaf is a better option. The other gates are not competitive in this regard.

It is possible to consider two criteria at a time for sensitivity analysis. Then the graph turn to three-dimensional. It is advisable not to focus on sensitivity analysis but to use them as the last tool in gate type assessment.

For each specific project different criteria and weighting factors are required. The example of gate assessment criteria is proposed by PIANC as Table 5-9. The criteria in this table are clustered in a small number of main criteria.

The number of gate types considered for the gate type assessment by using MCA is advised to be small, for example not larger than 4 to 6 [20]. Also, the number of assessment criteria is also advised to be not larger than 6 to 8. Using clustering is also recommended when the number of criteria is large.

Criteria	Weir projects		Barrier projects	
	W.f.	Sub-criteria	W.f.	Sub-criteria
Generalized costs	0.30	Initial costs (engineering, land purchase, construction etc.);	0.15	Initial costs (engineering, land purchase, construction etc.);
		Periodic costs (inspections and maintenance);		Periodic costs (inspections, testing and maintenance);
		Operation costs (personnel, energy, facilities, etc.);		Operation costs (personnel, energy, facilities, etc.);
		Costs of dismantling / modernization after service life;		Costs of dismantling / modernization after service life;
Reliability	0.15	Sensitivity to malfunctions, human errors, ship collisions;	0.25	Failure chance to close, when closed and loaded, to open;
		Vulnerability to foundation distortions, vibrations, bottom erosion, earthquake, etc.;		Vulnerability to foundation distortions, bottom erosion, earthquake, etc.;
		Vulnerability to sediments, ice, debris, algae etc.;		Sensitivity to malfunctions, human errors, ship collisions;
Operation	0.15	Capacity and accuracy of river control in all seasons, operation vulnerability to calamities;	0.15	Convenience and clarity of procedures, especially under extreme conditions;
		Convenience of operation, procedure clarity;		Unavailability for operation due to maintenance;
		Unavailability for operation due to maintenance;		Construction time, especially in reconstruction projects;
		Construction time, especially in reconstruction projects;		Sensitivity to technological aging, patented technology etc.
Navigation	0.10	Construction impact on navigation conditions;	0.15	Free navigation width, overhead space and depth;
		Maintenance impact on navigation conditions		Clarity of navigation regulations during closing and opening;
		Navigation safety and convenience (distances, currents etc.)		Construction impact on navigation conditions;
		Disturbances to maneuvering, radar signals etc.;		Maintenance impact on navigation conditions;
Maintenance	0.05	Maintainability (not in terms of costs!) of all areas and details	0.05	Compliance with ban on maintenance in stormy seasons;
		Access to maintenance sensible components		Maintainability (not in terms of costs!) of all areas and details
		Maintainability under operation conditions		Access to maintenance sensible components
		Health and safety of maintenance crews		Health and safety of maintenance crews
Environment	0.15	Operation impact on eco-system (vegetation, wide life etc.);	0.10	Required area, construction impact on eco-systems;
		Environmental "footprint" of materials (pollutions, energy consumption);		Environmental "footprint" of materials (pollutions, energy consumption);
		Environmental impact of gate construction and maintenance (e.g. painting, lubrication);		Residual environmental impact of storm surge passage;
		Possibility of winning "clean" (water) energy;		Environmental impact of gate maintenance (e.g. painting, lubrication);
Social impacts	0.10	Aesthetics, harmony with landscape, local culture etc.;	0.15	Aesthetics, harmony with landscape, local culture etc.;
		Daily impact on local community (jobs, economy, transport, agriculture, social contacts);		Daily impact on local community (economy, transport, agriculture, social contacts);
		Noise (water flow, machineries, maintenance vessels, etc.)		General image, feeling of safety for the local community;
		Tourism, sport and recreation benefit, science and technology popularization effect;		Tourism, sport and recreation benefit, science and technology popularization effect;

Table 5-9: Indication of gate assessment criteria for weir and barrier projects [20]

5.3 MCA of Navigational Gates for the Bolivar Roads

5.3.1 Method Selection and Description

On the basis of the information provided in the previous sections, for gate type selection of the Bolivar Roads barrier, the MCA by using the quantitative assessment with the performance rating and weighting factors is found to be the most suitable method. By using this method different aspects can be considered in gate type evaluation.

Firstly, in the initial assessment phase, the most appropriate gate types according to project needs are going to be selected from the 13 gate options mentioned in chapter before. Then MCA is going to be done for the selected gate types.

The criteria for MCA are going to be chosen on the basis of the different important aspects of the project by using the specialists knowledge and opinions through the discussions and personal meetings. Then the scoring of the criteria and weighting factors can be also derived through the same procedure and on the basis of available information from the desktop research.

The outcome of the MCA would be a matrix which shows the gate types with their total evaluation scores. The gate type with the highest score is going to be chosen as the most suitable one for the Bolivar Roads for further design.

5.3.2 Gate Type Options Argumentations

According to the navigational needs of the Bolivar Roads Pass, the New Panamax vessels should be able to navigate from the barrier without any obstacle. The air draft of at least 57.91 m is required (See Section 3.3.2).

The *vertical lifting gates* are reliable and feasible if there is no height requirements otherwise they are less attractive. Up to free clearance of 30 m, vertical lifting gates can be very attractive [22]. Also, the span of vertical lifting gate is limited to about 100 m. For these two reasons vertical lifting gates are not really suitable for this project. For the same reasons, *arch or visor gates* and *vertically rotating gates* are not attractive solutions and are eliminated from the options.

Mitre gates are mainly used as gates in the shipping locks where waves and currents are limited and the navigation width is usually less than 50 m. In the severe conditions, such as the current project, with strong tidal currents and high waves, mitre gates are not suitable. Furthermore, the span limitation of the mitre gates makes them not appropriate for Bolivar Roads barrier. *Stop logs and bulkheads* share the same disadvantages of the mitre gates and that is why they are not proposed for the current project too.

The design of a *Reduction Barrier* can be suitable for the entire span of the Bolivar Rods Pass; when the environmental part and the navigational part are considered together. For the purpose of this project, which is focused on the navigational part of the barrier, the reduction barrier is not going to be considered in MCA.

Rolling or trolley gate has the problem of large space required for it and also the opening is too large for this kind of gate. They have been also mostly used for the lock gates and not for the storm surge barriers.

The *mailbox (hinge) gate* is a new innovative concept which there is not enough knowledge and experience about it. It is not the proven technology and it is not considered for further analysis in this report.

To sum up, there are five gate types left for further assessment through MCA in later steps as follow:

1. Sector gates
2. Flap gates
3. Swing (barge) gates
4. Inflatable weirs
5. Parachute barrier

In the next sections, firstly the criteria for the evaluation are going to be selected and then the scoring of the criteria should be done for reaching the final decision about the most suitable gate type for the navigational barrier.

5.3.3 Criteria

Choosing the criteria for MCA is an important aspect which needs the specialist's opinions and experience. As it is mentioned before, the number of main criteria should be tried to be small to provide the opportunity for the appropriate evaluation. In selection of the criteria for MCA the special attention should be given to the project needs and requirements. The selected criteria and their descriptions are provided in the following sections.

5.3.3.1 Total Costs

Total costs of the gate type including the construction costs, operation and maintenance costs is the important criteria which should be considered. The information regarding the actual data for each gate type is sometimes hard to achieve but with respect to previous projects and estimations the evaluation can be done.

5.3.3.2 Maintenance Aspects

Maintenance of the gate is the other selected criterion. Access and simplicity of maintenance for sensible parts, compliance with ban on maintenance in stormy season, safety of the maintenance crew and maintainability of all areas and details are the aspects which should be considered for evaluation of this criterion.

5.3.3.3 Reliability and Operation Aspects

Operation of the gate is the other criterion for MCA. The operational procedures should be simple and clear especially under the extreme conditions. Indeed, operational comfort is important. Also, it should be possible to close the barrier within the short time. Reliability of the gate is the other important issue. The failure chance to close or when it is closed and loaded to open the gate should be considered. Also, vulnerability to foundation distortions, bottom erosion and sensitivity to malfunctions, human errors and sediments and stability under hydraulic loads from both directions are the other important issues in this regard.

5.3.3.4 Local Constraints

Local constraints such as aesthetics, required space for the gate, foundation characteristics needed for the gate and construction nuisance are the important issues here.

5.3.3.5 Realization Aspects

The realization aspects such as realization time, simplicity of construction, proven technology and experience from the previous projects, project control and risks in design process and project execution are the other issues which should be considered in this criterion.

5.3.4 Weighting Factors

Selection of the weighting factors for the above mentioned criteria is the next step in the MCA. On the basis of the project requirements and needs and the interests of the stakeholders, the weighting factors can be different. In fact, the selection of the weighting factors can be still considered as a subjective judgments. The weighting factors are from 0 to 1 where the higher score means a more important criterion. The total sum of the weighting factors should be equal to one.

For the navigational gate of the Bolivar Roads Pass, the weighting factors for each criteria are proposed as below:

1- W_1 (weighting factor of the total costs criterion):

Total costs of the project plays an important role in the decision making for the selection of the gate type. Reduction of the cost would provide the possibility for enhancing the construction procedures and reducing the construction time because the needed funding can be provided more easily and faster. Also, the utilization of the financial resources would be achieved by choosing the most economical solution. For these reasons W_1 is selected equal to 0.25 (*W_1 = 0.25).*

2- W_2 (weighting factor of the maintenance aspects criterion):

The gate should be maintainable during its lifetime. In case of possible failure of the different parts of the gate, the gate should fulfill the requirements for maintainability otherwise the purpose of the project which is reduction of the storm surge cannot be achieved. The weighting factor of 0.20 is assumed for the maintenance aspects (*W_2 = 0.20)*.

3- W_3 (weighting factor of the reliability and operation aspects criterion):

The reliability and operation aspects of the project can be considered as the most important criterion. The gate should be reliable and stable in any circumstance to be able to provide certain level of prevention during the storm condition. Also, the simplicity in the operation offers more safety level for the gate and decrease the chance of failure due to operational errors. Compared to the price, maybe with small increase in the cost of the project the better reliability can be achieved and that is why the weighting factor for the reliability should be higher. The weighting factor for the reliability aspects is proposed as equal to 0.35 (*W_3 = 0.35).*

4- W_4 (weighting factor of the local constraints criterion):

The local constraints aspects has less importance compared to above mentioned criteria. The gate design can be adjusted on the basis of the local constraints and it doesn't impact the gate selection significantly. The weighting factor of 0.10 is considered for this criterion. *(W_4 = 0.10).*

5- W_5 (weighting factor of the realization aspects criterion):

The realization aspects can be considered as important as local constraints. If the project takes more time it can be acceptable to some extends and also the construction difficulties can be accepted more easily in favor of the other weighted criteria. The weighting factor equal to 0.10 is assumed here (*W_5= 0.10).*

To summarize the above mentioned descriptions, the weighting factors are as below:

- *W_1 (Total costs) = 0.25*
- *W_2 (Maintenance aspects) = 0.20*
- *W_3 (Reliability and operation aspects) = 0.35*
- *W_4 (Local constraints) = 0.10*
- *W_5 (Realization aspects) = 0.10*

The above mentioned weighting factors are open for discussions. It has been tried to consider the weighting factors on the basis of the importance in terms of the project needs and requirements and also in consistency with the proposed weighting factors in the guideline provided by PIANC which has been shown in Table 5-9 [20] and reference projects.

5.3.5 Evaluation of the Gate Types

In the previous sections five gate types have been selected for the MCA. In this section, these gates are going to be evaluated for different criteria on the basis of the scoring. In the evaluation, different gate characteristics, advantages and disadvantages, should be considered. Also, the scoring of the gates should be done considering the comparison between the gate types. In fact, if a gate type is more desirable in one criterion compared to the other gate, it should score more for that criterion.

Five scoring levels have been defined for the evaluation:

- 1 = Not favorable
- 2 = Below average
- 3 = Average
- 4 = Above average
- 5 = Favorable

In the next parts, the evaluation is going to be done for each criterion separately.

5.3.5.1 Total Costs Criterion Evaluation

Evaluation of the gate types on the basis of costs is difficult if there is not detailed design for the specific project. Storm surge barriers are tailor-made to local situation and project requirements. The construction techniques, access to construction materials and labor costs are different for each project and each country [54]. That is why no actual cost estimate is available. However, for the purpose of the current MCA, from the project examples which have been already realized for different gate types, a rough estimation of the costs of each gate type can be assumed. Table 5-10 summarizes the cost information of some storm surge barriers in the world [22, 54, 20]. The prices are translated to the year 2014 by using the inflation information [56].

Gate Type	Barrier Name	Year	Country	Length (m)	Height (m)	Retaining Height (m)	Approximate Construction costs in 2014 (Million US$)	Costs per m3 (thousands US$)
Flap Gate	Venice Storm Surge Barrier	Not yet complete	Italy	1600	20	3	3.100	32
	Lagan Weir	1994	Northern Ireland	100	4,5	3	37	27
Inflatable Gate	Ramspol Barrier	2002	The Netherlands	240	8,2	4,4	82	9
Sector Gate	Maeslant Barrier	1997	The Netherlands	360	22	5	757	19
	Seabrook Barrier	2010	The United States	130	8	4	169	41
	IHNC Barrier	2010	The United States	250	12	6	764	42
Swing (Barge) Gate	Bayou Lafourche Saltwater Control Structure	1996	The United States	25	3	2,9	2,8	13
	Bayou Dularge Floodgate	1996	The United States	19,5	3,7	2,9	2	10

Table 5-10: Cost estimations of different gate types

The information about the parachute barriers is not available because they are not realized yet. The maintenance and operation costs of the project can be estimated as roughly 5% of the construction costs [54].

On the basis of information provided above, the sector gates can be estimated as most costly option. This is also obvious from their complicated systems. The flap gates comes next with average costs. The swing or barge gates have the above average cost rates and they are cheaper than flap gates and sector gates. The inflatable barriers [53] and parachute barriers [24] can be evaluated as the most economic options. Therefore, the scorings for the gate types regarding the total costs criterion are as below:

- Sector gate = 2 (Below average)
- Flap gate = 3 (Average)
- Swing or barge gate = 4 (Above average)
- Inflatable barrier = 5 (Favorable)
- Parachute barrier = 5 (Favorable)

5.3.5.2 Maintenance Aspects Criterion Evaluation:

The flap gates has the disadvantage of being under the water which makes the inspection and maintenance hard [22]. Due to this fact the possibility of corrosion is high which makes the needs for maintenance more [20]. Thus, the score for the flap gates is assumed to be 1, not favorable.

In case of the inflatable barrier, the sheet is mostly made from the durable rubber reinforced with synthetics fibers and the corrosion is almost impossible [53]. Also, the moving components are above the water level. Therefore, little maintenance is needed [22] and the criterion can be evaluated as above average because there are still some maintenance problems. The parachute barrier has the same characteristics and gets the same score.

In case of the sector gate the maintenance situation can be scored as above average and equal to 4. The maintenance of this kind of gates is not as difficult as in the case of flap gates. In these gate types, the hinges can be located in the dry dock which makes the maintenance easier than underwater. The barge gate has the average maintenance performance.

In short:

- Sector gate = 4 (Above average)
- Flap gate = 1 (Not favorable)
- Swing or barge gate = 3 (Average)
- Inflatable barrier = 4 (Above average)
- Parachute barrier = 4 (Above average)

5.3.5.3 Reliability and Operation Aspects Criterion Evaluation

Firstly, the sector gate is evaluated for this aspect. The sector gates have the most complicated operational systems. For example, in case of the Maeslant barrier deficiencies in the operating systems occurred. Many of the safety tests were conducted by using the operating computer systems. During the closure test of the barrier, operating software proved to be unreliable [57]. In terms of the closing time, for example in case of the Maeslant barrier, it takes 1.5 hours that the gates can be floated into the river and immersed [22]. If the sector gate is a floating type, the negative differential head may cause some problems for the stability, the possible sediments and objects may cause damage and make the closure procedure difficult and the gate would be sensitive to flow-induced oscillations.

Regarding the flap gates, the flap gates are sensitive to vibrations. Corrosion and marine growth is probable because the gate is located under the water which may cause problems for reliability and operation of the gate. The hinges of the gate may also wear out in sand. The gate has small stiffness during the operation and it may be subject of the wave loads. In case of the pneumatic gate, the gate cannot function in both flow directions. Also, the accumulation of the sand behind the gate might cause problems in the operation. In general, the gate is more vulnerable for sediments because it is located under the water. However, the speed of operation is rapid in this case. The operation procedure is more simple compared to the sector gates.

Considering the swing or barge gate, if the gate is not designed well the negative hydraulic head may cause some problems. The possible objects and sediments on the sill may cause difficulties in the operation procedure. There is a load concentration and transfer of forces to the hinges which is a disadvantage. In case of the floating swing gate, the gate is sensitive to flow-induced oscillations and to dynamic wave forces. The operation of the gate is rather simple with moderate closing and opening time. The barge gate in case of the simple design can be a reliable structure.

Inflatable barriers are flexible structures with low frequencies, small stiffness and great mass which affect the reliability of this gate. They are vulnerable to ship or object collisions. In the operation phase, control of the storage and immersion of the rubber sheets is hard. The length of the barrier parts should be small to increase the reliability of the gate. As an example, for the Ramspol barrier a reliability test and risk analysis has been performed. The failure of systems especially in case of the software, electrical and mechanical failure of the barrier have been found as important issues [53]. The speed of operation in this case is site specific but it is basically slow. The gate can be also subjected to damage from floating materials.

The information about the parachute barrier is not available vastly because it is not executed yet. From the results of theoretical analysis and laboratory tests, it can be mentioned that the gate is a flexible structure with considerable response to the wave loads which reduce the reliability. The parachute barrier is sensitive to the vibrations and not reliable for the reverse hydraulic heads and waves. The operation of the gate can be rather fast and simple.

For evaluation of the reliability and operation criterion, the different gate types should be compared to each other on the basis of the above mentioned information and also in the gate descriptions in the previous sections. By looking at this information, the performance of the flap gates and inflatable barriers can be evaluated as below average. The sector gate has the average evaluation while the swing or barge gate has the above average performance. The parachute barrier can be evaluated as not favorable.

The scoring for this criterion is then as follow:

- Sector gate = 3 (Average)
- Flap gate = 2 (Below average)
- Swing or barge gate = 4 (Above average)
- Inflatable barrier = 2 (Below average)
- Parachute barrier = 1 (Not favorable)

5.3.5.4 Local Constraints Criterion Evaluation

The sector gate requires too much space for the storage for example in the docks. Also, sector gate needs a foundation with the high strengths because it transfers large forces to subsoil while in case of the Bolivar Roads the foundation is not so strong. The construction nuisance of the sector gate can be also a problem because of the needs for the dry dock constructions and side abutments.

The swing or barge gate can be evaluated better compared to the sector gate in terms of the above mentioned issues. It requires less space and the less strong foundation is also acceptable. The space for storage recesses is less than in case of the sector gate.

The flap gates, inflatable barriers and parachute barriers have the advantage in terms of the aesthetics because they are completely invisible when not in use. The forces transferred to the foundation are not so large like in case of the sector gate and they require less space and less construction nuisance.

On the basis of this information, the evaluation can be done as follow:

- Sector gate = 3 (Average)
- Flap gate = 5 (Favorable)
- Swing or barge gate = 4 (Above average)
- Inflatable barrier = 5 (Favorable)
- Parachute barrier = 5 (Favorable)

5.3.5.5 Realization Aspects Criterion Evaluation

The sector gates have been executed already in some places and the knowledge of construction for them is available. However, they have the complicated design and construction procedure which increase the construction time and risks of errors in the design and realization. The project control is also hard in this case.

The flap gates are rather simple for construction and they take moderate construction time and efforts. They haven't been widely executed and maybe the unexpected and unknown challenges will come up during the design or realization of the gate. This will increase the risks of possible errors and delays for the project.

The swing or barge gates have rather the moderate construction and design difficulties. The project control can be easier than sector gates and construction time can be also less than sector gates but they are not executed for the large openings yet and the unexpected problems are possible.

The knowledge and expertise for the realization of the inflatable barriers and parachute barriers is not available for the storm surge barriers because they are not realized yet. They are actually unproven technologies for storm surge barriers. However, it can be expected that the project control is easier in comparison with sector gates and also the construction time is less. In case of the parachute barrier the execution problems and technology is even more unknown compared to the inflatable barriers.

According to the above mentioned issues, the evaluation for the realization aspects are as below:

- Sector gate = 3 (Average)
- Flap gate = 3 (Average)
- Swing or barge gate = 3 (Average)
- Inflatable barrier = 2 (Below average)
- Parachute barrier = 1 (Not favorable)

5.3.6 MCA Matrix and Results

On the basis of the information provided in the previous sections, the MCA matrix is shown Table 5-11.

Code	Criteria	Weighting factor	Gate Types				
			1. Sector gate	2. Flap gate	3. Swing (barge) gate	4. Inflatable barrier	5. Parachute Barrier
1	Total costs	0,25	2	3	4	5	5
2	Maintenance aspects	0,20	4	1	3	4	4
3	Reliability and operation aspects	0,35	3	2	4	2	1
4	Local constraints	0,10	3	5	4	5	5
5	Realization aspects	0,10	3	3	3	2	1
Total Score		**1,00**	**2,95**	**2,45**	**3,7**	**3,45**	**3,00**

Table 5-11: MCA matrix for navigational barrier selection

The MCA matrix shows that the swing or barge gate is the most appropriate gate type for the navigational barrier of the Bolivar Roads Pass. The inflatable barrier is the other option which is on the second place with small difference. It can be expected that with small changes in the weighting factors or the evaluations, inflatable barrier gets the better score than the barge gate. However, in this report barge gate is going to be further analyzed.

5.4 Conclusions

In this chapter the MCA has been done for the selection of the best option for the navigational storm surge barrier of the Bolivar Roads Pass. The results of the analysis have shown that the barge gate is the best option for the current project. The barge gate will satisfy the requirements of the project including the large opening space of 220 m and also it is no restriction for the air draft. Also, by making the barrier as floating, the minimum loads will be transferred to the weak subsoil of the structure which would be important from engineering and economical point of view. The barge gate is a reliable structure and it needs reasonable maintenance during its lifetime.

The status of the requirements for the navigational barrier can be checked after selection of the barge gate in the first step of the design which is shown in Table 5-12.

Program of Requirements for the Navigational Barrier (After DS1)		
Type	**Criteria**	**Status after DS1.**
General	Open in normal conditions	Expected to be ok
	Closed during the hurricane conditions	Expected to be ok
	Realizeable solution	Unknown
	Operable solution	Unknown
	Reliable solution	Unknown
	Economical solution	Unknown
Navigational	Enough width	Expected to be ok
	Enough depth	Expected to be ok
	Enough air draft	Expected to be ok
Current velocity	Acceptable current velocity in navigational channel	Expected to be ok
Safety level	Enough retaining height	Unknown
Lifetime	Enough lifetime	Unknown
	Durable solution	Unknown
Environmental	Environmentaly friendly solution	Expected to be ok

Table 5-12: Status of requirements after DS1

As it can be seen, control of requirements shows that some of the requirements are expected to be ok after DS1.. Indeed, the design can go further to check the requirements with unknown status in the next design steps.

6 Conceptual Design (DS2.)

6.1 Introduction

In this chapter the conceptual design of the navigational barrier is going to be described. This chapter deals with the integral design step 2 and its sub-steps (Figure 6-1).

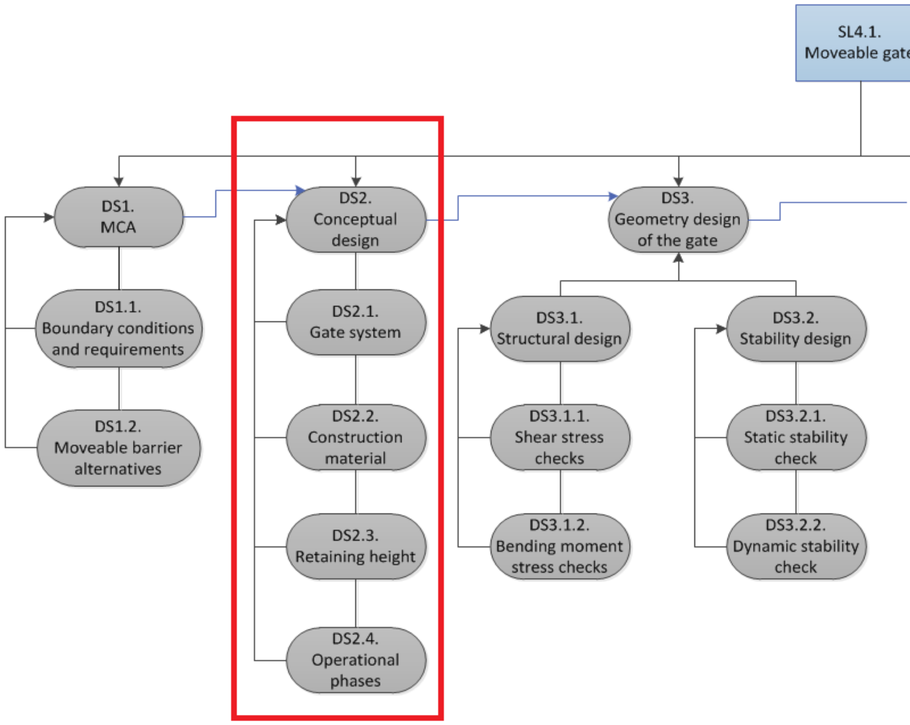


Figure 6-1: Design step 2; Conceptual design and its sub-steps

6.2 Gate System (DS2.1.)

For the navigational gate, in the previous chapter the barge gate is chosen as the best alternative which may fulfill the desired requirements of the project. The barge gate is going to be considered as the floating structure. This is beneficial for the financial issues of the project because it avoids the high forces to the weak foundation of the site and there is no need for a sill with the expensive possible foundation. The main concept of the gate is derived from the tender proposal for the Measlantkering [67]. Figure 6-2 to Figure 6-6 show the floating barge gate barrier in plan and cross-sections.

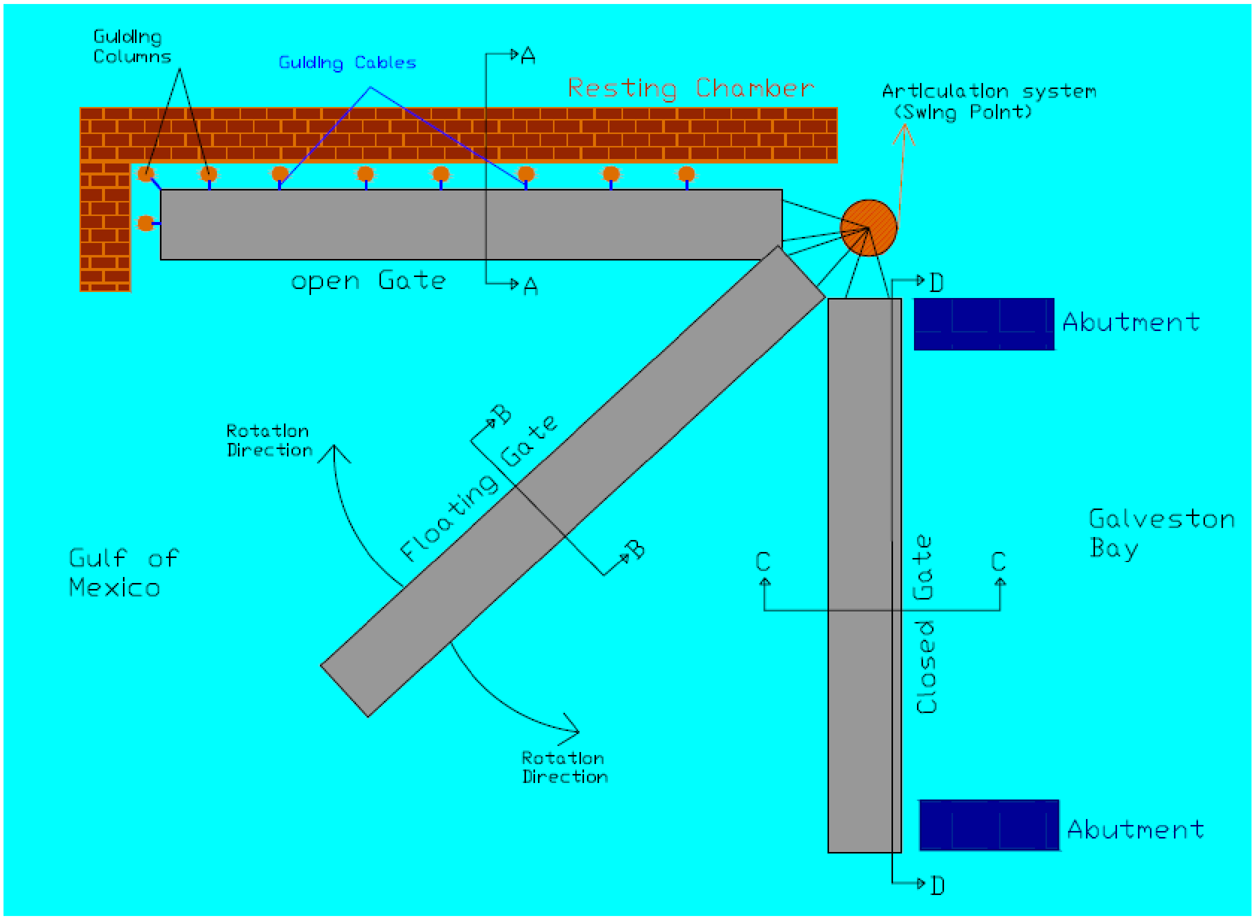


Figure 6-2: Plan of the barge gate (dimensions not to scale)

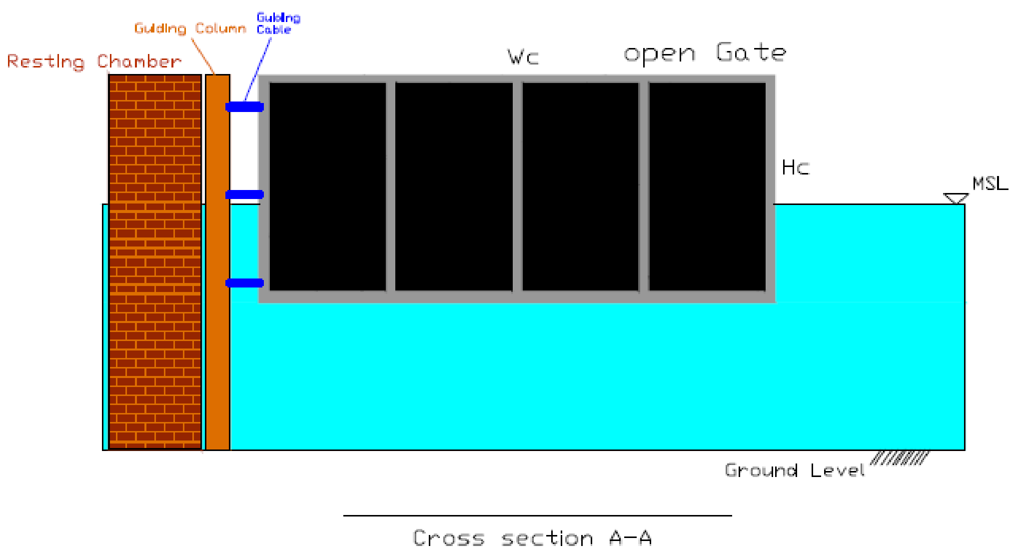


Figure 6-3: Open gate, cross section A-A, dimensions not to scale

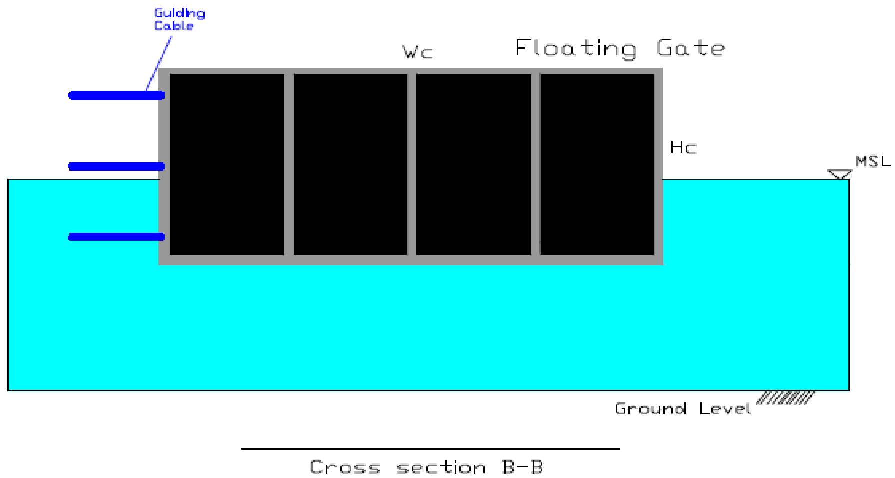


Figure 6-4: Floating gate during the closure, cross section B-B, dimensions not to scale

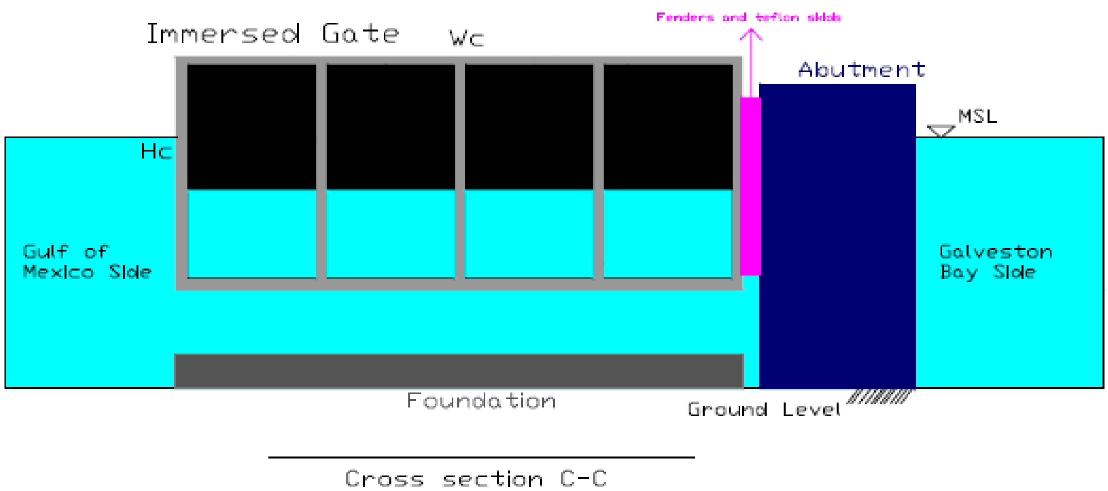


Figure 6-5: Ballasted gate during immersion, cross section C-C, dimensions not to scale

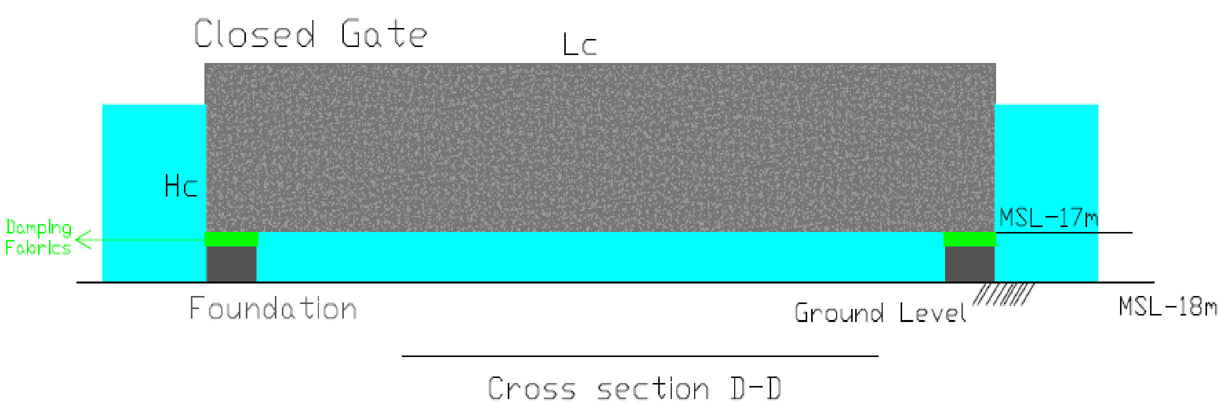


Figure 6-6: Closed gate, cross section D-D, dimensions not to scale

The main components of the barrier are as follow:

- ***Main structure of the gate*** which is a concrete caisson structure for the water retention function. The caisson is composed of watertight external walls, floor and roof slabs and internal walls (shown with grey color in Figure 6-2). The cross-section of this structure is shown in Figure 6-3. (See sections 7 and 10.4 for more information about the design of this structure.)
- ***Articulation system (swing point),*** which the barge gate rotates around it during the closure phase (is shown in Figure 6-2 with large orange circle). The articulation system should provide free degrees of freedom in all the directions except surge and sway. Particularly, at the final position the gate should be able to sink which needs the free degree of freedom in heave direction. (See sections 0 and 10.6.1 for more information about the conceptual design of this structure.)
- ***Two abutments*** which the gate rests against them horizontally at the closed situation (shown with dark blue color in Figure 6-2 and Figure 6-3). The abutments should be able to withstand the horizontal forces from the gate and transfer them to the foundations. (See section 11.2 for more information about the design of these structures.)
- ***Two foundations*** located on the ground at the end sides of the barge gate and near the abutments which gate rests on them vertically at the closed situation (Shown with dark grey color in Figure 6-6). These foundations could be shallow foundations. Because of the weak subsoil capacity of the project location, deep foundations may be used to transfer the loads to the deep strong soil layers. (These structures are not designed in this report, see section 11.3 for more information.)
- ***Guiding columns*** (shown with small orange circles in Figure 6-2) and ***spring cables*** (shown with dark blue lines in Figure 6-2) which provides the guidance during the operation of the floating gate. The uniformly extended spring-cable allows to keep continuously the gate in close contact with the columns acting like a guide. (Not treated in this report.)
- ***System of floats*** (Figure 6-7) which should be implemented in the gate to create the double catamaran effect (transverse and longitudinal) when floating which leads to very general stability during the operational phases of the gate. (Not treated in this report.)

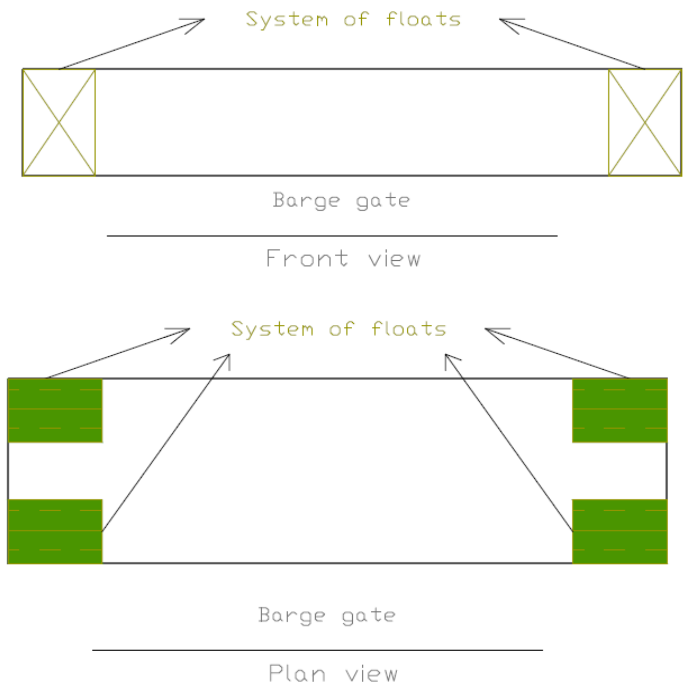


Figure 6-7: System of floats in the barge gate

- ***A group of side wall openings*** (optional) which are controlled by regulating valves and located in the walls of the gate (not shown here). When the gate is floating they reduce the impact of exceptional waves and when the gate is closed they allow a control of the discharge and of the upstream and downstream level. Because of the high costs the openings are optional and the applicability of them should be investigated more in the future research. (Not treated in this report.)
- ***A group of propellers*** to initiate and control the rotating movement of the floating gate which are supported by a group of cables in case of the emergency and failure (not shown here). (Not treated in this report.)
- ***A group of tyres and Teflon skids*** (shown with pink in Figure 6-5) which act like fenders and are fixed at the ends of the gate. They provide berthing support during the closing phase of the gate. These tyres act like the shock absorbers. (See section 0 for more information about the design of this structure.)
- ***A ballasting system*** including the valves and pumps to control the ballasting of the gate and facilitate the sinking of the gate during the closure phase (not shown here). (Not treated in this report.)
- ***A service chamber*** for the controlling devices and ballasting facilities (not shown here). (Not treated in this report.)
- ***Resting chamber*** (shown with orange wall in Figure 6-2) in which the gate will be stored when not in used and in open situation. (Not treated in this report.)

- ***A hydraulic braking system*** which is located in the resting chamber. This system will make the safe berthing of the gate during the after hurricane situation possible (not shown here). (Not treated in this report.)
- ***Damping fabrics*** (shown with light green layers in Figure 6-6) located on the foundations and would be for example from the rubber materials. The main function of them is to absorb the energy of the gate when it contacts the foundations and make the sinking process safe. (Not treated in this report.)
- ***A mechanical lock or a civil structure*** (not shown here) to provide stability against the negative head. (See section 10.5 for more information.)

It is worthwhile to mention that this conceptual design of the barge gate is tested previously in Belgium [67] in 1988. Maneuverability and feasibility tests were performed including gate rotation, tanks ballasting, wave and current actions and control of natural heave and roll motions. The experimental tests confirmed the reliability of the gate rotation and floating systems which are supported by guiding columns and spring cables. The gate remains in close contact with at least one guide-column without including any limitations on the gate motions by using the uniformly extended spring cable system. After the rotation of the gate, the cable extension is released.

6.3 Construction Material (DS2.2.)

6.3.1 Material Options

Steel is the most common material which is used for the gates of storm surge barriers and locks. The main reason is that steel has the high strength to weight ratio. The gates are affected by weather and water conditions and steel is highly sensible to corrosion. That is why coating is used for the steel gates. However, mostly the steel gates should be recoated during the life of the barrier and in the maintenance period which has the extra costs.

The other material is the ***rubber fabrics*** which is used in the inflatable rubber dams or can be used in the parachute barriers. ***Concrete*** is the other used material in storm surge barriers. This is the main material of the barriers body while the moving parts of the barriers are mostly from steel.

Fiber Reinforced Polymer (FRP) material with a high strength to weight ratio and low maintenance costs is the other material which is recently proposed to be used in the gates of the storm surge barriers [45]. The study by Kok [45] reveals that when the FRP is used for the barrier gates also larger spans become possible. The limitations in span are only dependent on the practical possibilities of the manufacturer and not on the material properties. Also, according to the comparison of the costs between the steel and FRP gate, the FRP gate is more cost-efficient. In his work, application of the FRP is especially in the lifting gates has been examined and it has been found out that usage of FRP in large hydraulic structures, specified to lifting gates is technically and economically feasible.

High Performance Lightweight Concrete (HPLC) is the other possible material. HPLC generally refers to concrete that can achieve high performance such as high-strength, high workability, low permeability and high durability while having low density. Berner et al. has studied the application of HPLC in a floating barge gate [46]. The barge gate with the design life of 100 years is designed for the Inner Harbor Navigation Canal (IHNC), Lake Borgne Hurricane Protection Barrier Project, in New Orleans, LA. The gate has the dimension of 57.9 m length, 19.1 m width, 13.3 height. The main reasons of the usage of the HPLC in the barge gate are permeability, durability, strength and the light weight of this material which makes it favorable for operability and movability of the gate (compared to normal concrete). In fact, HPLC can substantially reduce the dead weight while maintaining other high performance requirements. However, the formulation of the mix design requires special attention.

6.3.2 HPLC Advantages and disadvantages

The main reasons to use the HPLC for the gates are as follow:

- The adverse environment factors such as chloride corrosions, cyclic freezing and thawing, wave erosion are present for the barge gate. The function of the barge gate and also its service life is affected adversely because of these aspects. The concrete can be repaired when the deterioration is started but it is of course costly. Indeed, there is a need to think about the durability aspects in advance. So, to achieve the long service life for a gate, HPLC would be a material to be used.
- The floating barge gate should be a light structure to meet operational requirements (lighter structure would be moveable easier and faster). The conventional concrete types would significantly increase the weight and draught of the gate. Indeed, HPLC with low density can be used for reducing the weight of the gate.
- Permeability, durability and high strength are the other advantages of HPLC.

The high strength of the concrete type will also help in the design phase to meet the requirements of high loading conditions. The strength of the concrete can be changed on the basis of the mix design which is used. The material properties of the concrete class C55/67 with compressive strength of 67 MPa has been shown to be achievable through the laboratory tests by using HPLC [66].

The main disadvantaged of the HPLC material can be summarized as below:

- Higher costs in comparison with the normal concrete (the additional costs can usually be recovered by the reduced construction costs because of reduced dead weight).

- Complicated mixed design procedure and requirements compared to normal concrete.

6.3.3 Material selection

For the navigational barrier of the Bolivar Roads Pass, ***HPLC*** is chosen to be more appropriate for the design mainly because of the advantages of the durability and environmentally friendly aspects. Usage of the HPLC will satisfy the need for a durable lightweight structure compared to traditional concrete types. Also, the need for coating the steel structure due to corrosion during the service life of the project is prevented in this way.

6.4 Retaining Height (DS2.3.)

The main purpose of the barrier is to reduce the surge height in the Galveston Bay in the hurricane conditions. The rise of the water in the Bay is relatively small due to inflow of water from the inlet because the Galveston Bay is a large basin comparing to its inlets. Some overflow from the barrier is then be possible which makes the construction costs cheaper.

The maximum surge level at the governing Northern side of the Bay is equal to 3.4 m [11]. The calculations by de Vries (2014) shows that a safe surge of 3.4 m occurs when the water depth in the Bay has increased to the level of 1.4 m. Indeed, the overflow from the barrier may make the water level rise in the Bay to the maximum of 1.4 m.

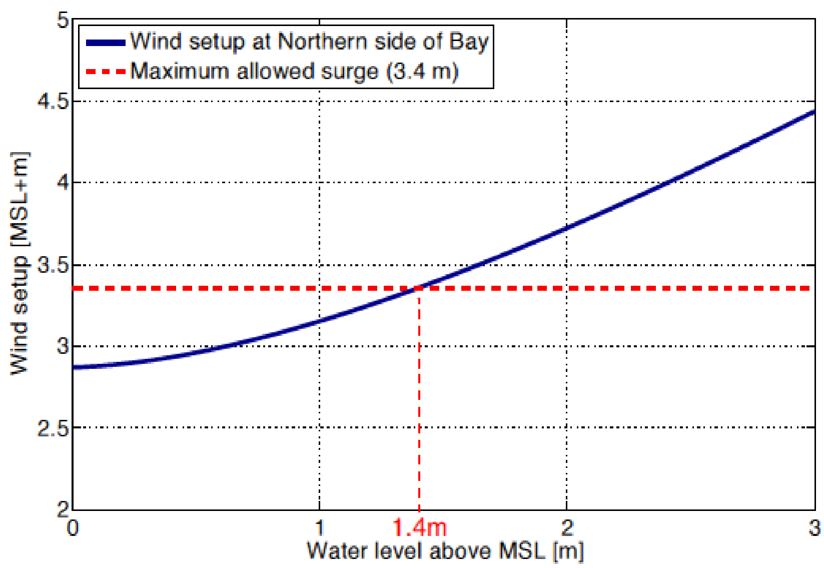


Figure 6-8: Surge due to wind setup related to increase of water level in the Galveston Bay [11]

Different options are possible for overflow of the water from the barrier for different retaining heights of the environmental section and the navigational section. Considering the costs estimations and surge levels, the optimizations have been done in the study of de Vries (2014) which shows that an equal retaining height over the full length of the storm surge barrier is the most cost effective option [11]. The retaining height of MSL+0.1 m has been found as sufficient in this study.

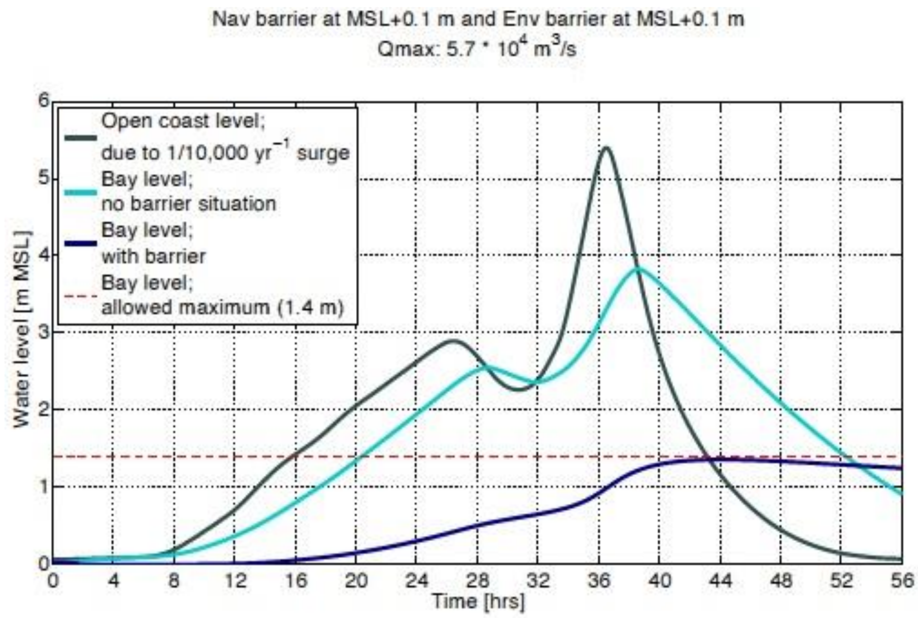


Figure 6-9: Retaining heights of the Environmental section and navigational section [11]

In this report, in the preliminary design step, the gate is going to be designed to retain all the occurring surge (no overtopping), which means the full protection and which is the critical condition of the gate design (because of the highest positive head). The design can be adjusted for less retaining height later if it is demanded.

6.5 Operational Phases (DS2.4.)

The floating barge gate is operated in different phases. In this section different situations of the gate are going to be described and the important loading cases are clarified. The information is then can be used in the design of the gate. It is important to consider the structural design and stability checks of the gate in each of these situations in the later steps. The dimensions of the gate are considered as follow:

- L_c = Length of the gate
- W_c = Width of the gate
- H_c = Height of the gate

6.5.1 Construction Phase

The first phase of the life cycle of the gate is the construction phase. It is assumed that the gate is going to be constructed in the building dock (See Section 12.1 for more information). In this phase there are no external loads on the barge gate (caisson structure). Indeed, this phase cannot govern the structural design of the gate. Special attention is needed for the removal of the formwork. It should be considered that the formwork should not be removed before the time that the concrete has gained required strength.

6.5.2 Transportation Phase

When the construction of the gate is finished, the gate should be floated and transported to the location of the project. Special attention is needed for the draught of the caisson in this phase. For the normal caissons, the floating phase is mostly the governing situation for the structural design of the caisson. During the floating the equilibrium should be achieved between the buoyant force and the weight of the caisson to satisfy the stability of the structure. Caisson in the transporting situation can be seen in Figure 6-10. The design of the caisson considering this operational phase is performed in section 7.4.1.

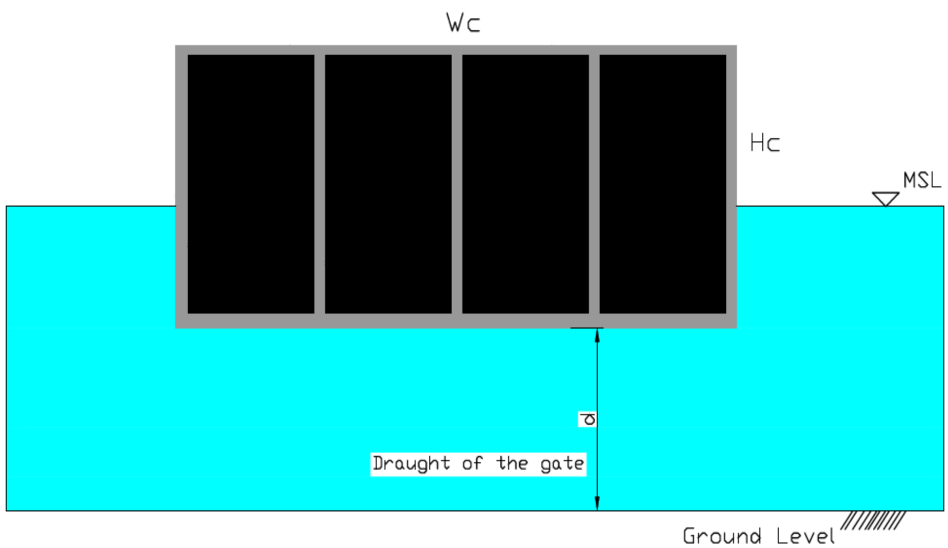


Figure 6-10: Transportation phase, floating caisson, cross section, dimensions not to scale

6.5.3 Gate in Open Position

In the open position the gate is stored parallel to the bank and perpendicular to the flow direction (Figure 6-3). This allows the passage of the ships in the normal condition without any hindrance. In this situation the gate is in a floating situation before resting on the foundations. The gate is connected to the guiding columns and the cables which help the gate to stay in the open position. The load cases in this situation should be considered as the floating conditions. The critical situation is when the gate is not ballasted and it is empty. Then the maximum loads act on the structure because of the lack of counter ballasting water weights. In the open situation the gate can be ballasted up to certain desired level to sink to the foundations. The design of the gate for the critical condition for this situation is same as the previous floating situation in the critical condition (Section 7.4.1). Also, when resting in the foundations, the floor slab should provide enough strength for bearing the ballasted water.

6.5.4 Gate During the Closure Process

The closure starts by setting the barge gate up to float (Figure 6-4). The floated barge gate is rotated around the articulation system (swing point) by means of the propellers and by help of the guiding cables. The gate can be ballasted to have more draught or floated while has the initial draught without any ballasting. This procedure is estimated to take around 20 to 30 minutes.

In this phase of the operation, the gate is in floating condition and the loads should be considered in the critical situation when the gate is empty of ballasting. The design of the gate in this situation is as same as the previous floating situation considering the hydrostatic loads (Section 7.4.1).

The floated gate should berth on the abutments horizontally. The gate then has the berthing energy which acts against the abutments. The fender system should be located on the gate in to absorb the berthing energy. The design of the berthing system is going to be considered in section 0. The berthing loads also should be considered in the design of the abutments.

6.5.5 Gate During the Immersion at Final Location

When the gate is arrived at its final location and it rests on the abutments, then the gate should be immersed by using the ballasting system with the water (Figure 6-11). This phase may take around 40 to 60 minutes. The gate is finally rests vertically on the two foundations which are located at two ends of the span near the abutments.

The immersion should be controlled by using the compressed air in the ballasts. During the immersion the openings on the walls (if applicable) can remain open which reduce the forces on the structure. At the beginning of the immersion the gate rolls on the tyres (fenders) located at the ends of the gate. The fenders provide the movement of the gate vertically possible. However, when the water pressure acting on the gate is too high, the tyres can be compressed and the gate should rest then on the Teflon skids. The gate is also supported horizontally on the abutments facing to the port side at the final location.

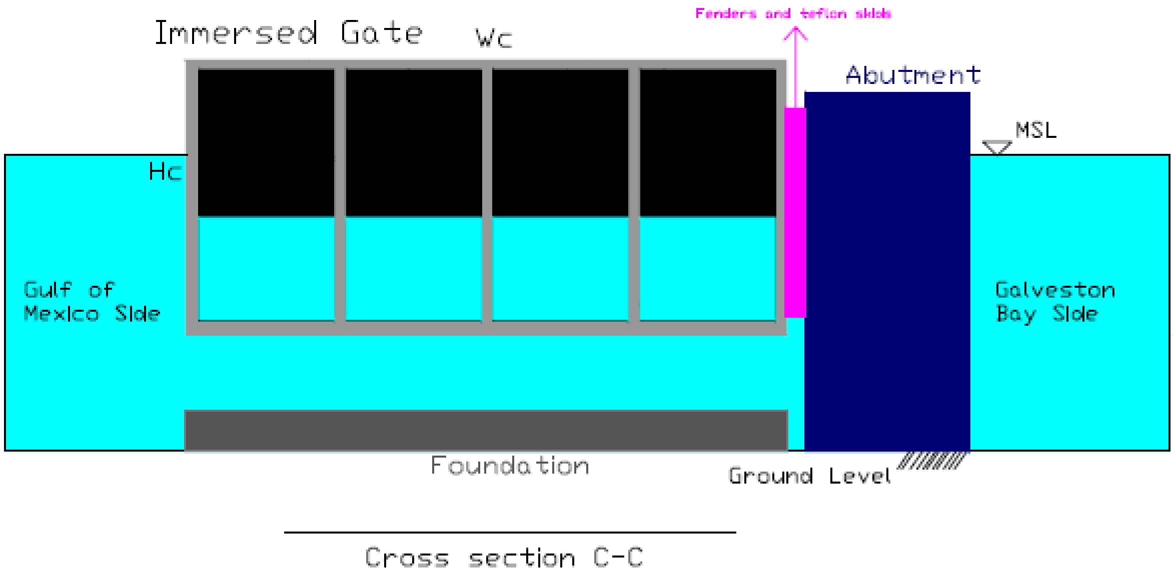


Figure 6-11: Ballasted gate during immersion, cross section C-C, dimensions not to scale

In this situation the design should be done for the caisson when it is ballasted with water (considering different ballasting heights inside the caisson). The ballasting takes time and the caisson should be stable during the immersion. The design should also consider the structural aspects. However, because of the presence of the ballasting water inside the gate and its counter effect reaction, the loads acting on the caisson might not be as critical as an empty caisson. The design of the gate for this situation is considered in section 7.4.2.

6.5.6 Gate Immersed at Final Location During Normal Conditions

When the gate is totally immersed at final location in the normal condition, then the sea levels in the Gulf side and Bay side are almost equal (Figure 6-12). The critical situation here is the design of the ballasted gate which rests on the foundations vertically. The gate should be able to resist the vertical forces here which are mainly the weight of the gate and the ballasting water. However, the buoyancy force is favorable. To be sure that the gate will rest on the foundations during the hurricane, the ballasted water can be implemented more than required for the desired level of draught. Then the vertical forces on the downward side is more critical. The local stresses and the global stresses should be checked. The design of the gate in this condition is treated in section 7.4.3. The cross section D-D of the gate can be seen in Figure 6-6.

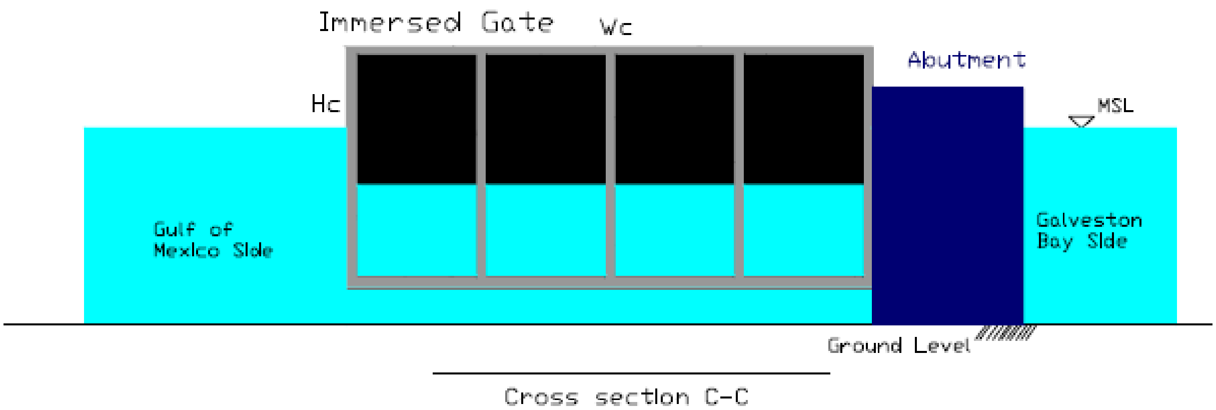


Figure 6-12: Immersed gate resting on foundation during the normal conditions, cross section C-C, dimensions not to scale

6.5.7 Gate Immersed at Final Location During Hurricane Conditions

The last step when the gate is immersed is closing of the valves and openings (if applicable) of the side walls under the controlled conditions. Then the closure of the flow through the gate is achieved during the hurricane conditions. If there is no valves or openings the loads are more critical and in this report this situation is considered for the design.

After ballasting, the gate will rest vertically on the foundations at its ends (Figure 6-6). Then actually the gate is supported horizontally with abutments and vertically with foundations.

The design of the structure should be done for different load cases in this situation. When there is a hurricane, the water level in the Gulf side is higher than in the Bay side. The loads should be considered for the global and overall structural design. In the overall structural design, the gate is going to be modeled as a beam resting on two supports, individually for horizontal and vertical cases. The design of the gate considering this end situation is considered in section 7.4.4.

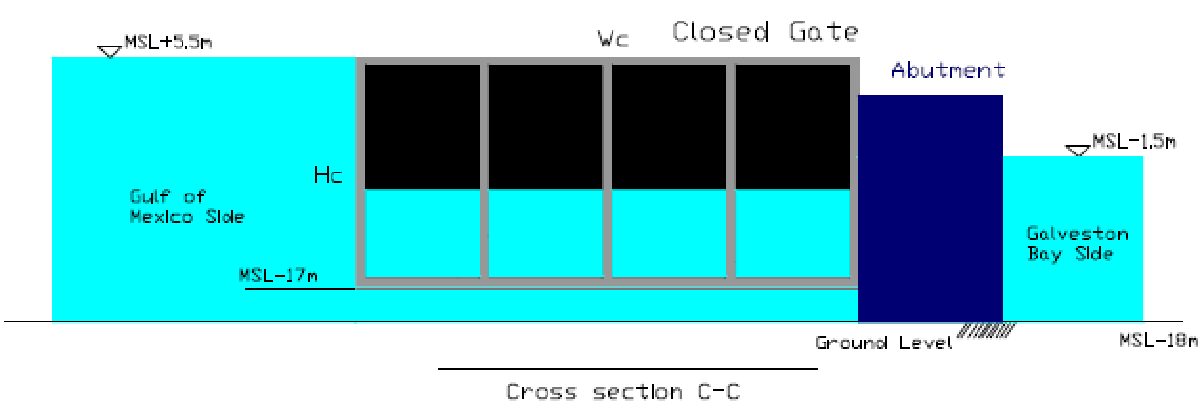


Figure 6-13: Closed gate during the hurricane conditions, cross section C-C, dimensions not to scale

At the final closing situation there is a 1 meter gap under the gate. The bed protection should be designed for this area to prevent the ground from scour during the hurricane condition on the depth of MSL-18 m. Also, this gap and the current flow from under the gate will cause the suction force to the gate which should be considered in the design of the gate itself. The design of the bed protection under the gate is treated in section 8.2.

6.5.8 After Hurricane Situation

After the hurricane, the gate is afloat again to its initial position. The ballasting system by using the compressed air makes the ballasting tanks empty and the gate is back to the open position by guiding through propellers and the cables. This procedure may take around 30 to 40 minutes.

It would be a situation when the sea level in Bay side is higher than the sea level in the Gulf side (Figure 6-14). Considering the static loading situations, the hydrostatic loads on the structure in this case are less critical than the during the hurricane condition (the loads on the structure because of the negative head are smaller compared to the positive head). Indeed, the structural design for the previous situation will fulfill the structural design requirements of this situation if the walls of the caisson would be identically designed. This is the case because it is preferable to have a symmetric caisson structure.

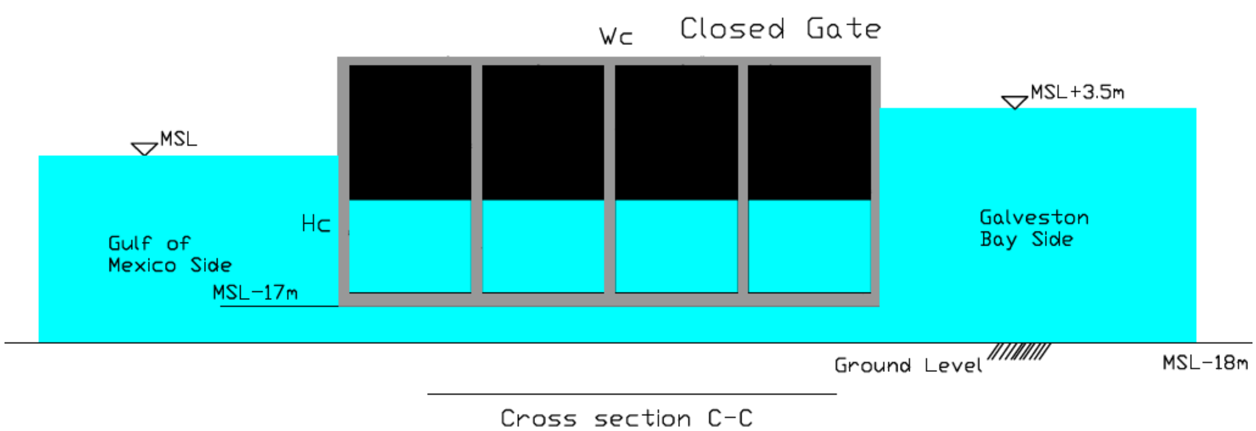


Figure 6-14: Closed gate after the hurricane conditions, cross section C-C, dimensions not to scale

In this situation, the gate will open under the current forces because it is not supported horizontally against the negative head (Figure 6-15). If the gate tends to open by itself under the current forces, possible damages can be expected to the gate and other structures.

Some solutions here are for example opening the environmental barrier to prevent the negative head on the navigational barrier, opening the valves and openings (if applicable) in the barge gate to reduce the negative head forces or implement a mechanical lock or a civil structure to resist the negative head forces and prevent the uncontrolled opening of the gate. For more information in this regard see section 10.5.

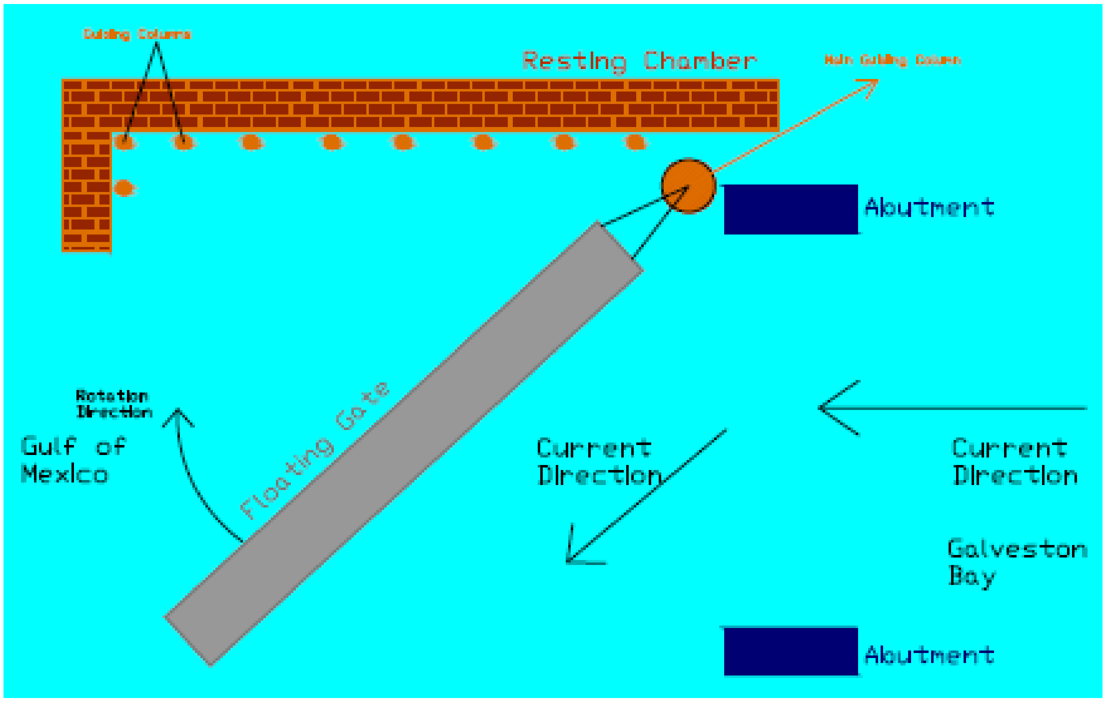


Figure 6-15: Opening of the gate after the hurricane under current, dimensions not to scale

6.6 Conclusions

In this chapter the conceptual design of the navigational storm surge barrier system is described. The system of the barrier is depicted considering different structural and functional elements. The gate is considered as the floating barge gate which is going to be designed as a concrete caisson structure. HPLC is chosen as the suitable material for the design of the barge gate. The retaining height of the barrier is assumed to be firstly fully water retention.

Different operational situations are described in this chapter. In each situation, the critical loads are going to be calculated and the gate is going to be designed to meet the structural design and stability requirements (See Section 7).

After DS2. the requirements of the barrier can be checked. Table 6-1 shows the status of the requirements after this design step. For the requirements with unknown status, the design should be continued for finding out about their status. For sure all of the requirements need more checks and design works in the future which is outside of the scope of the current report.

Program of Requirements for the Navigational Barrier (After DS2)		
Type	**Criteria**	**Status after DS2.**
General	Open in normal conditions	Expected to be ok
	Closed during the hurricane conditions	Expected to be ok
	Realizeable solution	Unknown
	Operable solution	Expected to be ok
	Reliable solution	Unknown
	Economical solution	Unknown
Navigational	Enough width	Expected to be ok
	Enough depth	Expected to be ok
	Enough air draft	Expected to be ok
Current velocity	Acceptable current velocity in navigational channel	Expected to be ok
Safety level	Enough retaining height	Expected to be ok
Lifetime	Enough lifetime	Expected to be ok
	Durable solution	Expected to be ok
Environmental	Environmentaly friendly solution	Expected to be ok

Table 6-1: Status of requirements after DS2.

7 Geometry Design of the Gate (DS3.)

7.1 Introduction

In this chapter the floating barge gate is going to be structurally designed. Firstly the design inputs and assumptions are going to be depicted. Then the gate is going to be designed for different load cases and situations on the basis of the operational phases described in section 6.5. In this design procedure initially the hydrostatic behavior of the gate is considered. This chapter treated the design step 3 and its sub-steps (Figure 7-1). It should be mentioned that the focus of the current chapter is mainly on the conceptual design of the barge gate.

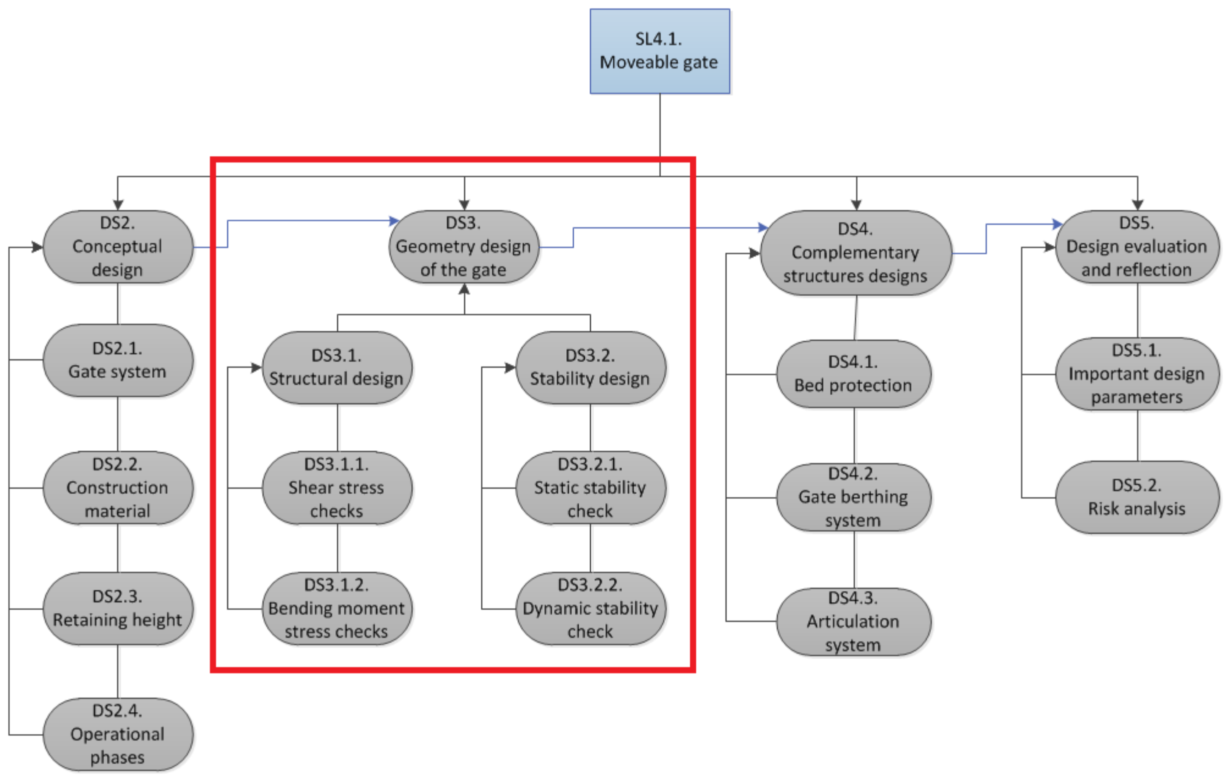


Figure 7-1: Design step 3; Geometry design of the gate

7.2 Gate Design Parameters

In this section the main parameters of the design are described.

7.2.1 Water and Wave Data

The inputs of the wave and water parameters are as follow:

Parameter		Value	Unit
ρ_w	Mass density of salt water	10,25	kN/m^3
h_{surge}	Maximum surge level	5,4	m
H_{max}	Maximum wave height	5,9	m
H_s	Significant wave height	3,3	m
$T_{p,\ hur}$	Peak wave period hurricane condition	7,9	s
$T_{p,\ reg}$	Peak wave period regular condition	4	s
SLR	Sea level rise in 100 years	1	m

Table 7-1: wave and water parameters in the Bolivar Roads Pass

7.2.2 Material Properties

The material is selected as HPLC. The concrete class is assumed as high strength lightweight concrete. The properties is adopted from the concrete class B65, C55/67 (Table 7-2). The mass density of concrete is assumed to be 17 kN/m^3.

It should be mentioned that firstly the concrete with the density of 24 kN/m^3 was chosen for the design. After calculations because of the heavy weight of the barge gate with this concrete type the lightweight concrete is selected as the best option for the gate.

Concrete class (old)	Concrete class	$f_{ck,cil}$ (MPa)	f_{ck} (MPa)	f_{cm} (MPa)	f_{ctm} (MPa)	$f_{ctk,\ 0.05}$ (MPa)	$f_{ctk,\ 0.95}$ (MPa)	E_{cm} (GPa)
B15	C12/15	12	15	20	1.6	1.1	2.0	27
B25	C20/25	20	25	28	2.2	1.5	2.9	30
B35	C30/37	30	35	38	2.9	2.0	3.8	33
B45	C35/45	35	45	43	3.2	2.2	4.2	34
B55	C45/55	45	55	53	3.8	2.7	4.9	36
B65	C55/67	55	67	63	4.2	3.0	5.5	38

Table 7-2: Characteristics of concrete class [58]

In the figure above the parameters are as follow:

- $f_{ck,cill}$ = characteristics compressive cylinder strength of concrete at 28 days
- f_{ck} = characteristics compressive cube strength
- f_{cm} = mean value of concrete cylinder compressive strength after 28 days
- f_{ctm} = mean value of axial tensile strength of concrete
- $f_{ctk,\ 0.05}$ = characteristics axial tensile strength of concrete, 5% fractile
- $f_{ctk,\ 0.95}$ = characteristics axial tensile strength of concrete, 95% fractile
- E_{cm} = secant modulus of elasticity of concrete

Material factors are shown in Table 7-3.

Material (property)	γ_m
Concrete (compression stress)	1.2
Concrete (tensile stress)	1.4
Reinforcement steel	1.15
Pre-stressed steel	1.1
Construction steel	1

Table 7-3: Material factors concrete and steel

The design value of compressive strength f_{cd} and tensile strength f_{ctd} of concrete can be derived from the following formulas:

$$f_{cd} = \frac{f_{ck}}{\gamma_{c,m}} \qquad \text{Eq. 7-1}$$

$$f_{ctd} = \frac{f_{ctk,0.05}}{\gamma_{c,m}} \qquad \text{Eq. 7-2}$$

The values are in N/mm^2.

The reinforcement steel is selected as B500B type with the R_e, characteristics yield strength of reinforcement (f_{yk}), equal to 500 MPa and the Young's modulus (E_s) of $2*10^5$ N/mm^2. The commonly used reinforcement bar diameters in hydraulic engineering structure are ϕ 12,16,20,25 and 32 [58].

7.2.3 Caisson Geometry

The initial dimensions of the caisson is considered as below:

- L_c = 230 m, Length of the caisson
- W_c = 36 m, Width of the caisson
- H_c = 22.5 m, Height of the caisson

The initial length of the caisson is considered as the required opening (220 m) plus 5 meter from each end of the gate reserved for resting on the supports (abutments and foundations). These dimensions might be adjusted in the later steps due to the design requirements.

The thickness of the walls and compartments are considered initially as follow:

- t_w = 1 m, Thickness of the external walls
- t_f = 1 m, Thickness of the floor slab
- t_r = 0.5 m, Thickness of the roof slab
- t_{iw} = 0.3 m, Thickness of the internal walls (compartments)

These values would be adjusted during the design procedure on the basis of the strength needs.

It has been assumed that the compartments are in every 6 meters in the width direction and every 10 meters in the length direction of the caisson. However, these spaces can be modified later on the basis of the design needs. Cross section of the caisson has been shown in Figure 7-2.

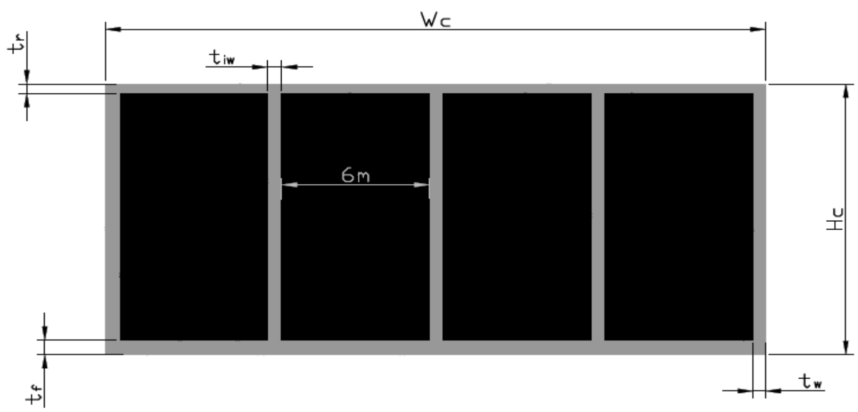


Figure 7-2: Cross section of the caisson in width direction for initial calculations (number of compartments and the space between them are not determined yet)

7.2.4 Loads

The main loads on the structure are as follow (initially only hydrostatic loads are considered):

Hydrostatic pressure:

The hydrostatic water pressure is as follow [58]:

$$P_{hyd} = \rho_w \,.g\,.h \qquad \text{Eq. 7-3}$$

In which:

- P_{hyd} = hydrostatic water pressure [Pa]
- ρ_w = density of water [kg/m^3]
- h = pressure head [m]
- g = acceleration due to gravity (≈10) [m/s^2]

The resulted force can be calculated as below:

$$F_{hyd} = \int P_w \, dA \qquad \text{Eq. 7-4}$$

In which:

- F_{hyd} = the hydrostatic force perpendicular to the plane [N]
- dA = a small part of the area [m^2]
- A = the total surface area [m^2]

Wave load:

According to the linear wave theory for non-breaking waves against a vertical wall, the maximum pressure against a wall in case of reflection is [58]:

$$P_{wave} = \rho_w g H_i \frac{\cosh(k(d+z))}{\cosh(kd)} \quad \text{for} \quad -d < z < 0 \qquad \text{Eq. 7-5}$$

$$P_{wave} = \left(1 - \frac{z}{H_i}\right) \rho_w g H_i \quad \text{for} \quad 0 < z < H_i \qquad \text{Eq. 7-6}$$

In which:

- P_{wave} = wave pressure [Pa]
- ρ_w = density of water [kg/m^3]
- g = acceleration due to gravity (≈10) [m/s^2]
- H_i = wave height of an incoming wave [m]
- K = wave number of the incoming wave = 2π/L [m^{-1}]
- L = wave length [m]
- d = depth of the water [m]
- z = desired depth [m]

The wave force per linear meter can be derived from integration over the water depth:

$$F_{wave} = \int P_{wave}\, dz \qquad \text{Eq. 7-7}$$

In which:

- F_{wave} = the wave force perpendicular to the plane [N]

Dead weight:

The dead weight of the elements can simply be calculated from the following formula:

$$F_{dw} = \rho \,.\, g \,.\, V \qquad \text{Eq. 7-8}$$

In which:

- F_{dw} = the dead weight vertical force of the element [N]
- ρ = density of the element [kg/m^3]
- V = volume of the element [m^3]

- g = acceleration due to gravity (≈10) [m/s^2]

This can be the vertical dead weight force of the concrete or the ballasted water.

7.3 Gate Design Requirements

The following design checks should be done for the structure in different situations and positions.

7.3.1 Shear Stress Checks (DS3.1.1.)

The shear stress in the structure components should be checked including the walls, floor slab and roof slab.

As the initial check, the shear stress (τ) can be checked by using the maximum shear force in the element compared to the maximum allowable shear stress [59]:

$$\tau_d = \frac{3\,V_d}{2\,b.t} \qquad \text{Eq. 7-9}$$

In which:

- τ_d = the design value of the shear stress [N/mm^2]
- V_d = the design value of the shear force in the considered cross section [N]
- t = thickness of the concrete part that should bear V_d [mm]
- b = width of the concrete part that should bear V_d [mm]

The maximum allowable shear stress in the element is as follow:

$$\tau_{max} = 0.4 f_b + 0.15\,\sigma'_{bmd} \qquad \text{Eq. 7-10}$$

In which:

- τ_{max} = Maximum allowable shear stress if no shear reinforcement is applied [N/mm^2]
- f_b = design value of concrete tensile strength [N/mm^2]
- σ'_{bmd} = average design value of concrete compressive strength [N/mm^2]

The following requirement should be checked:

$$\frac{\tau_{max}}{\tau_d} \leqslant 1 \qquad \text{Eq. 7-11}$$

To be more precise, the shear resistance of the element without shear reinforcement is as follow:

$$V_{Rd,c} = [C_{Rd,c}\,.\,k.\,(100.\,\rho_1\,.\,f_{ck})^{1/3} + k_1\,.\,\sigma_{cp}\,]\,.\,b_w\,.\,d \qquad \text{Eq. 7-12}$$

In which:

- f_{ck} = characteristics compressive cylinder strength of concrete at 28 days in MPa
- $k = 1 + (200/d)^{1/2}$ with d in mm
- ρ_1 = reinforcement ratio for longitudinal reinforcement = $\frac{A_{sl}}{b_w d} < 0.02$
- A_{sl} = the area of the tensile reinforcement which extends > (l_{bd} + d) beyond the section considered
- b_w = the smallest width of the cross section in the tensile area in mm
- σ_{cp} = compressiove stress in the concrete from axial load or prestressing
- k_1 = a coefficient, in the Netherlands 0.15
- $C_{Rd,c}$ = a coefficient, in the Netherlands, $0.18/\gamma_c$

The two above mentioned maximum shear stress are going to be checked for each element. If the above requirements are not met, the (vertical) shear reinforcement should be considered. The amount of the shear reinforcement can be derived from following formula [58]:

$$A_{sw} = \frac{s.V_{Rd,s}}{z.cot\theta.f_{ywd}} \qquad \text{Eq. 7-13}$$

In which:

- A_{sw} = the cross sectional area of the shear reinforcement (two times because the reinforcement crossed two times the cross sectional area of the concrete)
- s = the spacing of the stirrups
- f_{ywd} = the design yield strength of the shear reinforcement
- θ = angle between the concrete compression strut and the beam axis perpendicular to the shear force ($21.8 \leq \theta \leq 45$ degrees)
- $V_{Rd,s}$ = occurring shear stress
- z = arm of internal leverage for a member with constant depth, corresponding to the bending moment in the element under consideration. (≈0.9 d)

7.3.2 Bending Moment Stress Checks (DS3.1.2.)

The required reinforcement should be calculated for the bending moment occurring in the element. The requirement should be satisfied that the reinforcement steel must yield before the concrete will fail and the minimum reinforcement percentage must be large enough to be sure there will be no brittle failure when cracking of the concrete occurs [58].

The maximum allowable bending moment for an element can be computed using the following formula [58]:

$$M_u = A_s \,.\, f_{yd} \,.\, d \,.\, (1 - 0.52 \,.\, \rho \,.\, k) \qquad \text{Eq. 7-14}$$

In which:

- M_u = ultimate absorbable bending moment
- A_s = total cross sectional area of reinforcement
- k = ratio between the strength of concrete and steel = f_{yd}/f_{cd}
- f_{yd} = design yield strength of reinforcement
- f_{cd} = design value of concrete compressive strength
- ρ = reinforcement percentage = $\frac{A_S}{b\ .\ d}$
- b = width of the concrete structure
- d = effective height of the cross section = h – (c + ½ ϕ)
- c = concrete cover
- ϕ = bar diameter
- h = height of the cross section

The reinforcement should be enough for satisfying the following condition:

$$M_{ed} \leqslant M_u \qquad \text{Eq. 7-15}$$

In which:

- M_{ed} = design value for the bending moment

For the initial estimation of the ρ, by considering the thicknesses of the elements, the following flowchart can be used. Then the above mentioned criteria can be checked for this reinforcement percentage and possibly adjusted.

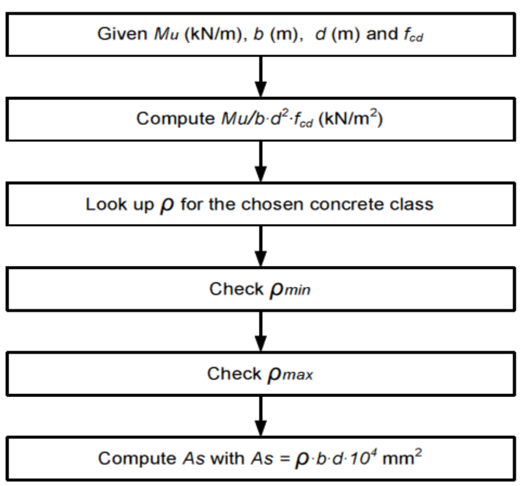


Figure 7-3: Flowchart for the preliminary design of reinforcement [58]

$\frac{M_d}{bd^2 f_{cd}}$	ψ	k_x	k_z	ρ [%]				
				C20/25	C28/35	C35/45	C45/55	C53/65
10	0,010	0,013	0,99	0,03	0,05	0,06	0,08	0,09
20	0,020	0,027	0,99	0,07	0,10	0,13	0,15	0,18
30	0,030	0,240	0,98	0,10	0,15	0,19	0,23	0,27
40	0,041	0,055	0,98	0,14	0.20	0.25	0.31	0.37
50	0,051	0,068	0,97	0,18	0,25	0,32	0,39	0,46
60	0,062	0,083	0,97	0,21	0,30	0,39	0,47	0,56
70	0,073	0,097	0,96	0,25	0,35	0,45	0,55	0,66
80	0,084	0,112	0,96	0,29	0,41	0,52	0,64	0,75
90	0,095	0,127	0,95	0,33	0,46	0,59	0,72	0,85
100	0,106	0,141	0,94	0,37	0,51	0,66	0,81	0,95
110	0,117	0,156	0,94	0,40	0,56	0,73	0,89	1,05
120	0,129	0,172	0,93	0,44	0,62	0,80	0,98	1,16
130	0,140	0,187	0,93	0,48	0,68	0,87	1,06	1,26
140	0,152	0,203	0,92	0,52	0,73	0,94	1,15	1,36
150	0,164	0,219	0,91	0,57	0,79	1,02	1,24	1,47
160	0,176	0,235	0,91	0,61	0,85	1,09	1,34	1,58
170	0,188	0,251	0,90	0,65	0,91	1,17	1,43	1,69
180	0,201	0,268	0,90	0,69	0,97	1,25	1,53	1,80
190	0,214	0,285	0,89	0,74	1,03	1,33	1,62	1,92
200	0,227	0,303	0,88	0,78	1,10	1,41	1,72	2,04
210	0,240	0,320	0,88	0,83	1,16	1,49	1,82	2,16
220	0,253	0,337	0,87	0,87	1,22	1,57	1,92	2,27
230	0,267	0,356	0,86	0,92	1,29	1,66	2,03	2,39
240	0,281	0,375	0,85	0,97	1,35	1,75	2,13	2,52
250	0,295	0,393	0,85	1,02	1,43	1,83	2,24	2,64
260	0,310	0,413	0,84	1,07	1,50	1,93	2,35	2,78
270	0,325	0,433	0,83	1,12	1,57	2,02	2,47	2,91
280	0,340	0,453	0,82	1,17	1,64	2,11	2,58	3,05
290	0,356	0,475	0,81	1,23	1,72	2,21	2,70	3,19
300	0,372	0,496	0,81	1,28	1,80	2,31	2,82	3,34
310	0,388	0,517	0,80	1,34	1,87	2,41	2,94	3,48
320	0,405	0,540	0,79	1,40	1,96	2,51	3,07	3,63

Table 7-4: Reinforcement percentage for rectangular cross sections, reinforced with B500B [58]

7.3.3 Static Stability Checks (DS3.2.1.)

By accounting the sensitivity to tilting, moments equilibrium about a floating element is not sufficient. Therefore metacentric height (GM) should be estimated, according to the Figure 7-4 [58].

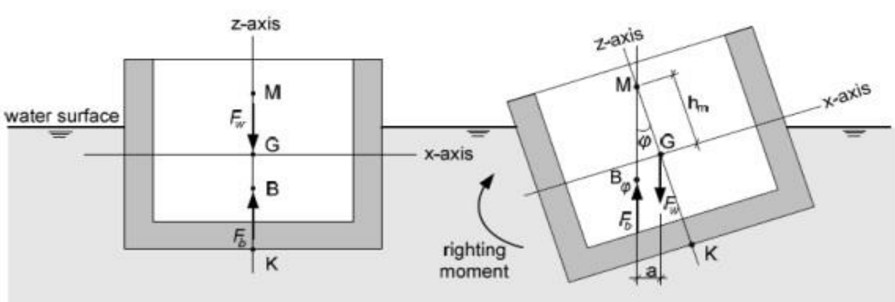


Figure 7-4: Floating caisson static stability check [58]

The Figure 7-4 indicates three vital points of importance for stability as well as ϕ (tilting angle):

- The centre of buoyancy (B), is the point of application of the buoyant force and thus the centre of gravity of the displaced water. In tilted position the centre of buoyancy shifts to a new position due to the changed geometry. The shifted centre of buoyancy is indicated with B_ϕ and the horizontal shift is a.
- The centre of gravity (G) of the element, including ballasting, where the centre of gravity generally remains fixed with respect to the element because it just depends upon the position of the element's weight and ballast. The centre of gravity at the same time is the rotation point.
- The metacentre, M, indicates the point of intersection of the axis of symmetry, the z-axis, and the action line of the buoyant force in tilted position.

For static stability, rotation of the element should be compensated by a righting moment caused by the buoyant force and the weight of the element. This is the case if M is located over G: the line segment GM, also known as the metacentric height (h_m), which must be positive [58].

Static stability (outset stability, due to small rotations on the investigated element) includes the following steps:

- Estimation of the weight F_w , G centre of gravity of the floating element with reference to K (KG), with K being the intersection of the z-axis with the bottom line of the element.
- As a next step the draught d of the element should be calculated
- The location of the centre of buoyancy B and its position above the bottom of the element should be calculated (KB). In case of rectangular elements, KB = d/2.
- Determine the shape and the smallest moment of area, as the most vulnerable one
- The volume of the displaced fluid V should be computed
- BM=I/V should be computed
- Calculate meta centric height as, h_m= GM = KB + BM - KG
- If $h_m > 0$, the body is stable. In practice, $h_m > 0.50$ m is recommended for the caissons [58, 59].

The dimensions of the caisson should be designed in a way that the requirement of the $h_m > 0.5$ meter is met.

7.3.4 Dynamic Stability Checks (DS3.2.2.)

The dynamic stability check of the caisson should be considered for two cases below:

Sway check:

If the dimensions, length or width, of the floating caisson are too small compared to the length of the waves or swell, the element will start swaying on the waves. The following rules of thumb should be checked:

$$L_w < 0.7\, L_c \;\&\; L_w < 0.7\, W_c \qquad \text{Eq. 7-16}$$

In which:

- L_w = wave length or swell length [m]
- L_c = length of the caisson [m]
- W_c = width of the caisson [m]

Natural oscillation:

The most severe situation however is the resonance that occurs when the waves and swell frequencies approach the natural frequency of the floating element [58]. For preventing this, the natural oscillation period of the element should be larger than that of the waves or swell.

If the natural oscillation period needs to be changed, but re-design do not offer a solution, or is too expensive, the transportation and positioning should occur in favourable conditions as far as waves and swell are concerned. Then the increased costs of the additional measures and/or design alterations should be counteracted by the costs of possible delays.

Ignoring the hydrodynamic mass and damping, the natural oscillation period can be estimated as [59]:

$$T_0 = \frac{2\pi j}{\sqrt{g h_m}} \qquad \text{Eq. 7-17}$$

In which:

- T_0 = natural oscillation period [s]
- h_m = metacentric height [m]
- g = gravitational constant [m/s^2]
- j = polar inertia radius of the element [m]

The polar inertia radius can be found according to:

$$j = \sqrt{\frac{I_{oplar}}{A}} \qquad \text{Eq.7-18}$$

In which:

- A = the area of concrete in a vertical cross-section [m^2]

- I_{polar} = polar moment of inertia [m^4] = $I_{xx} + I_{yy}$
- I_{xx} = polar moment of inertia around the z-axis [m^4]
- I_{zz} = polar moment of inertia around the x-axis [m^4]

The larger polar inertia radius results into larger natural oscillation period, while a favorable larger metacentric height, for the static stability, decreases the natural oscillation period [58].

If the natural oscillation frequency is much larger than the wave or swell frequency, the element is dynamically stable for oscillations. For the current design, the safety factor of at least 2 is considered. Then the following criteria should be checked:

$$T_0 > 2 \cdot T_{p,reg} \qquad \text{Eq. 7-19}$$

In which:

- $T_{p,reg}$ = the peak wave period in regular circumstances

7.4 Design Checks of the Gate

In this section different situations are going to be considered for the structural and stability design of the barge gate. In each situation the design checks mentioned in section 7.3 are implemented to adjust the dimensions of the caisson on the basis of the strength and stability needs. The calculations are done in the Serviceability Limit States (SLS) by considering the material factors. The final design will be the design in which all the design requirements and checks in different operational situations are fulfilled.

Flowchart of Figure 7-5 summarizes iterative process of all the design checks which have been considered. Also, the important load cases and information regarding the design in each phase are summarized in Figure 7-6.

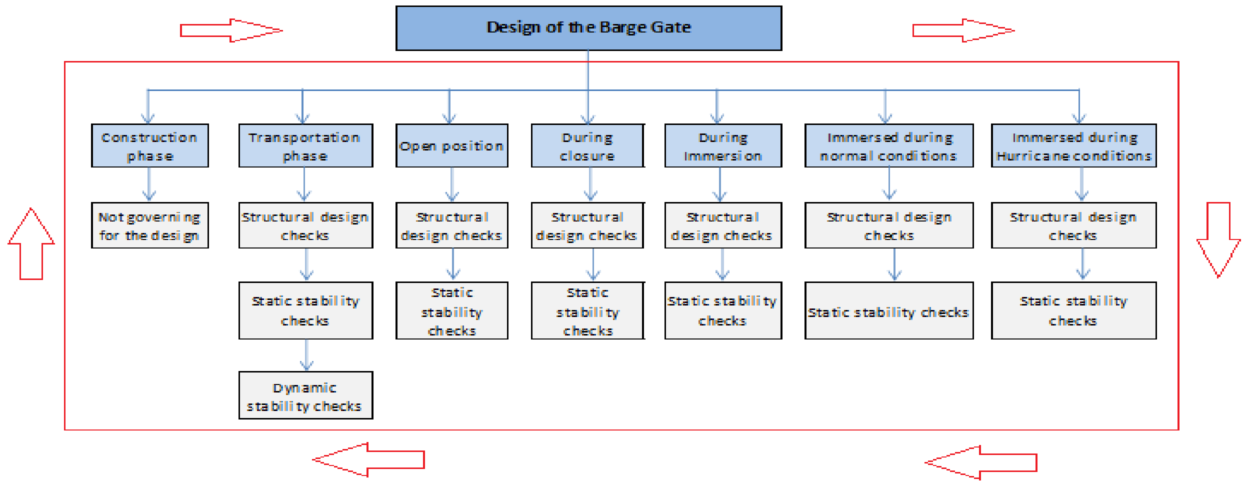


Figure 7-5: Different design situations of the gate

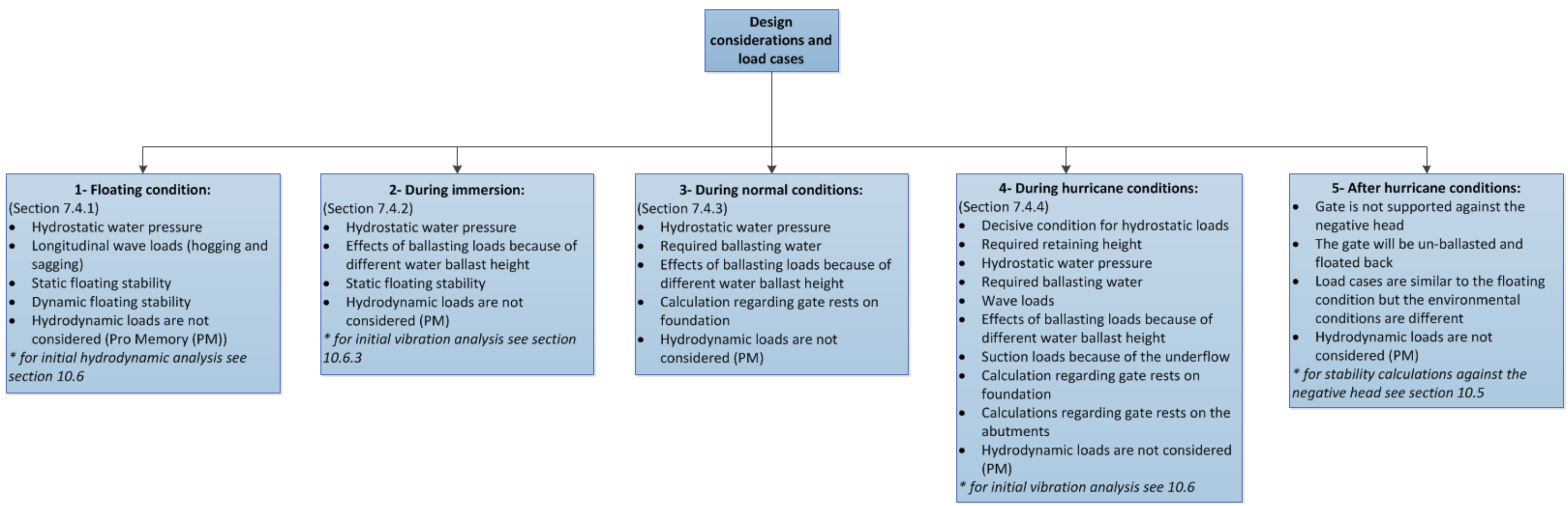


Figure 7-6: Design considerations in each operational phase and load cases

7.4.1 Design Check 1: Floating Caisson

The floating situation is the case during the transportation (Section 6.5.2), open position (Section 6.5.3) and during the closure process (Section 6.5.4). In this case, in the worst case scenario, the caisson is empty and the water pressure from the outside of the caisson to the walls and floor slab are critical. Attention should be made that in all of these situations the normal water and wave boundary conditions are applicable. (See section 14.1.1 for results of the calculations in detail for design check 1.)

The following design checks are used for this phase:

- Shear stress checks (according to section 7.3.1)
- Bending moment stress checks (according to section 7.3.2)
- Static stability checks (according to section 7.3.3)
- Dynamic stability checks (according to section 7.3.4)

The following load cases should apply in this situation:

Hydrostatic water pressure:
For design of the caisson in this situation, firstly the initial draught (d_i) of the caisson considering the initial dimensions is calculated. The hydrostatic water pressure on the caisson is shown in Figure 7-7.

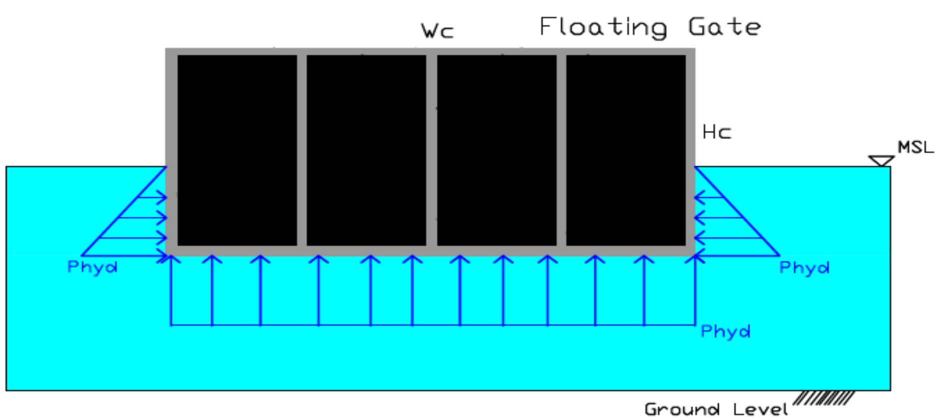


Figure 7-7: Floating caisson cross section, hydrostatic loads (dimensions not to scale)

A cross section in the middle (length direction) of the caisson should be considered. The influence of the headwalls is neglected which works out on the conservative side because in reality the head walls take over part of the horizontal forces.

Longitudinal wave loads:
When a wave passes by the floating structure, hogging and sagging moments will occur. The floating structure behaves in this situation like a ship which is shown in Figure 7-8.

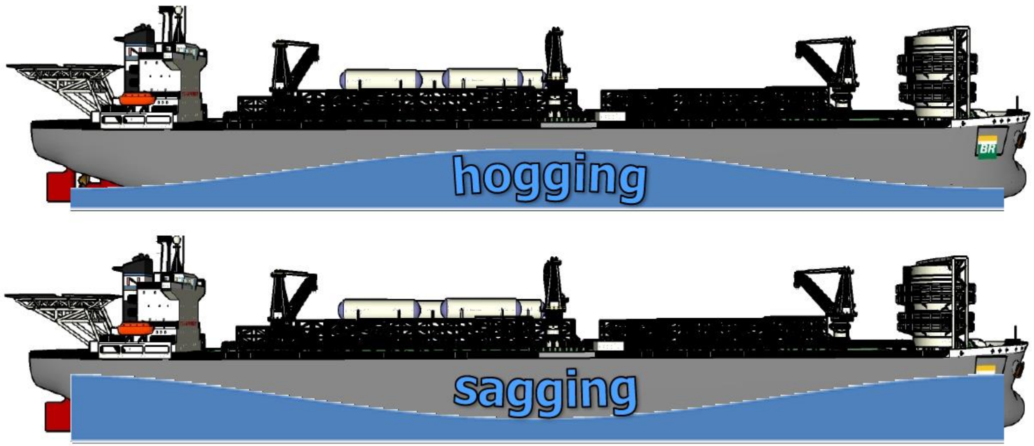


Figure 7-8: Hogging and sagging for a floating structure [69]

The design wave can be considered with the trochoidal shape with the wave length and wave height during the floating situation (which is the normal condition). A trochoidal wave can be constructed as a rolling wheel (Figure 7-9).

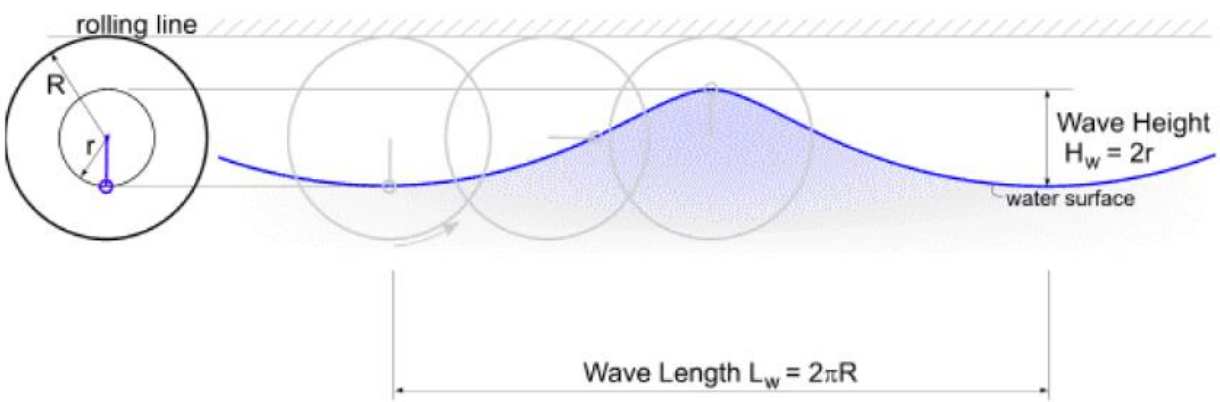


Figure 7-9: Trochoidal wave [70]

The gate can be modelled as a beam in this situation (Figure 7-10).

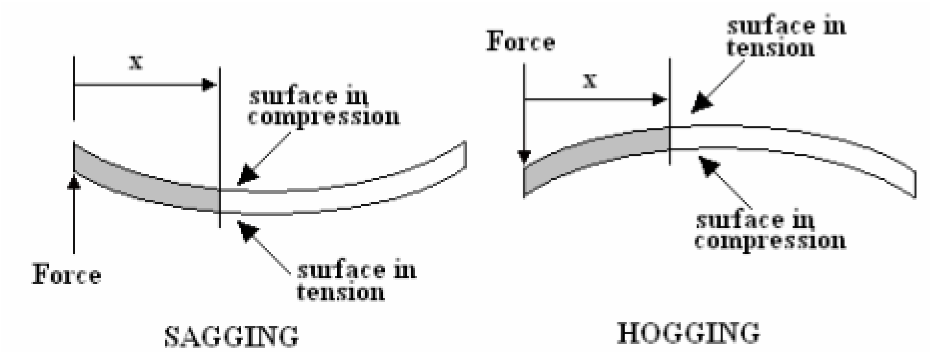


Figure 7-10: Hogging and sagging bending moments

7.4.2 Design Check 2: Gate During the Immersion at Final Location

The design checks should be done for the gate when it is ballasted to get immersed up to the desired location. This situation is for the operational case of section 6.5.5. The required amount of water for ballasting to immerse the gate up to the MSL-17 m can be calculated in this step. (See section 14.1.2 for results of the calculations in detail for design check 2.)

In this design check only the static stability of the gate during immersion is considered (according to section 7.3.3). The structural design checks for shear stress and bending moment stress is not considered because obviously the empty caisson has the more critical situation. In the current situation the ballasting water inside the gate has the counter effect for the hydrostatic loads which is favorable in terms of the structural design. The loads cases in this situation are shown in Figure 7-11.

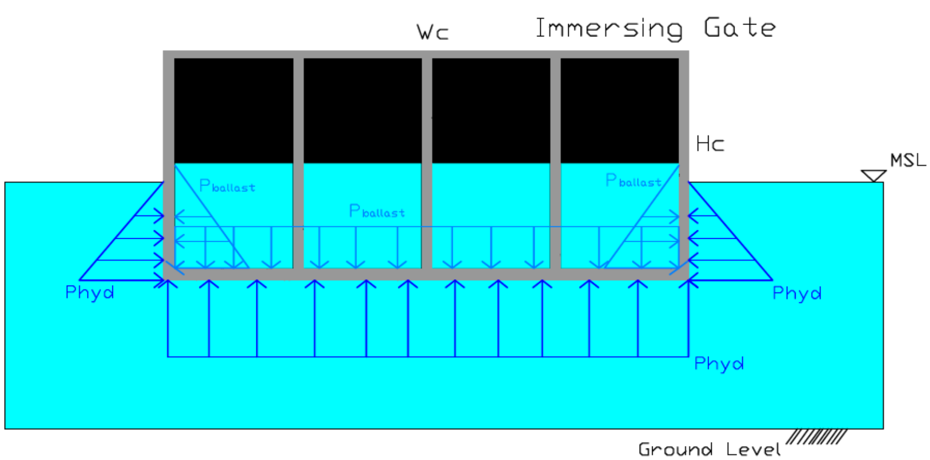


Figure 7-11: Ballasted and immersing caisson section view, dimensions not to scale

7.4.3 Design check 3: Gate immersed at Final Location During Normal Conditions

In this situation gate is totally immersed up to the desired draught level of MSL-17 m and it rests vertically on the foundations (Section 6.5.6). In the normal conditions, the gate should be able to resist the vertical forces which are mainly the weight of the gate and the weight of the ballasting water in the downward direction and the buoyancy force in the upward direction. The water level on both sides of the gate are equal so the horizontal forces are not considered here. In this case the overall structural analysis of the structure is going to be considered. The local stresses are more critical when the gate is empty as it is mentioned before.

The following design checks are used for this phase for overall forces on the gate:

- Shear stress checks (according to section 7.3.1)
- Bending moment stress checks (according to section 7.3.2)

The gate in this situation can be modeled as a beam located on two supports at its ends (Figure 7-12). See also Figure 6-6 for more information.

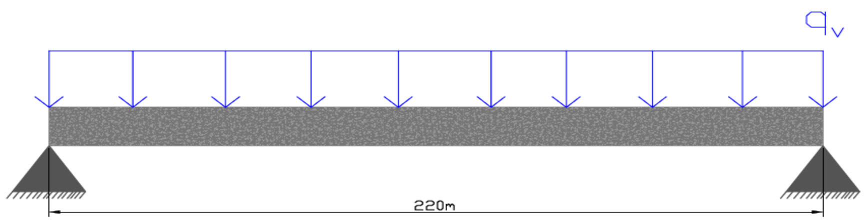


Figure 7-12: Gate modeled as a beam on two vertical supports

In this situation q_v, the uniform distributed load on the beam, can be considered as the component of the vertical forces. The results of the calculations for this design check can be found in section 14.1.3.

7.4.4 Design Check 4: Gate Immersed at Final Location During Hurricane Conditions

The situation when the gate is ballasted to its final position and it is the hurricane condition would be the most critical situation for the design of the caisson. This situation is shown in Figure 7-13. The calculations results can be seen in section 14.1.4.

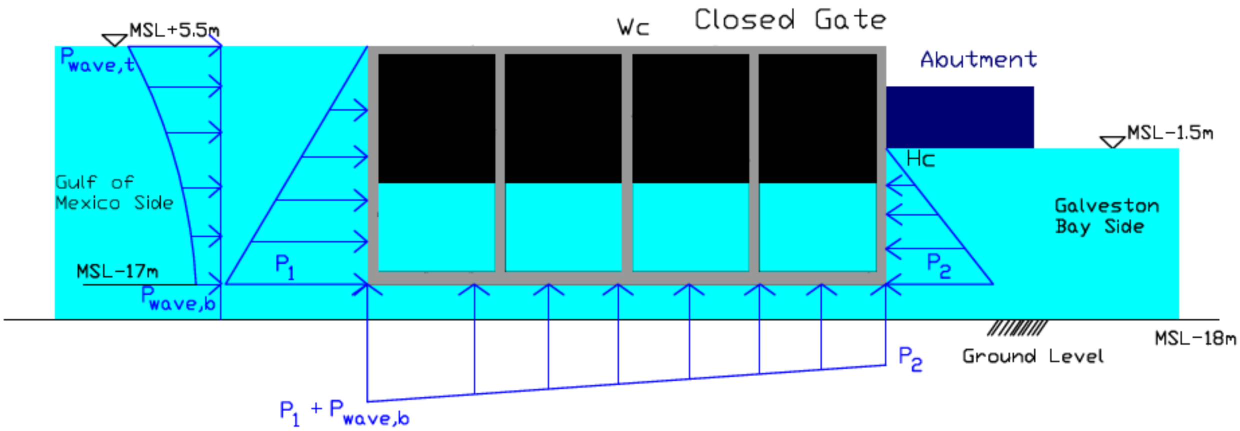


Figure 7-13: Gate immersed at final location during hurricane conditions cross section view, dimensions not to scale

The following design checks are used for this phase:

- Shear stress checks (according to section 7.3.1)
- Bending moment stress checks (according to section 7.3.2)
- Static stability checks (according to section 7.3.3)

Load cases:

In this phase, the loads are including hydrostatic pressure from the gulf side (P_1), hydrostatic pressure from the Bay side (P_2), wave load (P_{wave}) which has different values for top and bottom of the structure, and the hydrostatic pressure to the floor slab which is equal to the sum of hydrostatic pressure in the gulf side and wave load at the bottom of the structure in the Gulf side and in the Bay side it is equal to the P_2.

In the calculations the wave load has been considered with the trapezoid distribution on the wall of the structure. The ballasted water in the caisson also has the hydrostatic pressures on the walls and floor slab which are not shown here but are considered in the calculations.

Also, there is a suction force under the gate because of the hydraulic head differences in the sides of the gate and underflow. Figure 7-14 shows the situation of the gate at its final position with the opening under it [60].

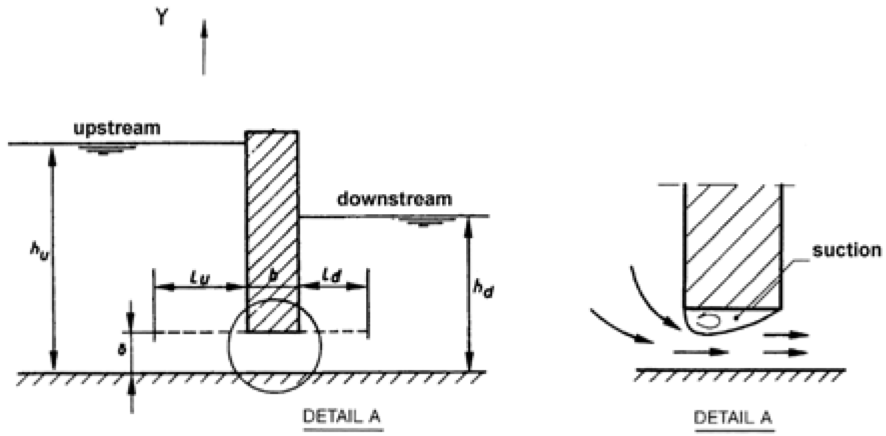


Figure 7-14: Diagram and symbols for the vibrating gate with flow-through discharge underneath [60]

The discharge of the submerged flow can be calculated from the equation below:

$$q = m\delta\sqrt{2g\Delta H} \qquad \text{Eq. 7-20}$$

In which:

- q = discharge per unit of width [$m^3/s/m$]
- m = discharge coefficient (Contraction coefficient)
- ΔH = hydraulic head [m]
- δ = height of the opening under the gate [m]

The suction force, in the downward direction, can be calculated from the equation below [60]:

$$F_s = C_s \,.\, \rho \,.\, g \,.\, \Delta H \qquad \text{Eq. 7-21}$$

In which:

- F_s = suction force [N]
- C_s = suction force coefficient

In general, three different design situations should be considered in the hurricane condition which are as follow.

7.4.4.1 Gate in Floating Condition During the Hurricane

The gate should be ballasted with the water to finally immersed and rests on the supports which are located on the ground. The amount of ballasted water within the caisson can be calculated in a way that the sum of vertical forces are positive in the downward direction.

The design checks in this situation initially starts with considering the caisson as floated and without resting on the support in the horizontal or vertical direction. In this situation, the local stresses are calculated and the design is done for these stress cases considering different ballasting heights within the caisson.

7.4.4.2 Gate supported on Abutments (Horizontal Supports) During the Hurricane

When the gate is ballasted and immersed to its final position, during the hurricane condition, it should be supported on the abutments in the horizontal direction. The length of the gate has been considered as 230 m which is 10 m more than the required 220 m of the opening which was calculated in the section 3.3.2. The idea behind this is that the gate has 5 m extra length in each end of it to rest on the abutments. Figure 7-15 shows the gate during the hurricane supported on the abutments.

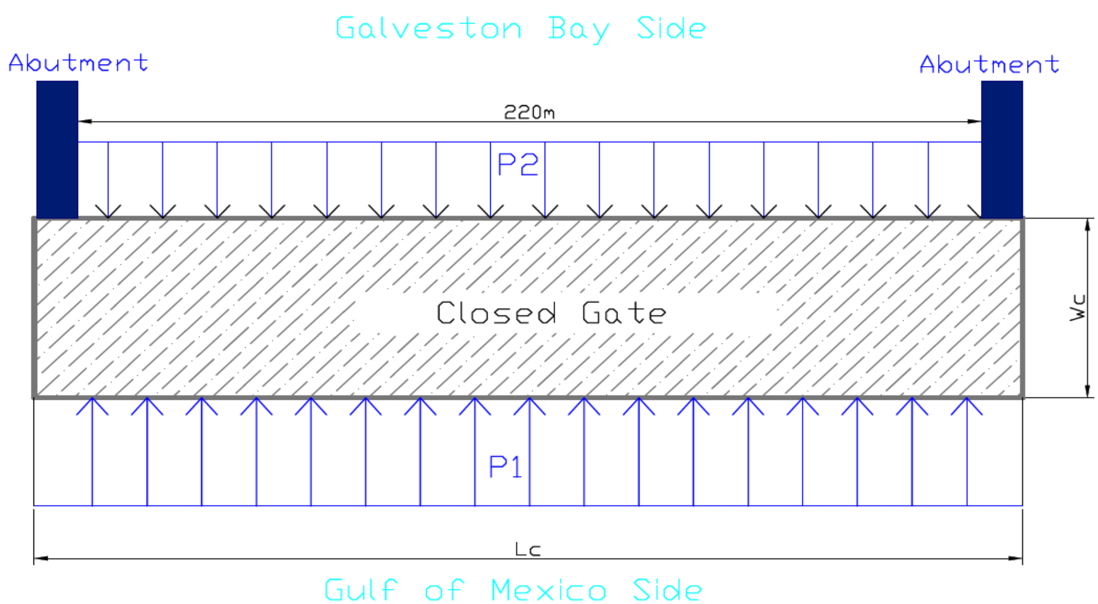


Figure 7-15: Barge gate supported on the abutments plan view, dimensions not to scale

The gate can be modeled here like a beam in which the cross section height is equal to the width of caisson W_c. The loads which are shown above are the P_1, the hydrostatic pressure plus wave pressure from the gulf side and the P_2, the hydrostatic pressure from the Bay side. In the design, the situation is going to be considered which is the most critical, so the highest difference between the amount of these two loads is going to be act as a uniform distributed load on the span of the L_c meter beam. In this way the rough evaluation of the maximum forces, stresses and the caisson wall thicknesses and reinforcement percentage to resist these stresses can be achieved. Figure 7-16 shows the simple model of the structure which is used in the calculation.

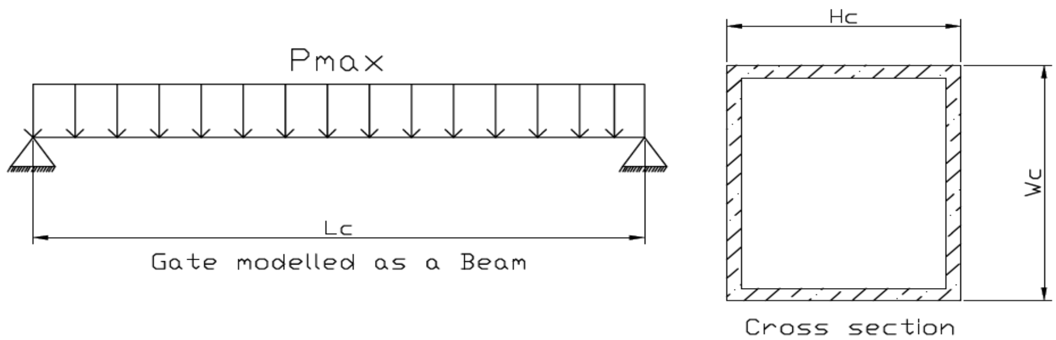


Figure 7-16: Gate modeled as a beam supported on the abutments

7.4.4.3 Gate Supported on Foundation (Vertical Supports) During the Hurricane

In this situation, the gate is ballasted and it rests on the foundation support which are located at two ends of the structure. Same as the previous section, 10 m of the length of the caisson is assumed more than the required opening and the gate can rest with 5 m on each end on the foundation supports. Figure 6-6 shows the gate at its final position supported by the foundations.

The gate can be modeled here like a beam with the cross section height of equal to the height of caisson H_c (Figure 7-12). The loads implemented to the downward are the weight of the caisson, the weight of the ballast and the suction force. The forces to the upward are the hydrostatic water pressure under the gate. These forces are going to be used in the design of the gate which is supported on two supports at each end. The maximum bending moment amount of the middle of the span is equal to $q_vL_c^2/8$ in which q_v is the uniform distributed load on the gate in the downward direction.

7.5 Final Preliminary Gate Design Results

On the basis of the design calculations in different situations, the final design of the structure and the optimizations regarding the dimensions and reinforcement, to fulfill the needs of all the situations, can be derived. In the optimizations, the dimensions of the gate should be adjusted in a way that each element would be structurally designed with the reinforcement within the regulated boundaries. The Table 7-5 to Table 7-8 are the summary of the results of the gate design.

The caisson is designed as a symmetric structure. The amounts of reinforcement are calculated for the maximum forces roughly. The reinforcement map can be calculated later with the softwares in the detailed design step in the future research. The calculations show that the design condition of the gate in final situation when it is supported horizontally during the hurricane is the most normative situation of the forces.

Final Barge Gate Geometry			
Parameter		Value	Unit
L_c	Length of the caisson	230	m
W_c	Width of the caisson	36	m
H_c	Height of the caisson	22,5	m
t_w	Thickness of the wall	1,1	m
t_f	Thickness of the floor slab	1,4	m
t_r	Thickness of the roof slab	0,5	m
t_{iw}	Thickness of the compartments	0,3	m
n_x	Number of compartments in width direction	5	-
n_y	Number of compartments in lenth direction	22	-
W_c	Caisson weight (with reinforcement)	70.779	ton

Table 7-5: Final barge gate geometry

Draught and Ballasting information			
Parameter		Value	Unit
d_i	Initial draught of the gate during transportation	7,88	m
$^{1}h_{ballast}$	Height of the ballast water in caisson normal condition	10,59	m
$^{2}h_{ballast}$	Height of the ballast water in caisson hurricane condition	15,00	m

Table 7-6: Draught and ballasting information

[1] This amount of ballast is desired for making the gate gets the draught of 17 m when it is normal condition
[2] This amount of ballast is desired for making the gate gets the draught of 17 m when it is hurricane condition

Stability checks		
Design Parameter	Value	Design Requirement
Static stability floating condition	8,36	should be > 0,5
Dynamic stability floating condition	2,55	should be > 2
[1]Static stability during immersing	6,88	should be > 0,5

Table 7-7: Stability checks in different operational situations of the gate

1 The static stability has been checked for different immersed depths. The current amount is for the desired final immersed level.

Structural Design checks		
Design Parameter	**Value**	**Design Requirement**
Design Check 1: Floating Caisson		
Shear check wall	19,16	should be > 1
Shear check floor	8,82	should be > 1
Shear check roof	18,10	should be > 1
Moment check wall	1,49	should be > 1
Moment check floor	1,01	should be > 1
Moment check roof	1,01	should be > 1
Reinforcement % wall	0,08	should be < 1
Reinforcement % floor	0,30	should be < 1
Reinforcement % roof	0,48	should be < 1
Design check 3: Gate immersed at Final Location During Normal Conditions		
Shear check floor	1,01	should be > 1
Moment check floor	1,12	should be > 1
Reinforcement % floor	0,78	should be <1
Design Check 4: Gate Immersed at Final Location During Hurricane Conditions		
Gate in Floating Condition During the Hurricane		
Shear check wall	1,96	should be > 1
Shear check floor	7,92	should be > 1
Shear check roof	20,11	should be > 1
Moment check wall	1,24	should be > 1
Moment check floor	2,75	should be > 1
Moment check roof	1,01	should be > 1
Reinforcement % wall	0,83	should be < 1
Reinforcement % floor	0,36	should be < 1
Reinforcement % roof	0,52	should be < 1
Gate supported on Abutments (Horizontal Supports) During the Hurricane		
Shear check wall	1,01	should be > 1
Moment check wall	1,12	should be > 1
Reinforcement % wall	0,97	should be < 1
Gate Supported on Foundation (Vertical Supports) During the Hurricane		
Shear check floor	1,01	should be > 1
Moment check floor	1,13	should be > 1
Reinforcement % floor	0,93	should be < 1

Table 7-8: Structural design checks in different operational situations of the gate

The stability design checks mentioned in Table 7-8 are as follow:

- Static stability floating conditions: Metacentric height of the structure in the floating condition which should be more than 0.5 m
- Dynamic stability floating condition: T_0 /$T_{p,reg}$ which should be more than 2 at least
- Static stability during immersion: Metacentric height of the structure during immersion which should be more than 0.5 m

The structural design checks mentioned in Table 7-8 are as follow:

- Shear check: Maximum allowed shear stress per running meter divided by maximum occurring shear stress per running meter ($\frac{\tau_d}{\tau_{dmax}} \geq 1$)
- Moment check: Maximum allowed bending moment stress per running meter divided by maximum occurring bending moment stress per running meter ($\frac{M_u}{M_{ed}} \geq 1$)
- Reinforcement percentage: Percentage of the designed reinforcement per running meter of the element divided by maximum allowable reinforcement per running meter of the element (ρ/ρ_{max})

The design which is performed in this chapter includes the assumptions of the hydrostatic situation and SLS. The hydrodynamic analysis of the structure is recommended in the future research.

7.6 Conclusions

In this chapter the design of the barge gate is performed. The gate is designed with the HPLC material. Different situations have been considered for the design of the gate including the transportation, floated gate, during the immersion and at the final situation when the gate is supported with horizontal and the vertical supports during the hurricane. In each of these situations the loads are different and different shear stresses and bending moments occur. The design has been done by considering the critical situations and the proposed structure will satisfy the needs for different operational phases.

The gate is considered as the caisson structure and the dimensions of the structure and the amount of reinforcement to resist the loads are calculated (This can be seen in appendices for different elements). However, for more precise results and reinforcement plan use of the 3D software is recommended in the future research. Also, the combination of the horizontal and vertical supports in the critical situation needs to be further investigated.

The gate has the dimension of 230 m * 36 m * 22.5 m (Figure 7-17 and Figure 7-18). The initial draught of the structure is equal to 8.02 m. The weight of the gate is 70,779 tons.

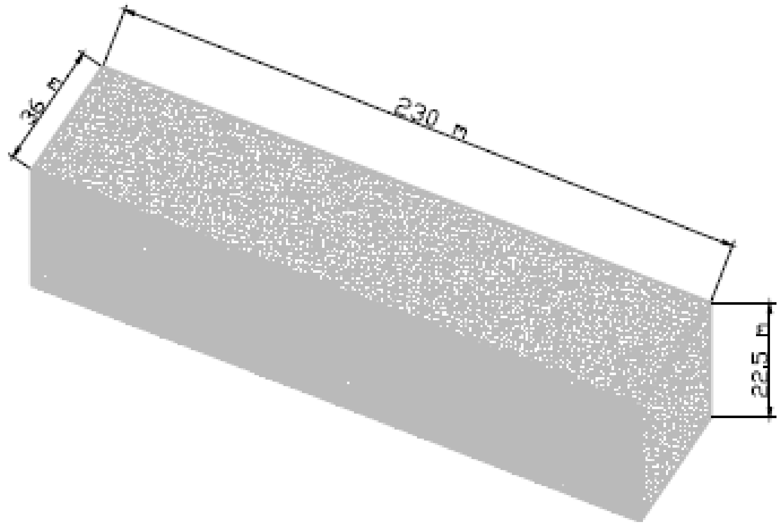


Figure 7-17: Final design for the barge gate, 3D view

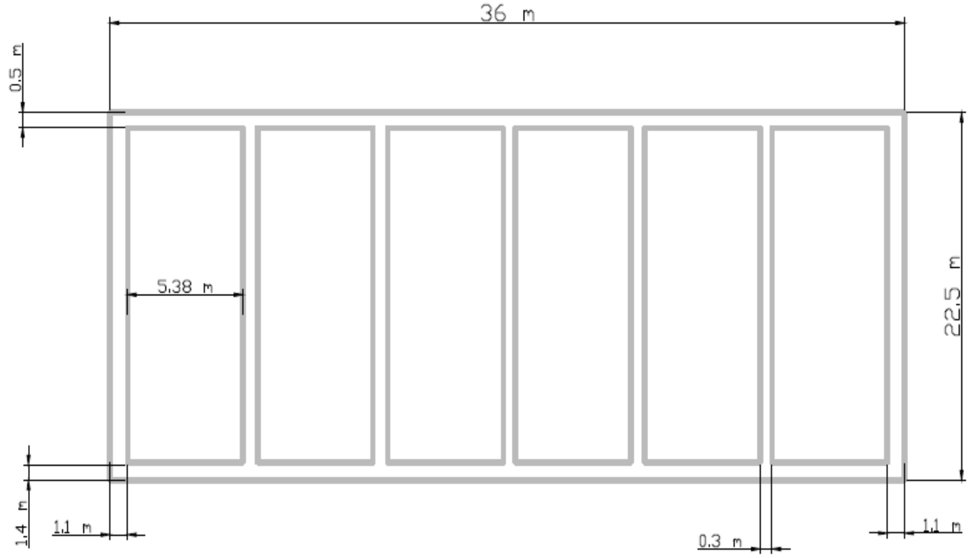


Figure 7-18: Final design for the barge gate, cross-section

Regarding the design requirements, Table 7-9 shows the status of the requirements after DS3.. The results of analysis show that the barge gate is a realizable solution. Reliability and economical aspects should be checked in the later steps of the design.

Program of Requirements for the Navigational Barrier (After DS3)		
Type	**Criteria**	**Status after DS3.**
General	Open in normal conditions	Expected to be ok
	Closed during the hurricane conditions	Expected to be ok
	Realizeable solution	Expected to be ok
	Operable solution	Expected to be ok
	Reliable solution	Unknown
	Economical solution	Unknown
Navigational	Enough width	Expected to be ok
	Enough depth	Expected to be ok
	Enough air draft	Expected to be ok
Current velocity	Acceptable current velocity in navigational channel	Expected to be ok
Safety level	Enough retaining height	Expected to be ok
Lifetime	Enough lifetime	Expected to be ok
	Durable solution	Expected to be ok
Environmental	Environmentaly friendly solution	Expected to be ok

Table 7-9: Status of requirements after DS3.

This chapter has considered only the hydrostatic design of the barge gate. The following tasks are recommended for the future research:

- Including the hydrodynamic effects and loads in the design
- Optimizations of the elements and reinforcement map using the FEM softwares
- Checking the strength of the internal walls for different levels of ballasting water in compartments

8 Complementary Structures Designs (DS4.)

8.1 Introduction

In this chapter the design step 4 and its sub-steps are considered (Figure 8-1). Due to the time constraints, only the design of the bed protection, berthing system and articulation system are considered here. Design of the ballasting system, hydraulic breaking system and etc. are out of the scope of this report.

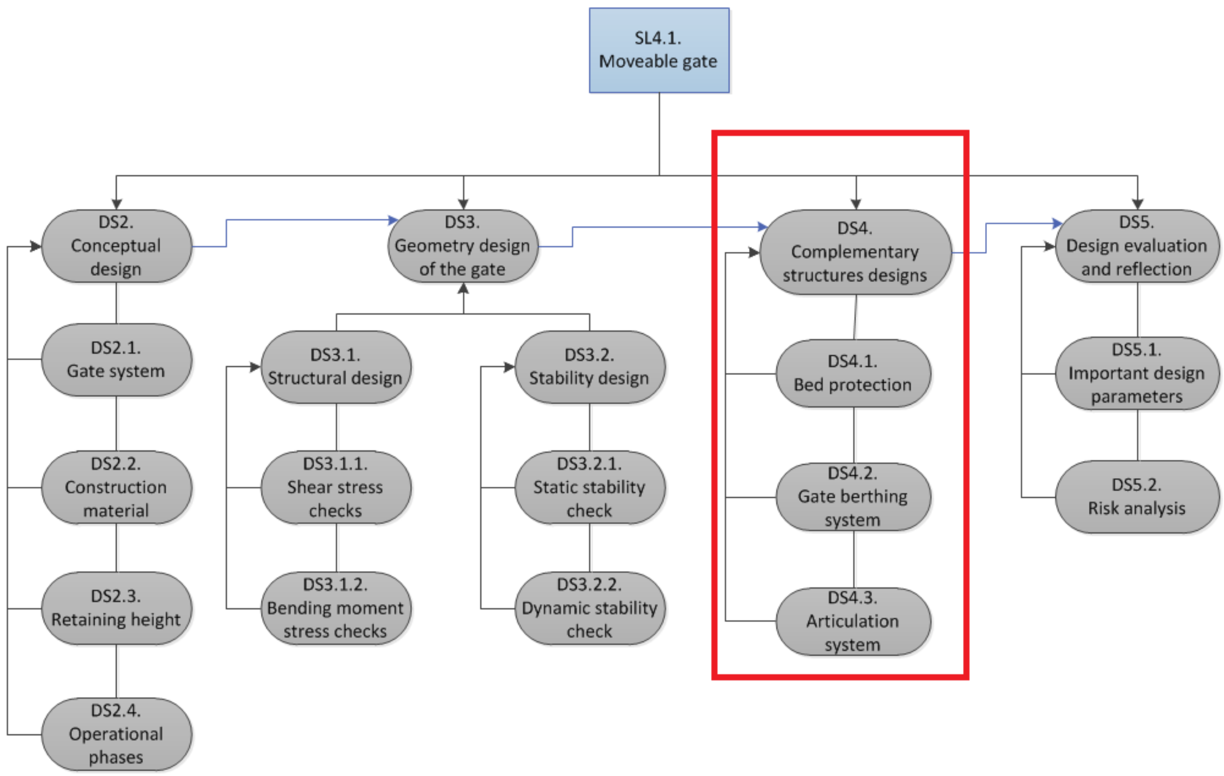


Figure 8-1: Design step 4; Complementary structures designs

8.2 Bed Protection (DS4.1.)

8.2.1 Introduction

The boundaries of water bodies should be protected against the current attack otherwise they will lose their shapes and ultimate purpose due to the erosion. Design of the bed protection in the navigational section of the barrier is then another important aspect of the current project. In this section this issue is investigated in more detail. The scour depth during the critical conditions should be calculated. Then the bed protection should be designed to prevent the scouring.

8.2.2 Scour Depth Under the Gate

Flow under a gate or barrier has a considerable potential for scour [62]. The flow under the gate can be assumed as the submerged jet. In general the form of the scour depends on a number of factors such as submergence and the degree of dissipation of the jet energy. For the current project, the situation is considered as the two-dimensional jet situation for the initial estimation.

There are many empirical and semi-empirical equations for calculation of the scour depth. A useful equation is from Qayoum (1960) which is the result of a study of the flow under gates with no bed protection. The following empirical equation is proposed for calculation of the equilibrium scour depth [62]:

$$y_{m,e} + h_t = 2.78 \frac{q^{0.4} H^{0.22} h_t^{0.4}}{g^{0.2} d_{90}^{0.22}} \qquad \text{Eq. 8-1}$$

In which:

- d_{90} = particle diameter for which 90% of the mixture is smaller than d_{90} [m]
- g = acceleration of gravity [m/s^2]
- h_t = tailwater depth [m]
- H = difference in height between upstream and downstream water levels [m]
- q = discharge per unit width [m^2/s]
- $y_{m,e}$ = equilibrium scour depth [m]

The normative condition for the calculations is considered as when the maximum positive head is present (Figure 8-2).

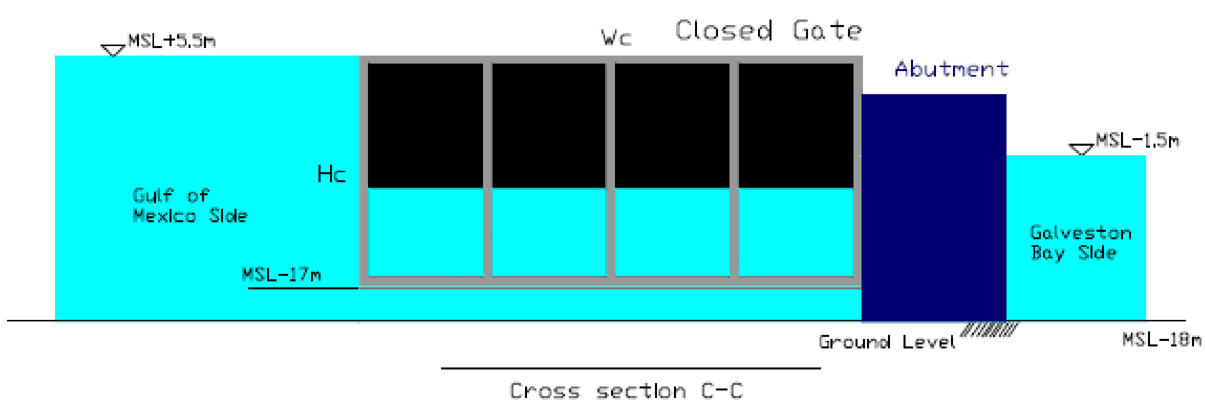


Figure 8-2: Normative situation for the calculation of the bed protection system

The current velocity under the gate in critical condition of the hurricane is calculated as <u>7.1 m/s</u>. In case of the Bolivar Roads storm surge barrier, the geotechnical data was mentioned in Table 3-2. As it can be seen in this figure the first 3 meters of the ground is very soft clay which can be assumed as the grain size of about 0.002 millimeter [63]. The results of the analysis show that this layer cannot resist the high current velocity under the gate. Indeed, the scour depth is calculated with the data from the next layer, which is a sand layer.

The results of analysis can be seen in Table 8-1. It is obvious from the results that the bed protection is needed. Also, because of the large predicted scour depth, the above mentioned equation can only be used for first estimates. Further research about the time-scale and 3D modeling of the situation is recommended.

Scour Depth Calculations			
Parameter		Value	Unit
H	Water level difference	7,00	m
u	Current velocity	7,10	m/s
$y_{m,e}$	Equilibrium scour depth	7,57	m

Table 8-1: Scour depth calculations under the gate

The calculations regarding the scour depth can be found in section 14.3.1. Obviously when the opening under the gate is bigger, then the underflow velocity will be reduced and consequently the scour depth would be smaller. This could be an opportunity for the optimization of the system.

8.2.3 Bed Protection Design Theory

The bed protection is needed for the ground of the navigational section of the storm surge barrier. Different systems can be used in this regard including armourstone and rip-rap, block stone, hand-pitched stone, grouted stone, gabions (box gabions, gabion mattresses, sack gabions) and bituminous materials [64].

There are many stability formulas for various types of bed protection systems. One of the most used equations are the Pilarczyk (1995) formula. A unified relationship between the armourstone size for stability and the hydraulic and structural parameters is proposed which combines various design formula. The equation below should be used [64]:

$$D = \frac{\phi_{sc}}{\Delta} \frac{0.035}{\psi_{cr}} k_h k_{sl}^{-1} k_t^2 \frac{u^2}{2g} \qquad \text{Eq. 8-2}$$

In which:

- D = characteristic size of the protection element [m] (D_{50})
- ϕ_{sc} = stability correction factor
- Δ = relative buoyant density of the protection element
- Ψ_{cr} = critical mobility parameter of the protection element

- k_t = turbulence factor
- k_h = velocity profile factor
- k_{sl} = side slope factor
- u = flow velocity [m/s]

The parameters of the equation above can be derived by using the following design guidance (Table 8-2).

Characteristic size, D	• armourstone and rip-rap: $D = D_{n50} \cong 0.84 D_{50}$ (m) • box gabions and gabion mattresses: D = thickness of element (m) **NOTE:** The armourstone size is also determined by the need to have at least two layers of armourstone inside the gabion.
Relative buoyant density, Δ	• rip-rap and armourstone: $\Delta = \rho_r/\rho_w - 1$ • box gabions and gabion mattresses: $\Delta = (1 - n_v)(\rho_r/\rho_w - 1)$ where n_v = layer porosity $\cong$ 0.4 (-), ρ_r = apparent mass density of rock (kg/m^3) and ρ_w = mass density of water (kg/m^3)
Mobility parameter, ψ_{cr}	• rip-rap and armourstone: $\psi_{cr} = 0.035$ • box gabions and gabion mattresses: $\psi_{cr} = 0.070$ • rock fill in gabions: $\psi_{cr} < 0.100$
Stability factor, ϕ_{sc}	• exposed edges of gabions/stone mattresses: $\phi_{sc} = 1.0$ • exposed edges of rip-rap and armourstone: $\phi_{sc} = 1.5$ • continuous rock protection: $\phi_{sc} = 0.75$ • interlocked blocks and cabled blockmats: $\phi_{sc} = 0.5$
Turbulence factor, k_t	• normal turbulence level: $k_t^2 = 1.0$ • non-uniform flow, increased turbulence in outer bends: $k_t^2 = 1.5$ • non-uniform flow, sharp outer bends: $k_t^2 = 2.0$ • non-uniform flow, special cases: $k_t^2 > 2$
Velocity profile factor, k_h	• **fully developed logarithmic velocity profile:** $k_h = 2/\left(\log^2(1+12h/k_s)\right)$ where h = water depth (m) and k_s = roughness height (m); k_s = 1 to $3D_n$ for rip-rap and armourstone; for shallow rough flow ($h/D_n < 5$), $k_h \cong 1$ can be applied • **not fully developed velocity profile:** $k_h = \left(1 + h/D_n\right)^{-0.2}$
Side slope factor, k_{sl}	The side slope factor is defined as the product of two terms: a side slope term, k_d, and a longitudinal slope term, k_l: $k_{sl} = k_d\, k_l$ where $k_d = (1 - (\sin^2\alpha/\sin^2\phi))^{0.5}$ and $k_l = \sin(\phi - \beta)/(\sin\phi)$; α is the side slope angle (°), ϕ is the angle of repose of the armourstone (°) and β is the slope angle in the longitudinal direction (°)

Table 8-2: Design guidance for parameters in the Pilarczyk design formula [64]

The horizontal dimensions of the bed protection for the scour holes should also be assessed. Pilarczyk has studied the bed protection effects on the sediment transport and he has found that the stones in the top layer of the bottom protection cause turbulence of the water flow. This leads to considerable sediment transport through the protection layer and causes erosion below this layer which makes the development of a scour hole in front of the bottom protection [58].

The scour holes is mainly characterized by upper scour slope (β) and the maximum scouring depth ($y_{m,e}$) (Figure 8-3).

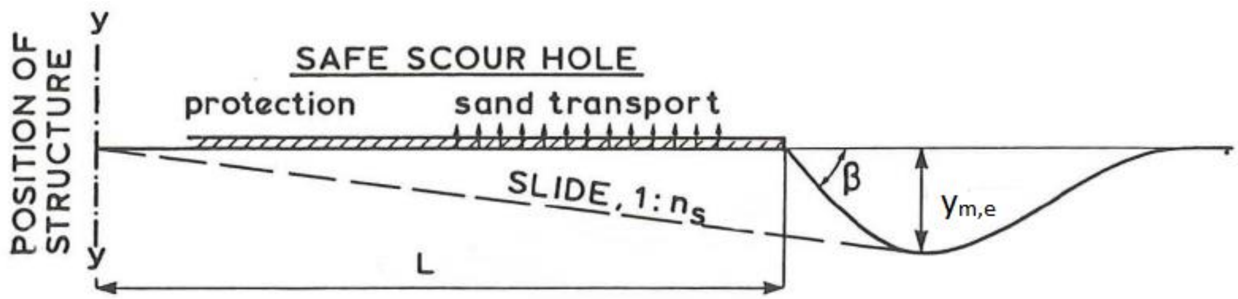


Figure 8-3: Length of bottom protection [58]

For a preliminary design and first estimate, the required length of the bottom protection should satisfy the following condition:

$$L_{b,pro} \geq S \,.\, n_s \,.\, y_{m,e} \qquad \text{Eq. 8-3}$$

in which:

- S = safety factor
- 1:n_s = average slope of the slide

In the calculation n_s can be assumed as 6 for densely packed or cohesive material and 15 for loosely packed material.

8.2.4 Bed Protection Design Results

The analysis can be done for the hurricane situation or for the normal conditions. In the normal conditions, the analysis from Ruijs (2011) shows that the current velocity in the navigation section due to 40% restriction of the flow area of the environmental barrier will reach 1.3 m/s [6].

The calculations have been done for two different bed protection systems; armourstone or rip-rap and the box gabions or gabion mattresses. The system has been considered as interlocked blocks and cabled blockmats. The non-uniform flow pattern is assumed under the gate with not fully developed velocity profile. The density of the rock material is considered as 2800 (kg/m^3). The results of analyses have been shown in Table 8-3.

Bed Protection Calculations				
Parameter		Value		Unit
		Hurricane condition	Normal condition	
D	Rip-rap and armourstone	1,09	0,03	m
D	Box gabions and gabion mattresses	0,91	0,03	m

Table 8-3: Bed protection calculations results

The table shows that both systems can be realizable for the current velocity during the hurricane condition. However, these systems need to be constructed with interlocked blocks. Obviously by realization of the bed protection for the hurricane condition, the needs of the normal conditions would be also satisfied.

Table 8-4 can be used to choose the type of the rock material for the bed protection [65].

Sortering	D_{50} [m]	M_{50} [kg]	D_n [m]	D_{85} [m]	D_{15} [m]	D_{85}/D_{15} (gem.)
30/80 mm	0,045 - 0,065	0,15 - 0,45	0,038 - 0,055	0,085 - 0,064	0,048 - 0,024	2,22
40/100 mm	0,063 - 0,090	0,4 - 1,2	0,053 - 0,075	0,118 - 0,086	0,067 - 0,038	2,01
50/150 mm	0,090 - 0,125	1,2 - 3,1	0,075 - 0,110	0,173 - 0,125	0,096 - 0,054	2,06
80/200 mm	0,125 - 0,180	3,1 - 9,3	0,11 - 0,15	0,226 - 0,169	0,136 - 0,075	1,96
5 - 40 kg	0,20 - 0,25	13 - 26	0,17 - 0,21	0,33 - 0,28	0,20 - 0,16	1,70
10 - 60 kg	0,25 - 0,31	26 - 46	0,21 - 0,26	0,36 - 0,32	0,25 - 0,20	1,52
40 - 200 kg	0,38 - 0,44	90 - 140	0,32 - 0,37	0,54 - 0,48	0,37 - 0,31	1,50
60 - 300 kg	0,45 - 0,52	150 - 220	0,38 - 0,43	0,61 - 0,55	0,43 - 0,36	1,47
300 - 1000 kg	0,72 - 0,78	595 - 760	0,60 - 0,66	0,92 - 0,83	0,66 - 0,60	1,39
1000 - 3000 kg	1,04 - 1,11	1800 - 2200	0,87 - 0,93	1,31 - 1,20	0,97 - 0,88	1,36
3000 - 6000 kg	1,40 - 1,47	4400 - 5050	1,18 - 1,23	1,64 - 1,51	1,32 - 1,26	1,22
6000 - 10000 kg	1,70 - 1,77	7850 - 8900	1,43 - 1,49	1,95 - 1,80	1,63 - 1,57	1,17

Table 8-4: Characteristics of grain diameter of standard grading of armourstone [65]

According to the table above, the rock material of 1 to 3 tons is required for the bed protection almost in both systems. The filter layers would be required for the bed protection. The filter mattress can be used with a very low failure probability in case of the failure of the rock material. For example a geometrically closed granular filter can be used [79].

Regarding the bed protection length, for the first estimate, considering the safety factor of S=1.5 and the densely packed material (n_s=6) for the bed protection, the length of almost $L_{b,pro}$=70 m is required from each side of the gate.

The calculations for the bed protection design can be seen in section 14.3.2.

8.2.5 Bed Protection Conclusions

In this section the scour protection under the gate is evaluated. Firstly, the scour depth is calculated. Considering the loose sand under the gate and high velocity of the flow in critical conditions under the gate, the scour hole depth is a considerable amount (7.57 m). In the normal conditions the velocity of the flow is significantly less than in the critical conditions and the problem is less.

The bed protection design has been done on the basis of the design formula of Pilarczyk for both normal condition and hurricane condition. In the hurricane condition the rock fill in gabions system is the best system. However, the significant rock material with 1 to 3 tons is required with the approximate length of 70 m from each side of the gate and filter layers.

The analysis which has been done here is just the preliminary design of the bed protection. For more precise design, it is recommended that the time-scale of the scour would be evaluated by an appropriate software or the laboratory models.

8.3 Gate Berthing System (DS4.2.)

8.3.1 Introduction

During the closure procedure, the gate will berth on the abutment with high forces. The forces of the berthing should be taken by a group of tyres and Teflon skids which acts like fenders. These fenders are located at the ends of the gate. In this section the fenders design and the forces of closure are going to be treated.

8.3.2 Fender Type

The fender system consists of tyres and Teflon skids which get the berthing energy of the gate and guide it to rest on the foundations finally. This system would be a complicated system and it should be tailor-made on the basis of the requirements of the project. However, for the preliminary design the fender is assumed to be wheel fenders.

Wheel fenders are used widely in the marine environment [61]. They are mostly used on exposed corners to help ships maneuver into berths and narrow channels such as locks and dry-dock entrance. For the current project, they may be adjusted to roll in the vertical direction.

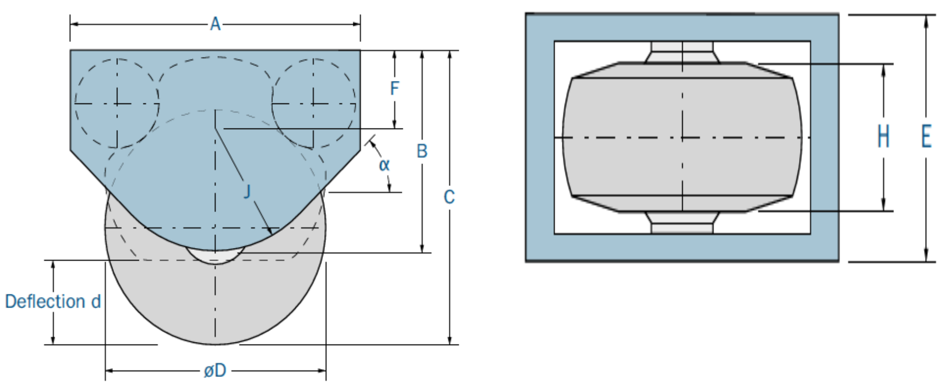
In this fender type, the main axle slides on bearings and the wheel recast against back rollers to provide high energy and minimal rolling resistance [61]. The main features of these fenders are:

- High energy absorption
- Very low rolling resistance
- Useable singly or in multiple stacks
- Composite and stainless steel bearing
- Low maintenance casing design

The above mentioned features make this fender type suitable for the current project. There is a need for low rolling resistance of the fenders while the group of fenders should be used. Maintenance cost is the other important factor which is favorable in case of this fender type.

Figure 8-4: Wheel fenders [61]

The typical wheel fender casing dimensions can be seen in Figure 8-5. For the special applications such as the current project, the casing shape can be adjusted for a perfect fit.

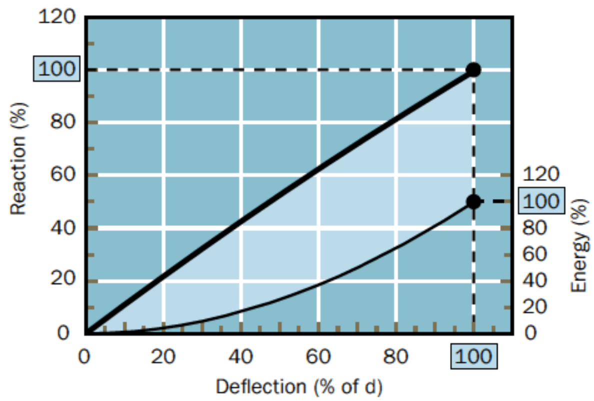


Fender	A	B	C	D	E	F	H	J	K	L	α
110-45WF	1700	1000	1450	1080	900	350	460	650	50	150	0–40°
130-50WF	2000	1200	1750	1300	1000	350	510	850	50	200	0–40°
175-70WF	2650	1500	2200	1750	1150	550	690	950	50	200	0–40°
200-75WF	2750	1750	2550	1980	1250	500	760	1250	50	250	0–45°
250-100WF	3350	2200	3200	2550	1600	850	970	1350	50	250	0–45°
290-110WF	4200	2500	3750	2900	1700	1000	900	1500	50	250	0–45°

[Units: mm]

Figure 8-5: Typical wheel fender casing dimensions [61]

The reaction, energy and deflection information of this fender system is shown in Figure 8-6.



Fender	Energy (kNm)	Reaction (kN)	Deflection (mm)	Pressure (bar)
110-45WF	33	150	400	5.5
130-50WF	61	220	500	3.5
175-70WF	100	315	600	4.8
200-75WF	220	590	700	5.5
250-100WF	440	920	925	5.5
290-110WF	880	1300	1200	5.8

Figure 8-6: Reaction, energy and deformation of a wheel fender

In the final position of the barge gate the wheel fender in the structure would be like Figure 8-7.

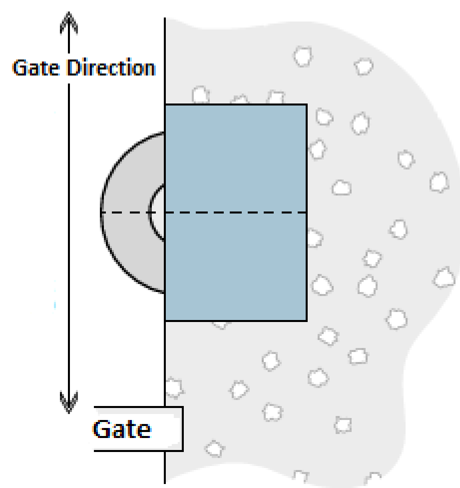


Figure 8-7: Wheel fender system in the final structure

In the next step, firstly the berthing energy of the gate is going to be calculated and then the suitable wheel fender type is going to be selected for the structure.

8.3.3 Fender Design Theory

For the fender design, firstly the force and the berthing energy should be calculated. The berthing energy of the gate should be resisted by the wheel fenders and finally by the abutments.

The berthing force on the fender can be calculated by the formula for the force on a linear elastic structure [58]:

$$F_{bth} = \sqrt{2kE_{kin}} \qquad \text{Eq. 8-4}$$

In which:

- F_{bth} = the berthing force of the gate [kN]
- k = the stiffness of the structure [kN/m]
- E_{kin} = the kinetic energy of the berthing of the gate [kNm]

The energy of berthing can be calculated from equation below:

$$E_{kin} = (1/2) \,.\, m_g \,.\, v_g^2 \,.\, C_H \,.\, C_E \,.\, C_S \,.\, C_C \qquad \text{Eq. 8-5}$$

In which:

- m_g = mass of the gate [kg]
- v_g = velocity of the gate and water [m/s]
- C_H = hydrodynamic coefficient
- C_E = eccentricity coefficient
- C_S = softness coefficient
- C_C = configuration coefficient

In the following section each of the variables of the equations above are going to be described.

Stiffness of the structure (k):

The stiffness of the structure k depends on the abutment and the fender. The abutment can be assumed as very stiff and then the fender is normative [58]. The stiffness of the wheel fender can be derived from the information in the Figure 8-5.

Velocity of the gate (v_g):

The velocity of the gate during the closure procedure is the other aspect. The berthing speed of the gate can be related to the dimensions and the weight of the gate. The design velocity for the different ship sizes have been given in Figure 8-8. This graph can be used for the gate velocity during the closure because the floating gate can be assumed as a ship.

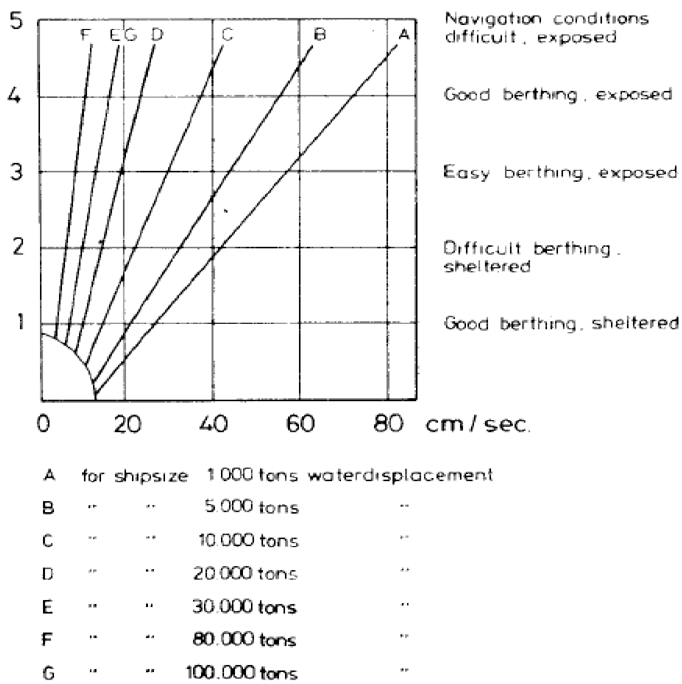


Figure 8-8: Design velocity [58]

Hydrodynamic coefficient (C_H):

The hydrodynamic coefficient is the ratio between the mass of the gate (m_g) plus the water moving with the gate (m_w) and the mass of the gate and can be calculated from formula below:

$$C_H = \frac{m_g + m_w}{m_g} \qquad \text{Eq. 8-6}$$

Mass of the gate can be assumed as the total mass of the caisson structure which is in conservative side. The additional mass of the water for the preliminary design can be calculated from equation below:

$$m_w = (1/4)\, .\, \rho\, .\, L\, .\, \pi\, .\, D^2 \qquad \text{Eq. 8-7}$$

in which:

- ρ = density of seawater [kg/m^3]
- L = length of the gate
- D = Draught of the gate

The draught of the gate for the preliminary design is assumed to be equal to the initial draught of the caisson.

Eccentricity coefficient (C_E):

This coefficient takes into the account the energy dissipation caused by the yawing of the gate when it moors eccentrically the structure. The yawing of a typical ship is shown in Figure 8-9.

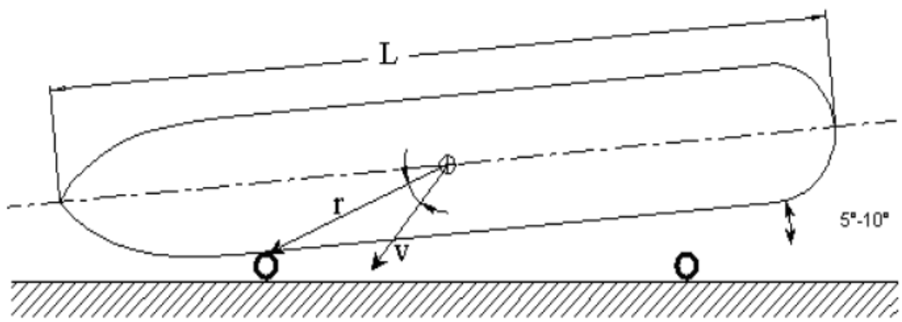


Figure 8-9: Berthing eccentrically [58]

The coefficient of eccentricity can be calculated as:

$$C_E = \frac{k^2 + r^2 cos^2(\gamma)}{k^2 + r^2} \qquad \text{Eq. 8-8}$$

in which:

- k = radius of gyration of the gate
- r = the radius between the center of mass of the gate and the point of collision between the gate and the structure
- ɣ = the angle between radius r and the velocity of the gate

The radius of gyration of the ship can be approximated as:

$$k = (0.19 \; . \; C_b + 0.11) \; . \; L \qquad \text{Eq. 8-9}$$

in which:

- C_b = block coefficient = I/LBD
- B = width of the gate
- D = draught of the gate
- I = (volumetric) water displacement of the gate

The C_b can be also approximated as:

$$0.6 \text{ (slender ships)} \leqslant C_b \leqslant 0.95 \text{ (container ships)} \qquad \text{Eq. 8-10}$$

Softness coefficient (C_S):

This coefficient takes into account the elasticity of the gate's side and it depends on the stiffness of the structure and that of the gate's shell and takes into account the part that is taken by the structure. In case of the stiff structure this coefficient can be assumed as 0.9 for the design purposes.

Configuration coefficient (C_C):

This coefficient takes into account the hydrodynamic friction. This is caused by the water mass between the gate and the structure. For safety reasons this coefficient can be assumed as 1.0 for a preliminary design.

8.3.4 Fender Design Results

The fenders are assumed as wheel fenders as it is mentioned before. The exact number of fenders in the vertical direction can be determined in the later steps. However, in the preliminary design the numbers of the fenders in the 8 m height of the abutments (equal to initial draught of the gate) can be defined by considering the type and size of the fender and the amount of energy that it can take. The wheel fender type of 200-75WF has been considered in the initial calculations. The summary of calculations are shown in Table 8-5. The detailed calculations in this regard are included in section 14.4.

Summary of the Fender Design Results			
Parameter		Value	Unit
K_{fender}	Stiffness of the fender	843	kN/m
v_g	Design velocity of the gate during closure	0,2	m/s
C_H	Hydrodynamic coefficient	1,20	-
C_E	Coefficient of eccentricity	0,50	-
C_S	Softness coefficient	0,9	-
C_C	Configuration coefficient	1	-
N_f	Number of fenders in vertical direction acting as energy observer	3	-
$E_{kin, t}$	Total berthing kinetic energy	509	kNm
$E_{kin, f}$	Berthing kinetic energy of each fender	170	kNm
$F_{bth,t}$	Total berthing force	926	kN
$F_{bth,f}$	Berthing force of each fender	309	kN

Table 8-5: Summary of the fender design

The berthing force calculated in this section should be also considered in the design of the abutments. It should be mentioned that the calculations which are done in this report are just the initial steps in the design of the fender system. The design of the complete system of the fenders and Teflon skids needs more effort and more data which is out of the scope of the current report. For sure the berthing forces can be reduced later on the basis of the other types of the fenders which would be useful in making the project more economical. However, in this report the design will go further with the current assumptions which are on the conservative side.

8.3.5 Berthing System Conclusions

In this section the fendering system of the gate is designed. The fenders are considered as the wheel fenders and they are designed for the closure energy of the gate when it has the initial draught (≈7 m) of the structure.

To reduce the energy of closure, the gate should be closed with the minimum of the draught because the bigger draught will increase the energy of the closure and consequently the closure forces on the abutment and the fenders. Also, the velocity of the gate during the closure should be controlled using the propellers and cables. The velocity which is considered here is proposed by the guidelines for the ships in the same dimensions. If the velocity of the closure is different, the berthing force should be recalculated.

In the current design only one row of fenders is considered which can be later on increased to two rows of fenders. However, currently 5 m is considered for the interaction section of the gate and abutments. In case of using two rows of fenders, this amount should be increased which will change the length of the gate and also the total design. Indeed, if the change in the numbers of the rows of the fenders will apply, the whole design should be repeated again.

8.4 Articulation System (DS4.3.)

8.4.1 Introduction

The articulation system or the swing point is the structure which the gate rotates around it during the closure (Figure 8-10). It should provide enough degrees of freedom for the movement of the gate during the closure and during the sinking in the final situation.

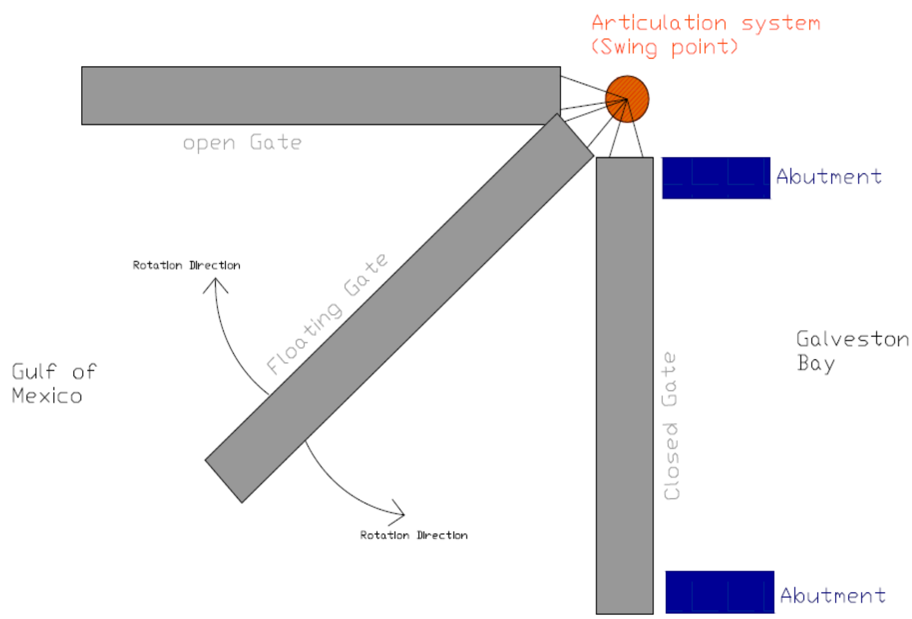


Figure 8-10: Articulation system (Swing point)

The possible motions of a floating body is shown in Figure 8-11. The definition of the motions are as below:

- **Surge:** a horizontal translation along the longitudinal x-axis of the body
- **Sway:** a horizontal translation along the transversal y-axis of the body
- **Heave:** a vertical translation along the vertical z-axis of the body
- **Roll:** a rotation around the longitudinal x-axis
- **Yaw:** a rotation around the vertical z-axis
- **Pitch:** a rotation around the transversal y-axis

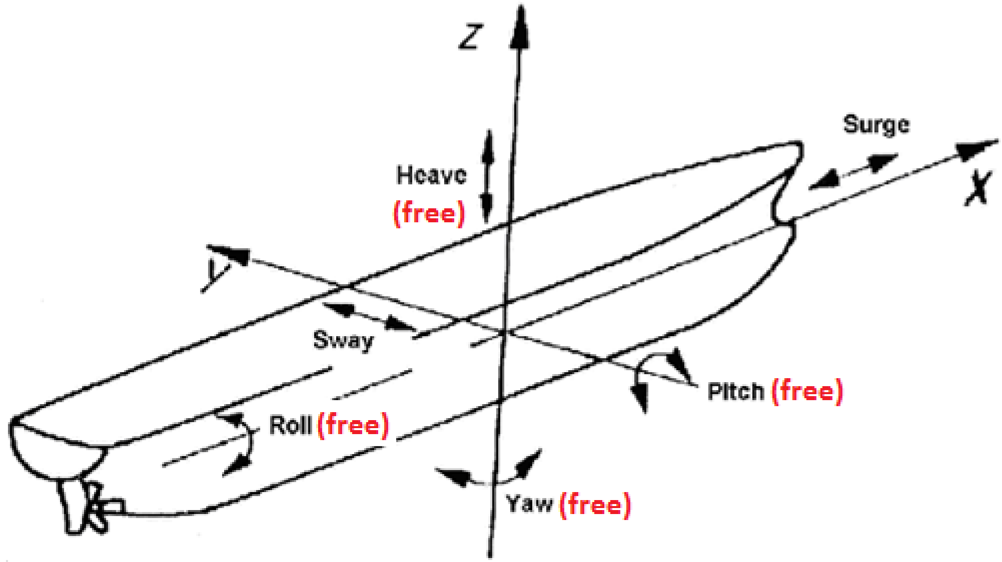


Figure 8-11: Motions of a floating structure and required freedoms for the articulation system

Considering the operational phases of the barge gate, the gate should freely rotate around the swing point (yaw motion), slides vertically freely to be able to immersed at the final location (heave motion) and should have the freedom in roll and pitch motions to provide the free floating conditions during the closure. This is suitable because the wave actions and other imbalances are resisted by the buoyancy force equilibrium of the gate instead of by the articulation system. The gate articulation system should at least satisfy these degrees of freedom. Indeed, only surge and sway motions should be restricted by the articulation system. In short:

- **Surge:** restricted to keep the gate connected to the articulation system (See Figure 8-12)
- **Sway:** restricted to prevent the gate from moving in the y direction during the closure (See Figure 8-13)
- **Heave:** free to provide vertical movement of the gate at final location
- **Roll:** free to provide the floating requirements of the gate due to the motions of the structure during closure and immersion
- **Yaw:** free to provide rotation of the gate around the swing point
- **Pitch:** free to provide the floating requirements of the gate due to the motions of the structure during closure and immersion

Figure 8-12 and Figure 8-13 show the movement of the gate during the closure procedure under the current forces if the surge and sway motions are not restricted by the articulation system in the swing point.

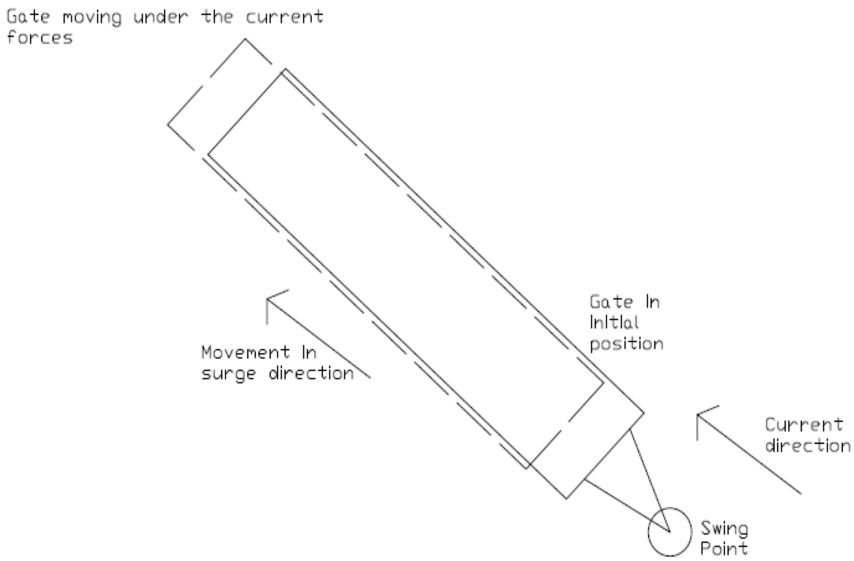


Figure 8-12: Movement of the gate during the closure if the surge motion is not restricted

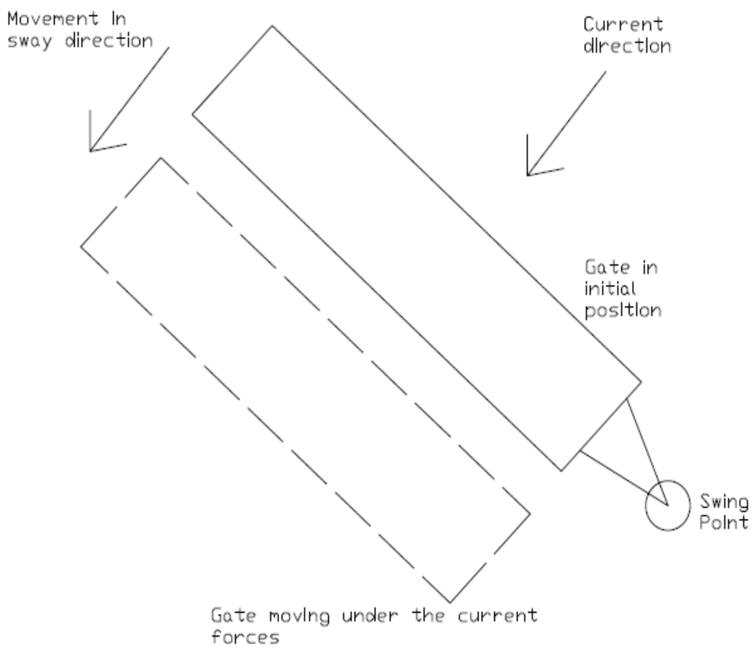


Figure 8-13: Movement of the gate during the closure if the sway motion is not restricted

8.4.2 Possible Articulation Systems

In this section possible systems for the articulation systems (swing point) are going to be evaluated. For getting an idea about the possible options, some project examples with almost the same requirements are going to be described.

8.4.2.1 Bayou Lafourche Swing Gate

This barge gate with the length of 25 meter (for more information see section 4.2.10) has the swing arm pivot system which connects the barge gate to the pivot piling [43]. The swing arm consists of two basic parts including the pivot assembly and the support arm. The pivot assembly allows three degrees of freedom of movement for the gate just like the desired system for the current project.

The steel pivot mechanism is a three-pronged barge connection with a translation-fixed gooseneck including a free-to-rotate annular attachment to the pivot pile fitting [43]. The pivot pile fitting can pivot around the pivot pile axis for gate closing and opening situations and the sliding up and down of the gate for vertical movement is also possible. The pivot pile fitting pin and jaw assembly and free-to-rotate annular attachment will allow the barge gate to rotate about both longitudinal and transverse axes. Therefore the wave actions and other imbalances are resisted by the buoyancy force equilibrium of the gate instead of by the swing pivot arm. The barge connection is a tripod assembly constructed of steel pipes (Figure 8-14 and Figure 8-15). The barge gate swings by using hydraulic winches for opening or closure.

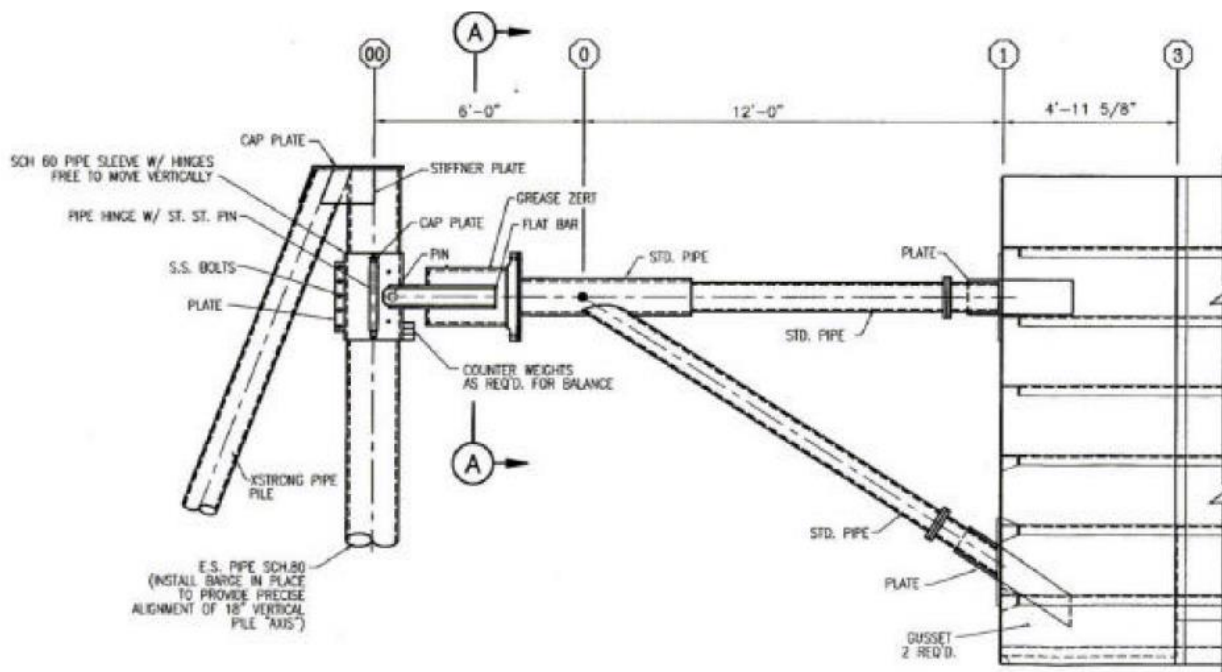


Figure 8-14: Bayou Lafourche Swing Gate's swing pivot arm section view [43]

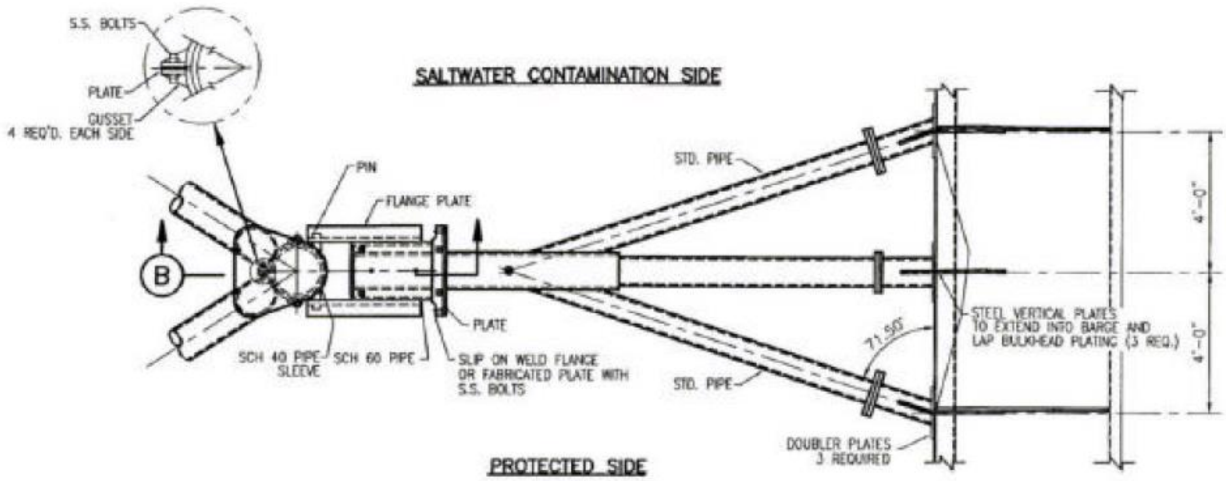


Figure 8-15: Bayou Lafourche Swing Gate's swing pivot arm plan view [43]

8.4.2.2 Bayou Dularge Barge Gate

This swing barge gate with the length of 19.5 meter (for more information see section 4.2.10) has the same articulation system as the Bayou Lafourche Swing Gate.

8.4.2.3 Monaco Floating Pier (Breakwater)

A key element of the extension of Condamine port at Monaco was a 352 m long and 163,000 tons semi-floating pier [71]. This highly prestressed reinforcement concrete structure is attached to the main land abutment with a very complex and 770 tons steel ball-joint system while the other end of the pier is secured with two sets of fixed anchor chains to seabed.

The articulation system is designed to allow rotation and to resist up to 10,000 tons of horizontal and vertical loads. This articulation system gives three degrees of freedoms; pitch, roll and yaw to the structure. The steel-ball has the diameter of 8 m at its largest part and is designed to act like a fuse in case of seism by separating the structure from the abutment. The steel ball is attached to the floating structure and the socket is attached to the abutment [71]. Both parts are attached by using the stressing bars. A complete monitoring system has been installed on the anchorage of the bars to follow their tensions.

Actually the tension in the stressing bars is a great indicator of the efficiency of the steel ball. Any increase of the tension shows that the steel ball is transferring a part of the movements onto the structure instead of rolling. The ultrasonic system monitor the bars carefully.

The fatigue and the mechanical wear of the steel bar and its socket is important because the structure is designed for a 100 years' service life. The mechanical wear is measured by very sensitive displacement sensors pointing at the surface of the steel ball [71]. The longitudinal and radial wears can be measured in this way.

8.4.2.4 Maeslant Barrier

The articulation system of the Maeslant barrier is a ball and socket joints (for more information about the barrier see section 4.2.8). In this storm surge barrier, the arms are connected to the ball-joint systems. The ball-joints system provides the gate with freedom of movement in all directions, both horizontally when sailing out and vertically when sinking down [72]. The sway of the gate is possible under the waves effects. Also, the water pressure is transferred to the foundations through this system. A ball-joint system which works like human hip or shoulder joint can assure all of the movements and functions. The joints have a diameter of 10 meters and weight of 680 tons.

Figure 8-16: Maeslant barrier ball-joints articulation system [72]

8.4.3 The Proposed Articulation System

The proposed system can be a tailor-made articulation system similar to the connection system of the Maeslant barrier or the Monaco breakwater for the main guiding system of the gate. The system can consist of a ball-joint systems located in a socket. Also, the articulation system of the swing arms like in case of the two above mentioned barge gates is the other candidate option which can be used with the suitable bigger scale for the current project. Selection of the final articulation system choice needs more research and information which is out of the scope of the current report.

The barge gate should be able to rotate freely about both longitudinal and transverse axes. In this way no force is transferred to the joint and the buoyancy equilibrium will work favorably. A monitoring system such as in case of the Monaco breakwater is needed to control the real performance of the articulation system.

Next to the ball-joint system or the swing arms as the main articulation system, the use of the guiding columns and cables (winches) in the rest chamber of the gate will provide the gate with more safety during the opening and closing procedure.

Design of the ball-joint articulation system or the swing arms can be done later when the information from the hydrodynamic analysis of the gate is available. It is expected that the system wouldn't have the high weight like in case of the Maeslant barrier. In the Maeslant barrier the ball-joint system has to transfer the water pressure on the gates to the foundations and this is the main cause for the enormous 10 m diameter of the ball-joint [81].

8.4.4 Articulation System Conclusions

In this section an overview is given for the needed articulation system of the barge gate. Project examples have been evaluated and on the basis of the project requirements and the available systems, ball-joints system or the swing arms have been proposed for the main articulation system of the gate. The guiding columns and cables which are located in the resting chamber can help the articulation system during the opening and closure phases.

In this section only a conceptual overview and design is given for the articulation system of the barge gate. Further design analysis should be done for this system considering the forces and structural design requirements which is out of the scope of this report. See section 10.6.1 for the initial estimation of the forces acting on the articulation system.

8.5 Chapter Conclusions

In this chapter designs of some complementary structures are treated. Due to time constraints, only bed protection system, berthing system and articulation system are described.

On the basis of the designs results up to now, most of the requirements of the barrier have been checked. For checking the reliability and economical requirements, evaluation and reflection should be realized in the next design step.

Design of the other complementary structures such as ballasting system, hydraulic braking system and etc. are recommended to be performed in the future research.

9 Design Evaluation and Reflection (DS5.)

9.1 Introduction

In the previous sections the integral design has been done up to design step 4. There are interconnections between different steps of the design and an iteration is needed to check different aspects of the design. In this part of the report, it is useful to look at the design up to now and evaluate the design regarding different design parameters and assumptions. In this way the effects of the design parameters and assumptions can be assessed before continuing to another step. Also, risk analysis will help to find out about the possible drawbacks of the design and will make the needed following tasks clear. This chapter treated the design step 5 and its sub-steps (Figure 9-1).

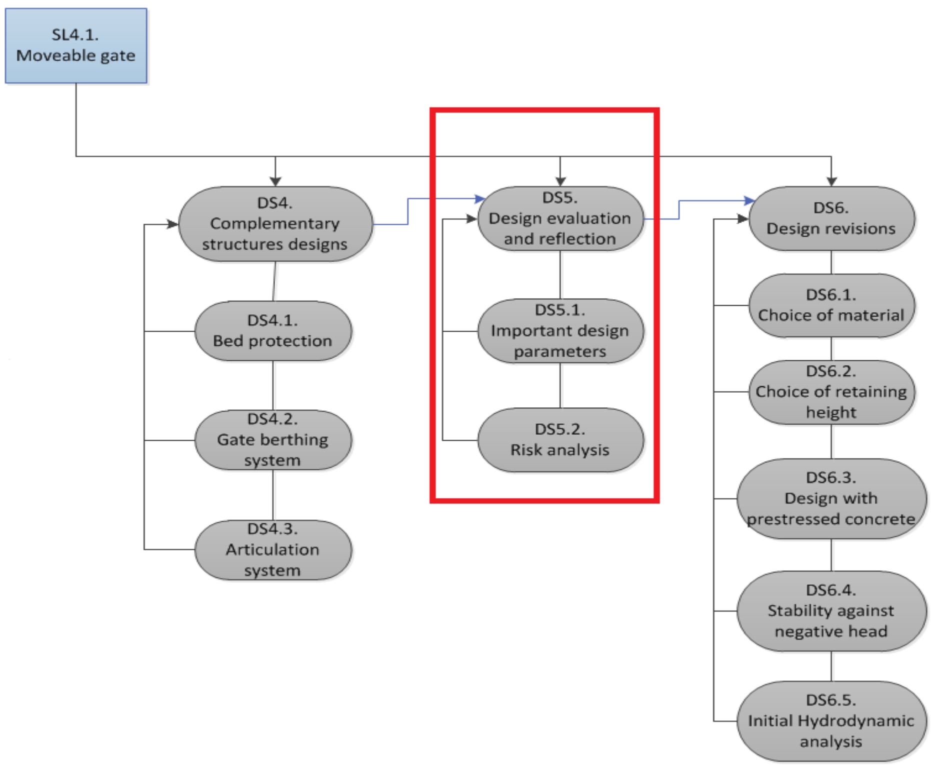


Figure 9-1: Design step 5; Design evaluation and reflection

In the previous chapters main requirements of the final design have been introduced. In this chapter evaluation will consider the most important requirements. Up to this design step the design requirements have been checked through the proposed solutions except the reliability and economical aspects. In this part the evaluation can consider these requirements too.

9.2 Important Design parameters (DS5.1.)

On the basis of the design outcomes, the important design parameters can be recognized. In this section these parameters and their influences on the final design aspects are going to be depicted in more details. It should be mentioned that the evaluation of the design parameters is limited to the navigational barrier which is the focus of the current report.

9.2.1 Construction Material Evaluation

The choice of material is a key design parameter. Whether steel, concrete or FRP are chosen for the gate, the final design would be different. The main influence of the material would be on the construction costs. Using the concrete might lead to a gate with more weight and different costs compared to steel or FRP. Also, the design difficulty and complexity would be different in case of each material.

It can be expected that all of the materials give a certain level of reliability for the final design because there is enough knowledge and expertise for design. In addition, maybe concrete is more environmentally friendly compared to the steel because there would be limited need of the corrosion prevention methods which are essential in case of steel.

Also, using the prestressed techniques may result in different final design results compared to the normal reinforcement concrete. It can be expected that the width of the elements of the caisson gate would be less in this case which leads to less weight of the structure. The cost of the project would be then different (and probably higher in case of the prestressed concrete).

9.2.2 Layout and Location Evaluation

The layout and location of the structure affects the final costs of the project. Currently the minimum length of the opening in Bolivar Roads Pass has been considered for the storm surge barrier. Navigational barrier as part of the storm surge barrier should be consistent with the requirements for minimum and enough width and consequently less costs. It is logical that this selection would be the best one for the final design of the storm surge barrier.

9.2.3 Retaining Height Evaluation

The retaining height of the structure is an important design parameter which affects all aspects of the final design. The higher retaining height will increase the safety level of the barrier while possibly increase the costs of the project. The effects of the retaining height on the environment should be considered for the interaction between the environmental barrier and the navigational barrier. The works of Ruijs (2011) [6] and de Vries (2014) [11] can be referred in this regard. It is obvious that the entire design would change on the basis of a different retaining height.

9.2.4 Gate System Evaluation

The gate system is an important factor which affects different aspects of the final design. An important issue here is the opening under the gate. In the current design it has been considered that there is a 1 meter opening under the gate. This will lead to high flow speeds under the gate and consequently the need for strong bed protection which increase the cost of the project.

Also, reliability of the gate in terms of the water retention would be higher if there would be no opening under the gate because less flow would enter to the Galveston Bay. However, according to the previous researches the leakage under the gate is acceptable because there is a large buffer capacity behind the barrier.

On the other hand, if the opening under the gate would be closed, there is a need for a sill under the gate which requires high soil strengths. In case of the Bolivar Roads Pass the quality of soil under the structure is not suitable. In this case, the soil improvements are needed which will lead to higher costs. Then the foundation design and articulation system would be also affected. Also during the closure, there is a need for appropriate connection between the gate and the sill under it. Thus the tolerances of the sill level and the settlements should be limited. The possible sediments on the sill should be washed away before the closure of the gate which costs more time and effort for closure.

9.2.5 Influence of the Design Parameters on the Design Aspects

The important design parameters and their influences on the final design aspects have been described in the previous sections. Figure 9-2 shows these issues in more details.

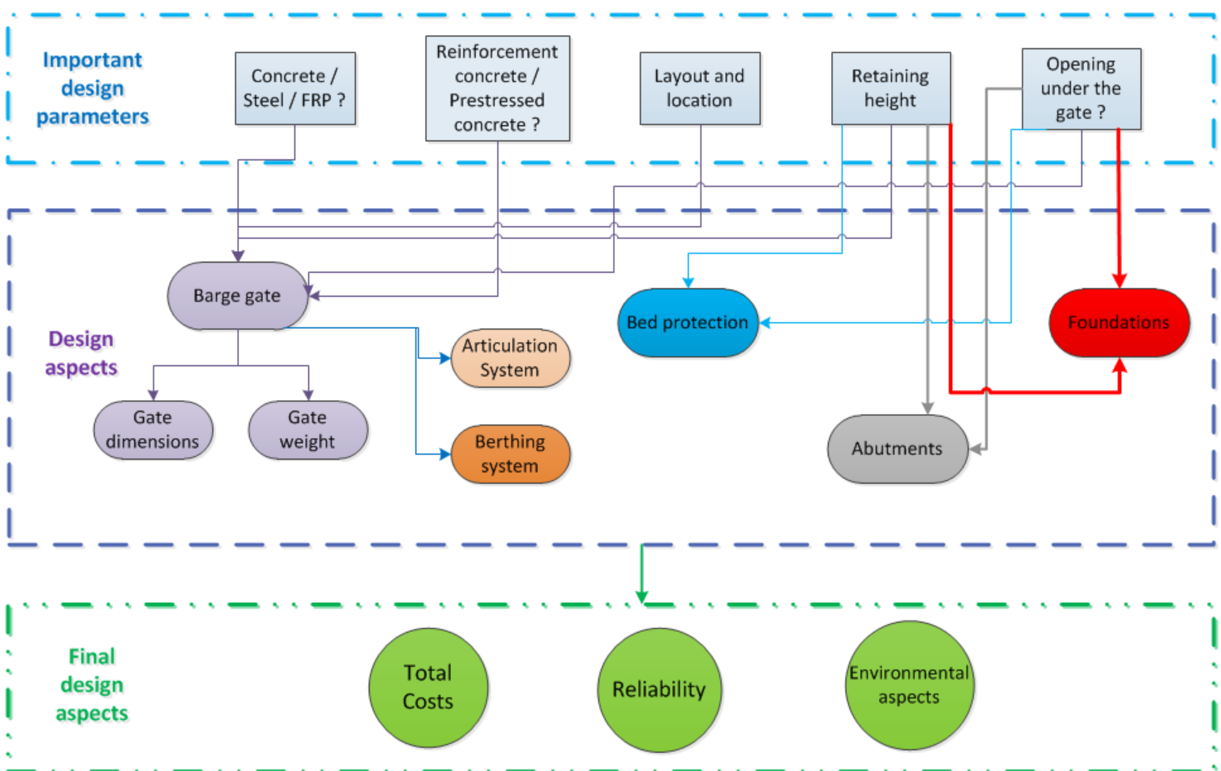


Figure 9-2: Important design parameters and influences on final design aspects

In this figure only the current design parameters have been shown. One can think about other design parameters in the total picture of the project design such as human errors, computer errors, maintenance and operational aspects, inspection and monitoring aspects, quality control and etc. These issues are not treated in the current report.

By using the information depicted in this section, in the next chapter some design revisions as part of the design iterations are going to be performed.

9.3 Risk Analysis (DS5.2.)

Risk analysis of the project can be done in different design steps for checking the requirements of the project in different system levels. Risks can be identified in different phases of the life cycle of the project. In this section, firstly the descriptions of risks in different project phases are going to be described and then risk register matrices are going to be made for the different situations and elements of the navigational barrier.

The method of risk assessment in this chapter is the qualitative risk assessment. The operational phases of the barge gate are very important and that is why the fault tree analysis is going to be performed for it. The focus of this chapter is on technical risks. The general risk analysis regarding the other aspects of this project (such as management risk, financial risk, external risks and etc.) are not considered in this report. Also, the risk analysis of the maintenance and demolition phases are not treated here.

9.3.1 Design Phase Risk Analysis

In the design phase of the barrier different aspects may lead to risks for the project. These aspects are described below.

The required safety level of the barrier has been determined by using cost benefit analysis and probabilistic approach by Stoeten (2012). Also, the prediction of the design storm and design surge have been done by Stoeten (2012) and de Vries (2014). Both of these researchers have used simplified numerical methods. More Extensive research is needed to find out about the design storm and appropriate safety level to reduce the possible risks in this regard.

Also, retaining height of the structure is the other aspect of the requirements. De Vries (2014) has found the MSL+0.1 m enough for the retaining height. On the other hand, it is proposed that the barrier can be designed for retaining at least the forerunner surge of hurricane. Modeling the Gulf, barrier and Bay system with more advanced numerical modeling systems will reduce the risks of using wrong retaining height in the design of the barrier and may lead to optimization of the design and consequently cost of the project.

The type of the navigational barrier has been chosen as a barge gate in the current study through MCA. However, in reality and after some design steps it might be appeared that the other alternatives may lead to more suitable end product. Indeed, the choice of the barrier itself is a risk. The design should go further to check if the barge gate still satisfies the project requirements. Also, interaction of the environmental barrier and navigational barrier should be studied carefully.

On the other hand, design complexity may cause some failure in the design procedure. Human and software errors should be added to the risks of the design phase.

To name a few, the risks regarding the design phase are as follow:

- Inappropriate choice of design storm and safety level
- Inappropriate choice of retaining height
- Inappropriate choice of navigational barrier type
- Unexpected happenings due to separate design for navigational and environmental barrier
- Human and computer errors

9.3.2 Construction Phase Risk Analysis

The interaction between the design phase and construction is inevitable. The construction considerations will be elaborated later in this report (See Section 12.1). Regardless of the construction method (in-situ or prefabricated) and construction material, construction of the large scale projects includes different risks.

Considering the current project, the concrete material is proposed for the structure. Mix design of the concrete, specially the HPLC, needs special attention and expertise. Quality control of the concrete is essential.

Also, in case of using the prestressed concrete, the tendons should be tensioned when the concrete has the certain level of comprehensive stress. The requirements in this regard can be later on determined on the basis of the type of the prestressed method which is going to be used. Also, the concrete should have certain amount of hardening before the formworks can be released.

If the dry dock is used for construction, the subsoil should be investigated to have enough strength for construction of a heavy structure like the barge gate. Otherwise using the deep foundations (piles) should be considered. The location of such a dry dock also should be determined on the basis of site data.

In short, the risks in the construction phase can be summarized as follow:

- Concrete mix design complexity
- Execution difficulties because of the large scale of the gate
- Concrete quality problems

- Accidents in the construction site
- Subsoil in the dry dock doesn't have enough strength
- Possible difficulties in procurement of material

9.3.3 Transportation and Placement Phase Risk Analysis

After the gate is constructed in the dry dock (See section 12.1.3) it should be floated and transported to the location. When the construction is finished, the dry dock should be filled with water and the gate should be floated.

The transportation should be realized in the normal weather conditions. The design of the gate on the basis of the transportation and floating requirements have been considered in the previous design steps. The sway consideration for a barge gate was almost near the problematic zone. However, under control of the tugboats the barge gate can be transported safely to the location.

Also, dynamic stability and static stability of the floating barge gate are important. In the previous steps of the design these requirements have been fulfilled. Structural aspects of the floating barge gate should also be considered specially when the caisson is empty. This issue is treated previously.

When arrived on site, the barge gate can be placed and connected to the articulation system with the help of the tugboats and guiding vessels.

The possible risks regarding this phase are as below:

- Stability problems due to unexpected circumstances
- Structural failure due to unexpected loads
- Heavy forces should be resisted by tugboats due to the high weight of the structure

9.3.4 Operational Phases Risk Analysis

The operational phases of the barge gate have been described in section 6.5 completely. In this section possible risks and important issues regarding these operational phases are depicted. In this analysis also the possible effects of the abutments and foundations have been considered.

9.3.4.1 Gate in Open Position Risk Analysis

During the normal condition the gate is stored in the resting chamber as mentioned in section 6.5.3. There are important issues regarding this phase. In general two different situations for the gate in the open position can be assumed.

Floating gate in open position:

The design of the gate in previous steps has been done while considering the gate in floating situation during the opening condition.

The draught of the gate in the open situation should be evaluated. For the current design, the initial draught of the structure is equal to 7.09 m. If the gate would be without any ballasting in the open situation, it means that 22.5 – 7.09 = 15.41 m of the gate is above the MSL.

The gate can be fixed in the floating situation by using the mooring system. In general the forces in mooring ropes are caused by [58]:

- Loads on the ships because of wind, current and water level differences
- Movements of the gate. In this case the mass of the gate and the stiffness of the mooring cables are of importance in this respect.

The mooring forces for the preliminary design of the mooring system can be derived from Table 9-1. Considering the current dimensions of the gate, the mooring force would be around 850 kN). The mooring system can be initially designed for this force. However, the modeling of the gate motions with computer programs in 3D is recommended.

Choice of the gate draught is also very important. The draught of the gate should be calculated for optimized the gate motions and mooring forces to the mooring system and articulation system.

SEAGOING SHIPS				BARGES	
Water displacement of ship [ton]	Mooring force			Class of ship	Mooring force [kN]
	[kN]	[%]			
< 2 000	100	5.0 %		Pleasure boats	55
2 000 ~ 10 000	300	3.0 %			
10 000 ~ 20 000	600	3.0 %		Professional shipping:	
20 000 ~ 50 000	800	1.6 %		Class I + II	140
50 000 ~ 100 000	1000	1.0 %		Class III + IV	210
100 000 ~ 200 000	1500	0.7 %		Class V + VI	280
> 200 000	2000	< 1.0 %			

Table 9-1: Mooring forces per bolder [58]

The important aspect of the barge gate in the floating condition is the hydrodynamic behavior of the structure. In the floating condition dynamic forces and fluctuation forces cause a dynamic response in the structure which leads to:

- A load on the floating structure

- An excitation of the structure into oscillatory response motions

In general dynamic forces are divided to the following loads [77]:

- First order wave forces
- Second order low frequency wave drift forces
- Wave forces from swell
- Wave forces from waves generated by passing ships
- Turbulence

The structure should be analyzed for the dynamic forces and the related motions. The main articulation system and additional guiding columns and cables should be able to provide enough resistance regarding the mooring of the gate. The gate motions should be considered under different loading situations. The certain motions of the floating body can cause huge forces on the articulations system which should be studied carefully. Ship collision is the other aspect which may lead to the failure of the barrier.

According to the information above, the main risks of the floating gate in open position are as follow:

- Unexpected and unfavorable motions of the gate due to dynamic forces
- High forces can be transferred to the articulation system (swing point)
- The need for controlled ballasting (if needed) due to different tidal situations, wave forces and wind forces to keep the gate in the desired position
- Ship collision

Ballasted gate in open position:

The gate can be ballasted and rests on the foundations on the ground in the open situation. In this situation the problems such as unexpected motions of the floating gate are prevented. The foundation can be a sill under the entire section or just in certain parts under the gate as shallow foundations.

The disadvantage of this option is for example transferring the high forces to the weak soil under the barrier. Also, before the closure phase the gate should be first un-ballasted and then floated to the closing situation. This takes more time than the first option. Indeed, the risks of this option are as follow:

- Higher costs for the foundation design under the gate in the open situation
- Closing of the gate takes more time

9.3.4.2 Gate During the Closure Process Risk Analysis

During the closure gate should be floated and turns around its pivot point (swing point) to close the opening. The gate can be floated keeping the initial draught of the structure. The propellers provide the energy for moving the gate while the articulation systems and guiding columns and cables take care of accuracy of the movement. The gate finally rests on the abutments horizontally while the berthing systems absorb the berthing energy of the gate.

The berthing energy of the gate is different on the basis of the moving velocity and draught of the structure. Increasing any of these two will result in more berthing energy and consequently more forces will transfer to the abutments. Indeed, the velocity of the moving and the desired draught play important roles.

The motions of the gate during closure process is the other important issue. The hydrodynamic analysis should be done to determine the motions of the structure and consequently the loads on the articulation system. Stability of the gate in this situation is vital. Using the system of floats with the double catamaran effect will facilitate the floating of the gate by giving more stability.

The main risks during the closure process are:

- Hydrodynamic forces may cause unexpected and undesired motions
- Unstable floating body
- High forces may act on the articulation system
- Possible errors and problems in the operation of the propellers
- Failure of articulation system
- Failure of berthing system

9.3.4.3 Gate During the Immersion at Final Location Risk Analysis

When the gate rests on the abutments, then the immersion of the gate should be done. The ballasting system provides the immersion of the gate. The gate should be stable during different phases of the immersions. This issue has been checked for the structure in the design phase.

During the immersion as the gate gets more draught, more forces will transfer to the abutments because of the hydrostatic forces. In this case the tyres may pressed totally and may stop functioning. The Teflon skids should provide the vertical movement of the gate without any problem.

Also, the articulation system should provide the vertical movement possibility for immersion. The motions of the gate during the immersion phase because of the dynamic forces are important. Shaking of the gate due to dynamic forces for example because of the underflow can also be expected.

The main risks during immersion are:

- Tyres may pressed and do not function anymore
- Controlled immersion is difficult
- Unexpected motions of the gate because of dynamic forces
- Shaking of the gate because of dynamic forces

9.3.4.4 Gate Immersed at Final Location During Normal Conditions Risk Analysis

When the gate immersed at its final position, it rests vertically on the foundations and horizontally on the abutments. The water level in this situation is almost the same in the Gulf side and Bay side. Tidal differences in the Gulf are almost 0.35 m as it is mentioned before (See Section 3.2.3). It seems that this tidal difference wouldn't cause any problem for the stability of the gate in this situation. However, a system for water level measurement is necessary.

It would be a situation when the water level in the Bay side is more than in the Gulf side. Then the gate will open under the negative current head because it is not supported horizontally. To overcome this problem a lock or a civil structure (as the horizontal support for the negative head) should be used for keeping the gate in position.

The gate in this phase can be ballasted more than required to reach the MSL-17 m. This means that by using more water ballast, more forces will transfer to the foundations. The gate has been designed for these forces in the previous steps.

The main risks in this situation are:

- Sea level fluctuations may lead to difficulties in keeping the gate in position
- If the sea level in Bay side is more than Gulf side, the gate is not supported for the forces in reverse direction and opens unexpectedly

9.3.4.5 Gate Immersed at Final Location During Hurricane Conditions Risk Analysis

During the hurricane, the immersed gate will resist the hydrostatic and wave forces from the Gulf side. Because of the sea level differences between the Gulf side and the Bay side, the underflow from the 1 m gap under the barrier is happening. The flow speed is high and a bed protection system is required.

Also, this underflow makes the hydrodynamic forces and suction force which cause shaking in the structure and also maybe unexpected motions. The foundations and abutments should be designed to resist the forces transferred from the gate in this situation.

Also, the forces on the gate in the longitudinal direction is not uniformly distributed. It means that different forces act on the different cross sections of the gate and the detailed analysis in this regard is needed.

The main risks in this situation are:

- Hydrodynamic forces to the gate
- Underflow with high speed
- Shaking of the gate because of dynamic forces and suction force
- High forces may act on the articulation system (swing point) because of the shaking
- Fluctuations of the sea level which makes keeping the gate in its position hard
- Un-uniformed distributed loads in the longitudinal direction acting on the gate

9.3.4.6 After the Hurricane Situation Risk Analysis

After the hurricane the sea level in Bay side might be higher than in the gulf side. In this situation the gate should un-ballasted and floated back to the resting chamber under the current forces. There is a need for very sophisticated ballasting system. The best situation would be the case that before a huge negative head the gate be un-ballasted and floated back by means of the normal current forces while the openings in the environmental sections and probably in the gate itself provide the exchange of water between the Gulf side and Bay side (and reduce the negative head on the barge gate).

The opening of the gate under the negative flow is very important. At first it would be a situation when the sea level in the Bay side is only small amount more than in the Gulf side. In this situation the ballasted gate can resist the loads (if not un-ballasted). However, when this negative head is a considerable amount, the gate cannot resist the forces and the deflection of the gate itself and the possible damages to the foundation is probable. A lock should be designed to bear these forces and keep the opening procedure of the gate under the control. Also, the openings in the environmental barrier should be opened before happening of a huge negative head.

Figure 9-3 shows the gate as a beam when it rests on the foundations, under the negative head which is presented here as the uniform distributed load. When the negative head has the small amount the gate behaves like a structure supported on the foundations and the deformation of the gate in this situation is shown by the dashed blue line in the figure. However, when the negative head has a huge amount, the gate will slide on the foundations and opens without control which may cause damages to the gate and foundations.

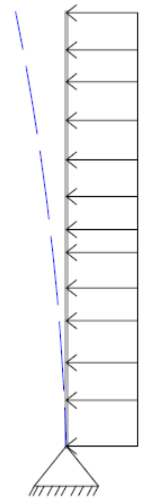


Figure 9-3: Gate deformation under the negative head

Also, the motions of the gate under dynamic forces should be investigated in this case and the current velocity which forces the gate to float back should be defined. The unfavorable motions in this situation are very probable which may cause damages to the articulation system and to the gate itself.

Another solution for the problem of negative head (next to a lock or a civil structure) is the water level measurement system. This system should be installed in both sides of the barrier, in the Gulf side and in the Bay side. The information regarding the water levels should be sent to the operational sections of the barrier and the gate should be open prior to extreme negative head. This requires a gate with the potential for fast opening. Due to the sea level fluctuations this is a hard procedure and needs special attention.

The hydraulic braking system in the resting chamber should provide the safe mooring of the gate in the resting chamber. However, because of the big dimensions and weight of the gate and also the velocity of the gate during opening, high forces act on the hydraulic braking system. Thus the hydraulic braking system should be designed for this critical situation.

The main risks for the gate during the opening after hurricane are as follow:

- Failure of water level measurement system
- Uncontrolled opening of the gate due to the negative head
- Hydrodynamic forces to the gate
- Unexpected and undesired motions of the gate
- Possible damages to the gate, articulation system, resting chamber or even foundations

9.3.5 Risk Register

According to the information provided in the previous sections of this chapter, the risk register matrices can be constructed for the navigational barrier considering the technical risks. Risk register as part of the qualitative risk assessment method helps the project manager to mitigate the risks in different phases of the life cycle of the project.

Risk register contains the risk category (here technical), risk events, causes of the risk events, consequences, affected promises, likelihood of the event, effect of the event and risk score. For the current project the risk register matrices have been made only in the general level. For sure these matrices can be further developed in more details later.

The promises of the project have been considered as follow:

- Project costs (shown with C in matrix)
- Project time (shown with T in matrix)
- Reliability (shown with R in matrix)

The likelihood of a risk happening and its effects on promises have been considered by three levels:

- Low (L) with the score of 1
- Medium (M) with the score of 2
- High (H) with the score of 3

The final score of the risks can be calculated obviously by multiplying the likelihood and effect scores. The risk register matrices can be seen in Table 9-2 to Table 9-5. The risk register for the operational phases has been shown considering different situations of the gate in operation. The column of adverse consequences on promises means more costs, time delay and reduction of reliability due to the risks.

The risk events with more score need more attention in the later steps of the integral design of the project. The importance of the risk events is shown with different color; red color with the highest importance, then the orange color and then the yellow color with less importance.

The idea behind the risk register matrix is subjective. However, it can give good indication regarding the most important aspects of the design that should be considered and elaborated in more details in the later steps.

Risk Register Matrix 1: Design, Construction, Transportation and Placement Phases							
Category	**Project Phase**	**Risk event**	**Causes**	**Adverse Consequences on Promises**	**Likelihood**	**Effect**	**Risk Score**
	Design		Wrong assumption for the retaining height	C / T / R	L	H	3
			Not appropriate barrier type	C / T / R	L	H	3
			Interaction problems of the navigational and the environmental barriers	C / T / R	M	H	6
			Human errors	C / T / R	M	M	4
			Computer/software errors	C / T / R	M	M	4
	Construction	Concrete mix design complexity	Lack of comprehensive instructions regarding mix design	C / T / R	L	M	2
		Execution difficulties	Lack of site management / instruction regarding execution	C / T	L	M	2
		Not appropriate quality	Lack of quality control	C / T / R	L	H	3
		Site accidents	Lack of safety instructions and control	C / T	L	M	2
		Dry dock subsoil problems	Not enough/wrong analysis of subsoil	C / T	L	M	2
		Material procurement problems	Not enough/wrong analysis in choice of material	C / T	L	M	2
	Transportation and placement	Stability failure	Wrong/not enough stability analysis	C / T	L	H	3
			Unexpected environmental conditions	C / T	L	H	3
			Tugboats failure	C / T	L	L	1
		Structural failure	wrong/not enough structural analysis	C / T	L	H	3
			Unexpected environmental conditions/unexpected loads	C / T	L	H	3
		Tugboats failure	Wrong/not enough transportation forces calculations	C / T	L	L	1

Table 9-2: Risk register matrix 1

Risk Register Matrix 2: Operational Phases (1)							
Category	**Project Phase**	**Risk event**	**Causes**	**Adverse Consequences on Promises**	**Likelihood**	**Effect**	**Risk Score**
Technical	**Gate in Open position**	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	L	H	3
			Ship collision	C / T / R	L	H	3
			Unexpected environmental conditions/unexpected loads	C / T / R	L	H	3
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	L	H	3
			Ship collision	C / T / R	L	H	3
			Unexpected loads	C / T / R	L	H	3
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	M	M	4
			Unexpected loads transfered from the gate to the system	C / T / R	M	M	4
		Mooring system failure	Wrong/not enough structural analysis	C / T / R	L	M	2
			Unexpected loads transfered from the gate to the system	C / T / R	L	M	2
	Gate in Closure process	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	M	H	6
			Unexpected environmental conditions/unexpected loads due to sea level fluctuations	C / T / R	L	H	3
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Unexpected loads due to sea level fluctuations / dynamic loads	C / T / R	M	H	6
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	H	H	9
			Unexpected loads transfered from the gate to the system	C / T / R	H	H	9
		Ballasting system failure	Wrong/not enough system design	C / T / R	L	H	3
			Unexpected failure	C / T / R	L	H	3
		Berthing system failure	Wrong/not enough system design	C / T / R	L	H	3
			Unexpected loads/failure	C / T / R	M	H	6
		Abutments failure	Wrong/not enough abutment structural design	C / T / R	L	H	3
			Unexpected loads transfered from the gate to the abutment	C / T / R	L	H	3
		Guiding system failure	Propellers failure	C / T / R	L	H	3
			Guiding columns and cables failure	C / T / R	L	H	3

Table 9-3: Risk register matrix 2

Risk Register Matrix 3: Operational Phases (2)							
Category	**Project Phase**	**Risk event**	**Causes**	**Adverse Consequences on Promises**	**Likelihood**	**Effect**	**Risk Score**
Technical	**Gate During the Immersion at Final Location**	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	L	H	3
			Unexpected environmental conditions/unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	L	H	3
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	M	H	6
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	H	H	9
			Unexpected loads transfered from the gate to the system	C / T / R	H	H	9
		Ballasting system failure	Wrong/not enough system design	C / T / R	L	H	3
			Unexpected failure	C / T / R	L	H	3
		Abutments failure	Wrong/not enough abutment structural design	C / T / R	L	H	3
			Unexpected loads transfered from the gate to the abutment	C / T / R	L	H	3
		Berthing system failure	Wrong system design	C / T / R	L	M	2
			Unexpected loads/failure	C / T / R	L	M	2
	Gate Immersed at Final Location During Normal Conditions	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	L	H	3
			Unexpected environmental conditions/unexpected loads	C / T / R	M	H	6
			Unexpected opening of the gate due to sea level fluctuations	C / T / R	M	H	6
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	M	H	6
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	H	M	6
			Unexpected loads transfered from the gate to the system	C / T / R	H	M	6
		Foundations failure	Wrong/not enough structural analysis	C / T / R	L	H	3
			Unexpected loads	C / T / R	L	H	3
		Water level measurement system	Wrong system design	C / T / R	L	M	2
			Unexpected failure	C / T / R	L	M	2

Table 9-4: Risk register matrix 3

Risk Register Matrix 4: Operational Phases (3)							
Category	**Project Phase**	**Risk event**	**Causes**	**Adverse Consequences on Promises**	**Likelihood**	**Effect**	**Risk Score**
Technical	Gate Immersed at Final Location During Hurricane Conditions	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	M	H	6
			Unexpected environmental conditions/unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	L	H	3
			Unexpected opening of the gate due to sea level fluctuations	C / T / R	M	H	6
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	H	H	9
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Unexpected loads transfered from the gate to the system	C / T / R	M	H	6
		Foundations failure	Wrong/not enough structural analysis	C / T / R	L	H	3
			Unexpected loads	C / T / R	L	H	3
		Bed protection failure	Wrong/not enough structural analysis	C / T / R	L	H	3
			Unexpected loads	C / T / R	L	H	3
		Abutments failure	Wrong/not enough abutment structural design	C / T / R	L	H	3
			Unexpected loads transfered from the gate to the abutment	C / T / R	L	H	3
	After the Hurricane	Gate Stability failure	Wrong/not enough stability analysis	C / T / R	M	H	6
			Uncontrolled opening due to negative head	C / T / R	H	H	9
		Gate Structural failure	Wrong/not enough structural analysis	C / T / R	M	H	6
			Uncontrolled opening due to negative head/unexpected loads due to sea level fluctuations or dynamic loads	C / T / R	H	H	9
		Articulation system failure	Wrong/not enough structural analysis	C / T / R	H	M	6
			Unexpected loads transfered from the gate to the system	C / T / R	H	M	6
		Foundations failure	Wrong/not enough structural analysis	C / T / R	L	H	3
			Unexpected loads	C / T / R	M	H	6
		Ballasting system failure	Wrong/not enough system design	C / T / R	L	M	2
			Unexpected failure	C / T / R	L	M	2
		Water level measurement system	Wrong system design	C / T / R	L	M	2
			Unexpected failure	C / T / R	L	M	2
		Hydraulic breaking system failure	Wrong/not enough structural analysis	C / T / R	M	M	4
			Unexpected loads	C / T / R	M	M	4

Table 9-5: Risk register matrix 4

9.3.6 Barge Gate Operational Fault Trees

Because of the importance of the operational phases of the barge gate and its reliability during the operation, the fault tree analysis has been done for different phases of the operation. In general, fault trees are modeling tools as part of quantitative analysis of a system. However, because of the lack of information in this stage of the design, fault trees are made without adding the quantitative data. Obviously these faults trees can be adjusted later quantitatively.

Theoretically in fault tree analysis one attempts to develop a deterministic description of the occurrence of an event (the top event) in terms of occurrence or non-occurrence of other intermediate events [78]. Fault trees can be used together with reliability data for the basis events to make estimates of system reliability.

For the barge gate, the top event is considered as failure in function of the navigational barrier. The fault trees can be made for different operational phases. The symbols which are used in the fault trees are shown in Figure 9-4.

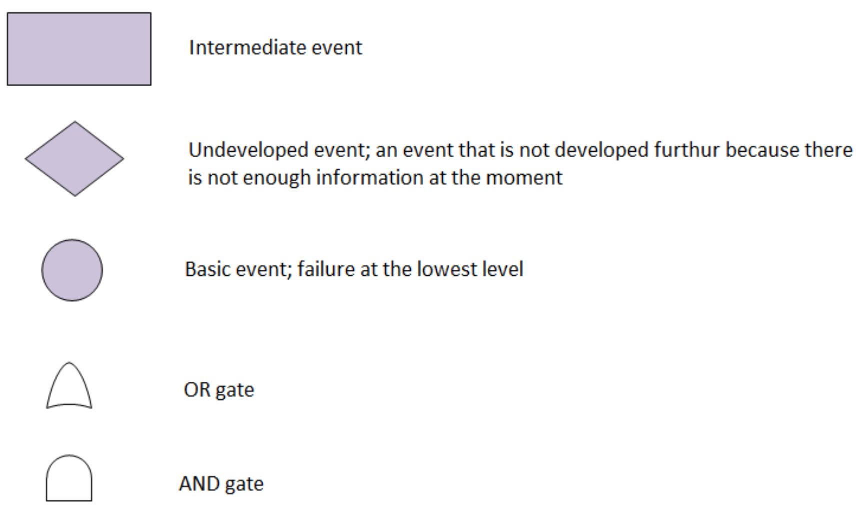


Figure 9-4: Symbols used in the fault trees

Due to the space limit, in Figure 9-5 only the top system of the fault tree has been shown. The fault trees of each operational phase can be seen in Figure 9-6 to Figure 9-11. Different colors have been chosen to make clarifying different phases easier. In the figures the following abbreviations have been used:

- FHS: failure due to hydrostatic loads
- FHD: failure due to hydrodynamic loads
- FSC: failure due to ship collision

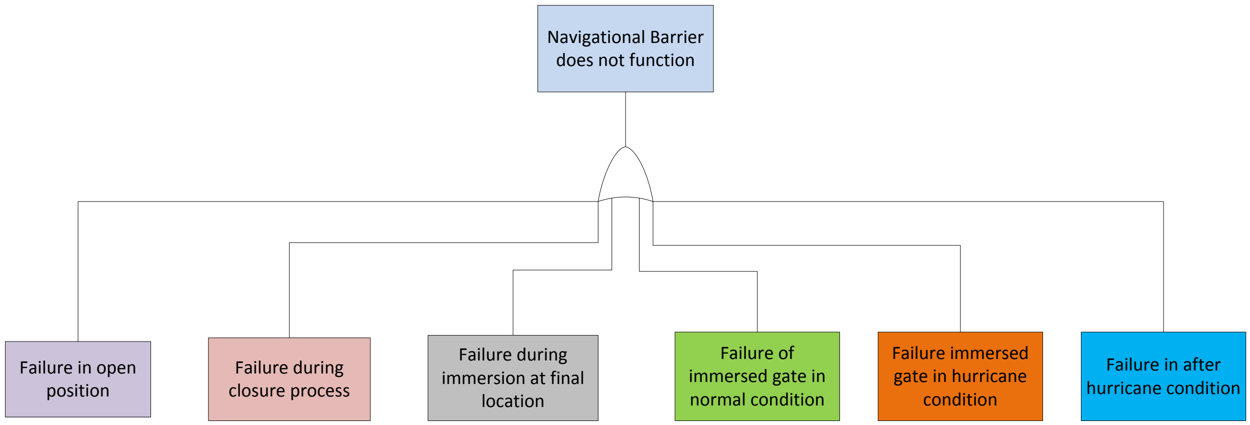


Figure 9-5: Main system of navigational barrier operational fault tree

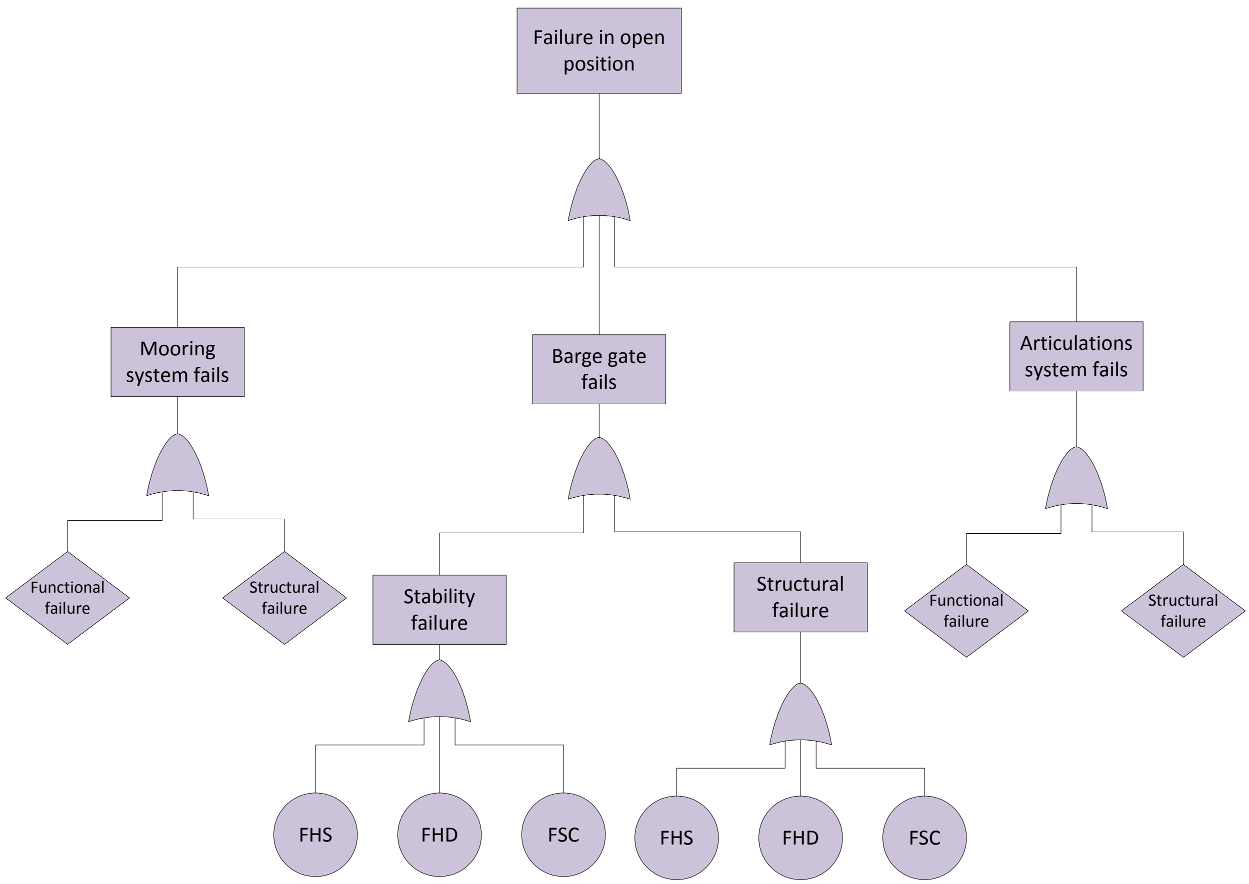


Figure 9-6: Failure in open position fault tree

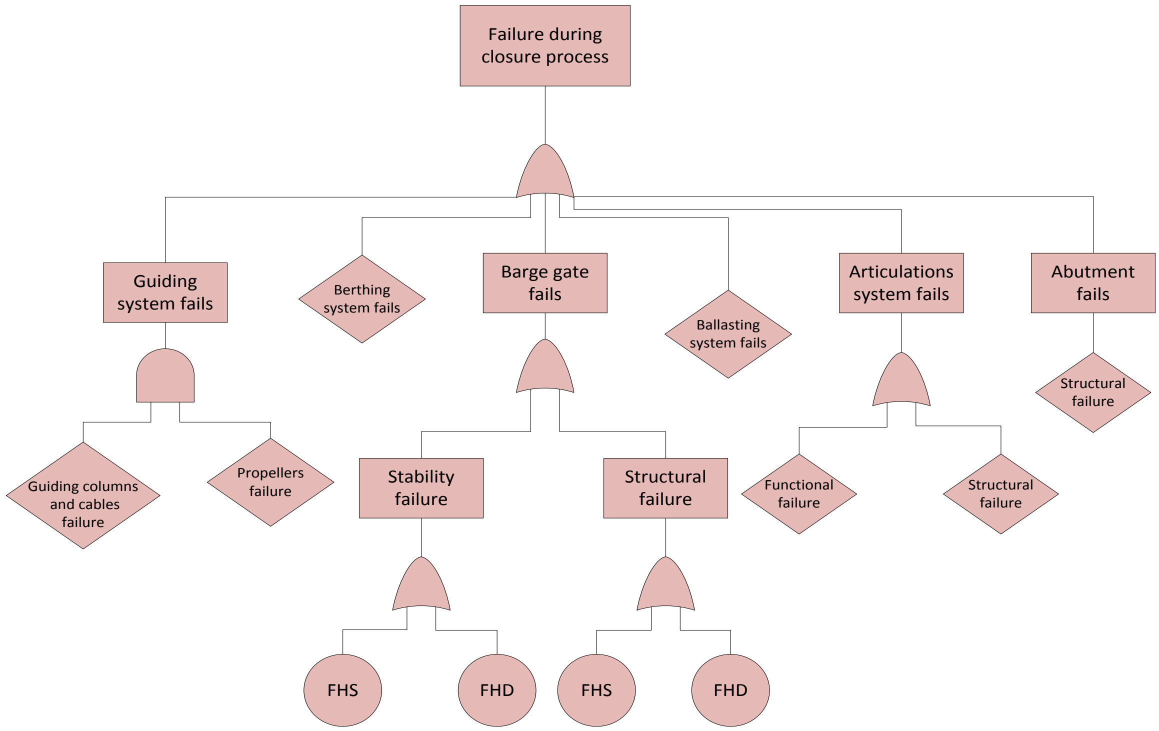


Figure 9-7: Failure during closure process fault tree

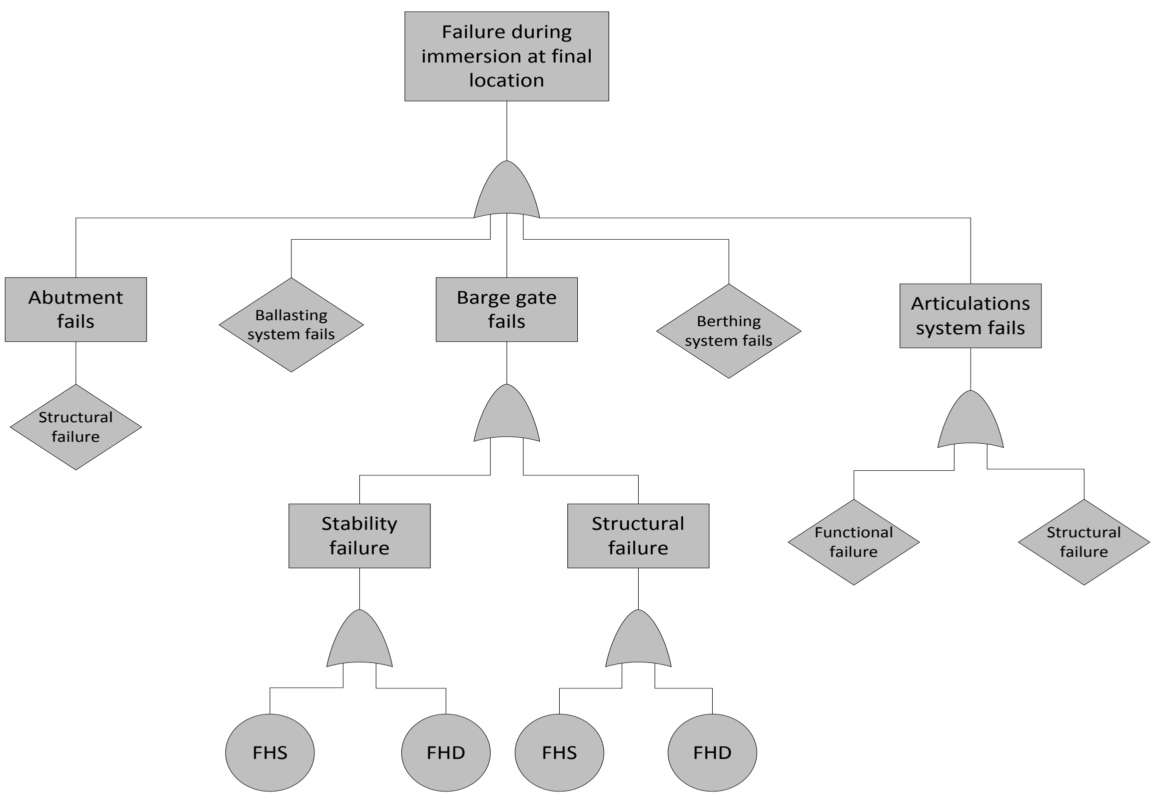


Figure 9-8: Failure during immersion at final location fault tree

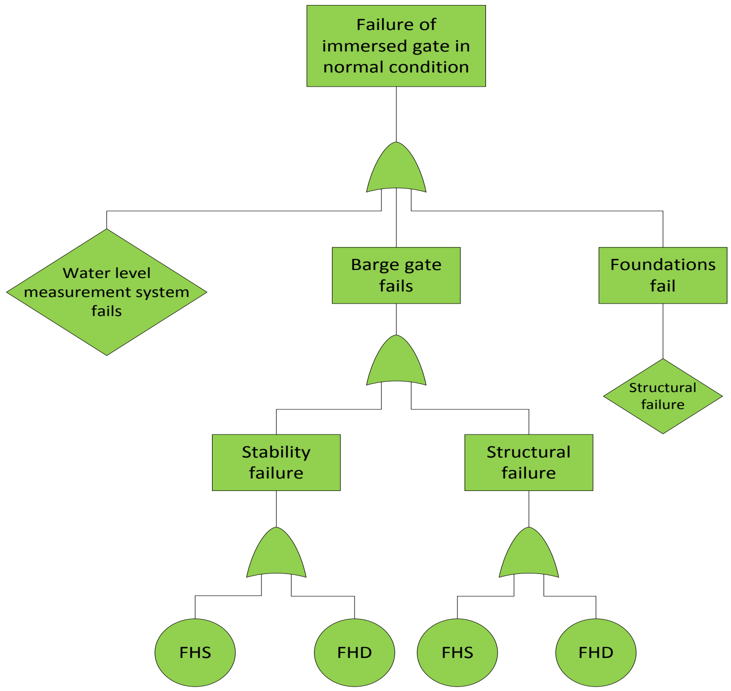


Figure 9-9: Failure of immersed gate in normal condition fault tree

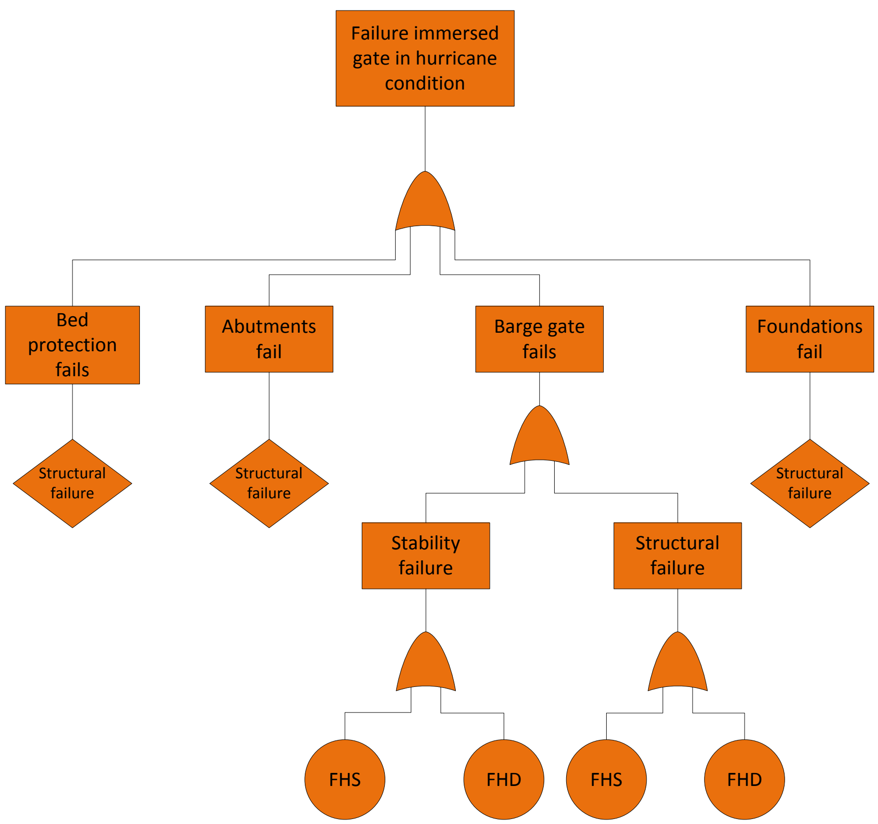


Figure 9-10: Failure immersed gate in hurricane condition fault tree

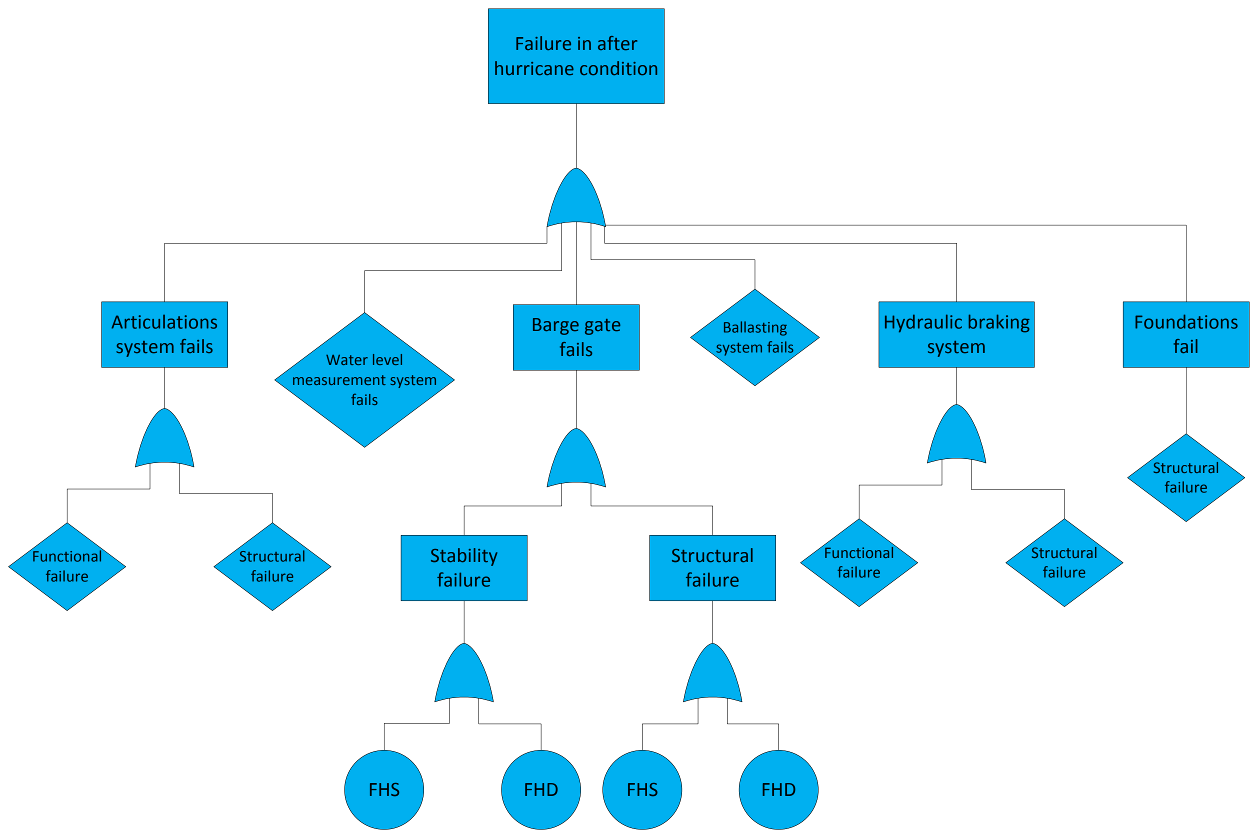


Figure 9-11: Failure in after hurricane condition fault tree

9.4 Design Challenges and Following Tasks

On the basis of the risk analysis in the previous sections, there is a need to make clear what are the missing points up to this design step and elaborate more some of them. Below the list of possible tasks to make the design more complete are provided for different design steps. According to this list in the following sections it is going to be tried to elaborate some of the design challenges in more detail. The other tasks should be followed in the future research.

Design Phase:

- Advanced numerical study of the safety level and the design storm
- Advanced numerical study of the retaining height
- Studying the interaction of the navigational and environmental barrier
- Iterative design procedure regarding the type of the navigational barrier and checking the requirements during the design steps
- Integral design of the barrier considering all the adjustments in the inputs
- Design optimization of the gate and complementary elements

Construction Phase:

- Precise instruction for the mix design
- Quality control and execution management and control
- Safety guidelines and controls
- Dry dock location selection and subsoil investigations

Transportation and Placement Phase:

- Stability and structural analysis of the structure in more detail
- Calculation of the forces transferred to the tugboats
- Ballasting system design and calculations

Operational Phases:

Open position

- Hydrodynamic analysis of the floating structure
- Calculation of the forces to the articulation system
- Design of the articulation system
- Design of the foundations
- Design of the mooring system (cables)

During closure:

- Hydrodynamic calculations of the gate during closure
- Calculations of the dynamic forces transferred to the gate and articulation system
- Design of the propellers
- Design of the berthing system (tyres and Teflon skids)
- Design of the articulation system, guiding columns and cables
- Design of the system of floats for the gate
- Calculations regarding the closing time and procedure of closing

During immersion:

- Hydrodynamic calculations and analysis of the gate during immersion
- Design of the ballasting system
- Design of the berthing system (tyres and Teflon skids) for hydrostatic loads during immersion
- Design of the articulation system
- Vibration analysis of the gate

Immersed gate normal condition:

- Dynamic analysis of the gate
- Design of the abutments
- Design of the foundations
- Design of the lock for negative head
- Hydrodynamic and vibration analysis

Immersed gate hurricane condition:

- Hydrodynamic analysis, vibration analysis and design of the gate in this situation (considering the underflow)
- Design of abutments
- Design of foundations
- Design of bed protection in more details

After hurricane condition:

- Hydrodynamic analysis of the gate in this situation
- Calculations regarding stability of the gate against the negative head
- Design of hydraulic braking system
- Design of ballasting system for un-ballasting situation
- Design of the openings in the gate (if applicable), a mechanical lock or a civil structure for the negative head stability
- Research about the water level measurement system and design of it

9.5 Conclusions

In this chapter design evaluations and reflections and risk analysis have been done for the project. Through the design evaluation and reflection the important design parameters and their influences on the final design have been investigated. Also, the risks analysis has been performed with the focus on the technical aspects. Risks in different phases of the project, including design, construction, transportation and placement and operational phases, have been identified and the mitigation actions and following tasks in the design have been described. The maintenance and demolition phases are not considered here. Also other risks such as management risks, financial risks, etc. are out of the context of this report.

By using the risk register tables, the qualitative analysis of the risks has been done and the most important risks have been identified. Also, because of the importance of the operational phases of the barge gate, the fault trees for different phases have been constructed.

According to the information gained from the risk analysis, the most important following tasks in the integral design procedure have been summarized. Due to the time constraints of the current project, in the rest of this report only few items of the required tasks can be realized. The other tasks are strongly recommended to be performed in future researches.

10 Design Revisions (DS6.)

10.1 Introduction

In this chapter some design revisions are going to be realized on the basis of the important design parameters and risks analysis which have been described in the previous chapter. In fact, this chapter considers the design iteration and revision (Figure 10-1). Checking the effects of all the important design parameters and needed tasks requires more time which is not possible because of the time constraints of the current research project.

The focus of the current design iteration and further investigations is on the construction material, retaining height, prestressed concrete design, negative head stability and initial hydrodynamic analysis.

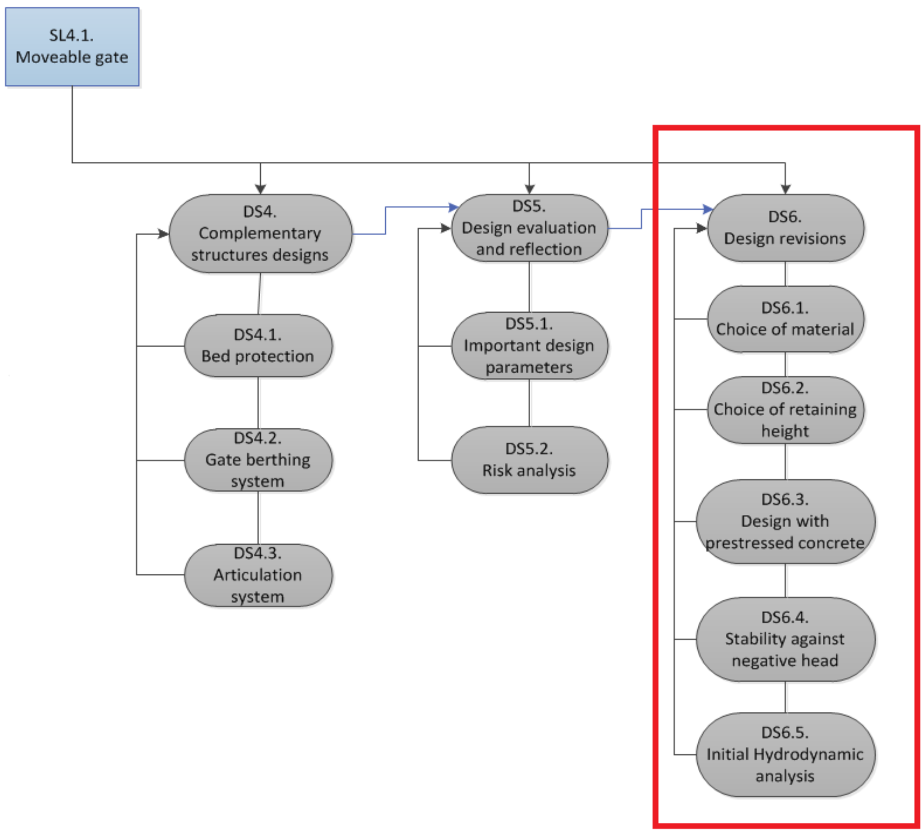


Figure 10-1: Design step 6; Design revisions

10.2 Choice of Material (DS6.1.)

In this report the barge gate has been designed by using the reinforced concrete material. It would be interesting to compare the results of this design with another design of the barge gate with different material.

Iv-Infra has designed the barge gate in steel [73]. In this design the barrier has been considered with a length of 220 m, width of 40 m and a height of 22 m. So, the length of the barrier is 10 m and height of the barrier 0.5 m less than the current concrete design and the width of it is 4 m more.

In the steel design the gate consists of an external shell (including skin plates, side plates, top and bottom plate), the buoyancy chamber and the frame structure (including columns, beams along two directions and diagonals) (see Figure 10-2). The material used is S355 and the gate is modeled in FE model with SCIA Engineer 2013 software.

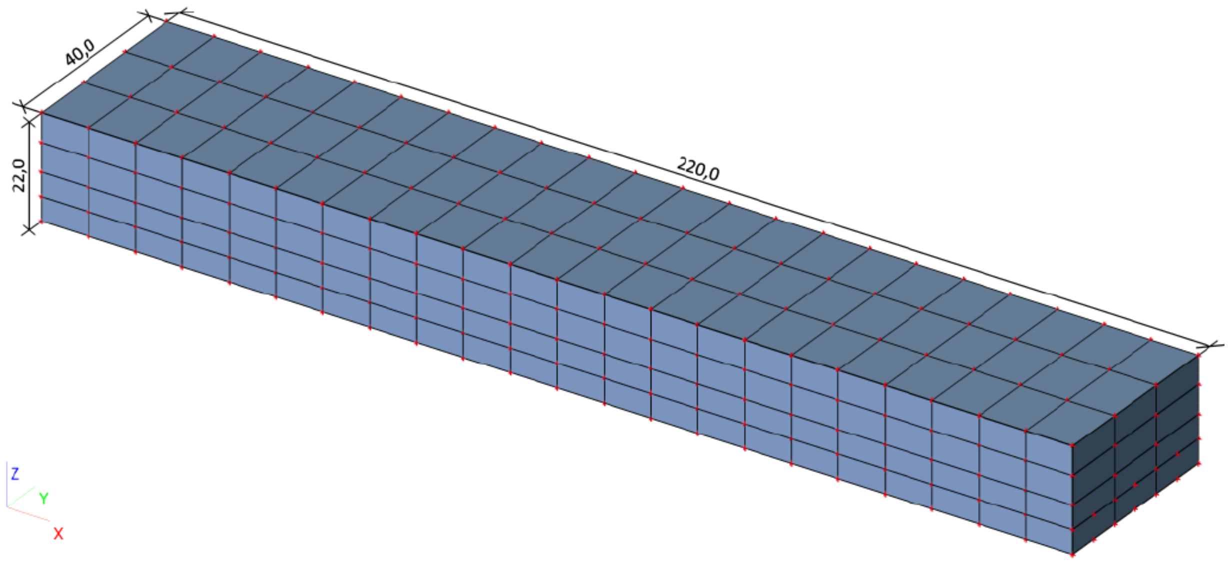


Figure 10-2: Barge gate designed with steel material [73]

This design considers the hydrostatic horizontal load as 1400 kN/m per running meter of the barrier, vertical hydrostatic load as 2000 kN/m per running meter and hydrodynamic loading as 20% of the hydrostatic load. Also, the load combination factors have been used in the design; 0.5 for self-weight, 1.30 for hydrostatic load and 1.00 for hydrodynamic load.

There are some differences between the assumptions of this design and the concrete design. For example, the loading assumptions are different and in the concrete design no load combination factors have been considered. Also, the steel design is done in Ultimate Limit State (ULS) while the concrete design in SLS.

The steel design results in a structure with a total weight of 31,715 tons. It can be seen that there is a huge difference between the weight of the structures in two designs. In the concrete design the final weight of the structure was 70,779 tons which is more than the weight of the steel barge gate. However, the concrete design is done roughly with the hand calculations.

Using the software for optimizations of the design will lead to probably less thicknesses for the walls and floor slab of the caisson and consequently less weight for the structure. Design of the gate in prestressed concrete will even lead to a lighter structure. In addition, the more length of the gate in concrete design is the other reason for more total weight.

The cost comparison between the two designs can be done. The steel structure including all the required works will roughly costs 4.5 euro per kg (4.5 * 1.38 = 6.21 $/kg) [74] while the concrete including the reinforcement and other required works will cost roughly 700 euro per cubic meter (966 $\$/m^3$) for high strength concrete [74]. For the concrete design of the barge gate the required amount of concrete material is 39,322 m^3 and considering 1.1 safety coefficient almost 43,254 m^3. Thus the cost calculation for the two designs are as follow:

- Steel barge gate costs ≈ *142,717,500 euros (196,950,150 $)*
- Concrete barge gate costs ≈ *30,277,940 euros (41,783,557 $)*

It can be seen that the concrete structure significantly costs less than the steel structure. This can be considered as an advantage of the concrete barge gate.

The other disadvantage of the steel is the amount of maintenance that is required to prevent corrosion [75]. Concrete (specially HPLC) has the advantage that it does not wear due to the presence of water as long as the concrete remains under compression. However, in case of cracks due to tensile stresses, water will reduce the fatigue resistance of the concrete.

Maybe the main reason that concrete is not used in shipping industry is that the weight of the concrete is more and it reduces the carrying capacity of the ship. This disadvantage is not applicable for the current barge gate because the structure does not need a carrying capacity.

However, due to higher weight of the concrete barge gate compared to steel barge gate, the closure procedure of the gate would take more time and also more forces would transfer to the articulation system. Also, ballasting and un-ballasting of the gate is more difficult when the weight is higher.

In general, the advantages and disadvantages of the concrete barge gate compared to steel barge gate are summarized in Table 10-1.

Comparison concrete material to steel material for the barge gate	Advantages	Disadvantages
	Durability	More total weight
	Less construction costs	Slower closure
	Less maintenance	More forces to connections
	Environmentally friendly	Higher costs of transportations
		Difficult ballasting

Table 10-1: Comparison of concrete material to steel material for barge gate

The final choice of the material should be used by evaluation of the different aspects such as design aspects, investment (life-cycle) costs, maintenance and operation of the barge gate.

10.3 Choice of the Retaining Height (DS6.2.)

As it has been mentioned in section 6.4, according to the research of de Vries (2014), the retaining height of the MSL+0.1 m has been found as sufficient for both of the environmental barrier and the navigational barrier [11]. In the previous chapters, the gate has been designed for the full retaining of the surge. Less retaining height might lead to less material usage and consequently less total costs of the project while maintaining the desired level of safety in terms of the hurricane. It can be proposed that in any condition the barrier should be able to withstand at least the forerunner surge (MSL+2.5 m).

Due to the fact that the barrier is afloat and there is a 1 meter opening under it, the leakage under the barrier during the hurricane happens. Then the retaining height of the navigational barrier can roughly be estimated as MSL+1.5 m to meet the requirement derived from de Vries (2014). However, this selection of the retaining height is just for checking the possibility of construction of the barrier with less retaining height and it does not consider the actual effect of the barrier on the see level rise during the hurricane on the Galveston Bay. For more precise view about this fact, one can model the afloat barrier (with 1 meter opening under the gate) with different retaining heights and check the best applicable level of retaining which is out of the scope of this research project.

As one step in the design revisions and iterations, the barrier is designed with the retaining height MSL+1.5 m. In this situation, the overflow of water from top of the barrier is happening. Figure 10-3 shows the barrier with reduced retaining height on its final position during hurricane condition.

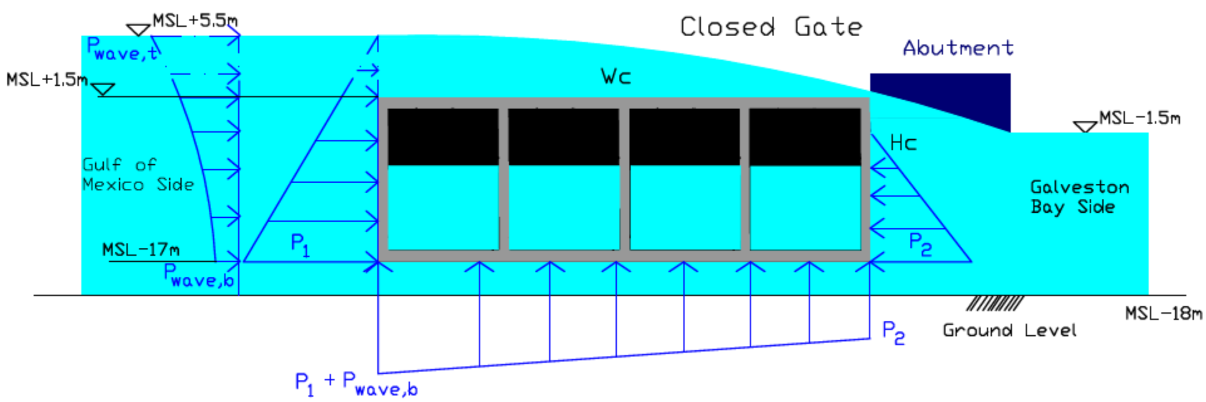


Figure 10-3: Barrier with reduced retaining height (MSL+1.5 m) during hurricane conditions at final position

In this situation the gate should be redesigned again for all the operational situations mentioned in the previous design steps. Indeed, all the design checks regarding structural design and stability checks should be done for the new dimensions and new load cases. The height of the gate in this case is reduced to 18.5 m (instead of 22.5 m in the previous design step) while the other dimensions are kept the same. The results of the design calculations can be seen in Table 10-2 to Table 10-5.

Final Barge Gate Geometry (Reduced Retaining Height)			
Parameter		Value	Unit
L_c	Length of the caisson	230	m
W_c	Width of the caisson	36	m
H_c	Height of the caisson	18,5	m
t_w	Thickness of the wall	0,9	m
t_f	Thickness of the floor slab	1,2	m
t_r	Thickness of the roof slab	0,6	m
t_{iw}	Thickness of the compartments	0,4	m
n_x	Number of compartments in width direction	5	-
n_y	Number of compartments in lenth direction	22	-
W_c	Caisson weight (with reinforcement)	63.889	ton

Table 10-2: Final barge gate geometry with reduced retaining height (Hc=18.5 m)

Draught and Ballasting information (Reduced Retaining Height)			
Parameter		Value	Unit
d_i	Initial draught of the gate during transportation	7,11	m
[1]$h_{ballast}$	Height of the ballast water in caisson normal condition	11,62	m
[2]$h_{ballast}$	Height of the ballast water in caisson hurricane condition	15,00	m

Table 10-3: Draught and ballasting information for the barge gate with reduced retaining height (Hc=18.5 m)

[1] This amount of ballast is desired for making the gate gets the draught of 17 m when it is normal condition
[2] This amount of ballast is desired for making the gate gets the draught of 17 m when it is hurricane condition

Stability checks (Reduced Retaining Height)		
Design Parameter	Value	Design Requirement
Static stability floating condition	10,70	should be > 0,5
Dynamic stability floating condition	2,02	should be > 2
[1]Static stability during immersing	7,26	should be > 0,5

Table 10-4: Stability checks for the barge gate with reduced retaining height (Hc=18.5 m)

[1] The static stability has been checked for different immersed depths. The current amount is for the desired final immersed level.

Structural Design checks (Reduced Retaining Height)		
Design Parameter	**Value**	**Design Requirement**
Design Check 1: Floating Caisson		
Shear check wall	19,24	should be > 1
Shear check floor	7,87	should be > 1
Shear check roof	18,10	should be > 1
Moment check wall	1,31	should be > 1
Moment check floor	1,05	should be > 1
Moment check roof	1,28	should be > 1
Reinforcement % wall	0,08	should be < 1
Reinforcement % floor	0,41	should be < 1
Reinforcement % roof	0,48	should be < 1
Design check 3: Gate immersed at Final Location During Normal Conditions		
Shear check floor	1,01	should be > 1
Moment check floor	1,14	should be > 1
Reinforcement % floor	0,74	should be <1
Design Check 4: Gate Immersed at Final Location During Hurricane Conditions		
Gate in Floating Condition During the Hurricane		
Shear check wall	1,79	should be > 1
Shear check floor	2,70	should be > 1
Shear check roof	20,11	should be > 1
Moment check wall	1,23	should be > 1
Moment check floor	38,28	should be > 1
Moment check roof	1,29	should be > 1
Reinforcement % wall	0,83	should be < 1
Reinforcement % floor	0,08	should be < 1
Reinforcement % roof	0,52	should be < 1
Gate supported on Abutments (Horizontal Supports) During the Hurricane		
Shear check wall	1,01	should be > 1
Moment check wall	1,16	should be > 1
Reinforcement % wall	0,60	should be < 1
Gate Supported on Foundation (Vertical Supports) During the Hurricane		
Shear check floor	1,01	should be > 1
Moment check floor	1,13	should be > 1
Reinforcement % floor	0,90	should be < 1

Table 10-5: Structural design checks for the barge gate with reduced retaining height (Hc=18.5 m)

The following main loading cases will change in this situation compared to the full retaining height:

- The hydrostatic loads will be reduced
- The wave loads will be reduced
- Suction forces on the bottom of the barrier will be reduced
- The water weight should be considered on top of the structure because of the overtopping

The differences between the barrier for full retaining height (designed in design step 3) and reduced retaining height (designed in design step 6.2.) are shown in Table 10-6.

Differences for full retaining height and reduced retaining height			
Parameter	**Hc = 22,5 m**	**Hc = 18,5 m**	**Difference**
Weight (tons)	70.779	63.889	6.891
Concrete (m^3)	39.322	35.494	3.828
Cost (euros)	30.277.851	27.330.198	2.947.652

Table 10-6: Differences between the gate designs with different retaining heights

Obviously, less retaining height will lead to less weight, less material and more economical design. However, as it is mentioned before, it is recommended that a modeling of the afloat barrier with opening under the gate should be done to find out about the most efficient retaining height. The detailed calculations regarding this section are not included in the report.

10.4 Design with Prestressed Concrete (DS6.3.)

As part of the design iterations, the barge gate can be designed in prestressed concrete. Prestressed concrete has the advantages such as more efficient members (smaller members to carry the same loads) and less cracking because the member is almost totally in compression. The disadvantages are however more expensive material, fabrication and delivery and design complexity.

The type of prestressing is determined mostly by the construction method of the elements. Methods of prestressing are as follow [76]:

- Prestressing with post-tensioned steel with bond
- Prestressing with post-tensioned steel without bond
- Prestressing with pre-tensioned steel

In this section the design of the barge gate with the full retaining height (MSL+5.5 m) is considered to be done by using the post-tensioned steel with bond. In this method, initially ducts are installed in the formwork or mould and the concrete is cast [76]. When the concrete has sufficient compressive strength, the tendons (or strands) should be installed. Then the tendons should be tensioned and anchored. After installing the anchorages, the open space between the tendons and the ducts should be injected by a special grout to protect the corrosion in steel. The anchorages should be also covered with for example concrete for the corrosion protection.

The calculations for the prestressed concrete design have been done roughly and only for the critical loading situations. The theories and the details of the calculations can be found in section 14.2.

The results of the analysis show that the dimensions of the concrete caisson can be reduced by using the prestressed concrete as it was expected (Table 10-7 and Table 10-8). These reductions are in the thicknesses of walls and floor slab of the gate. The weight of the barge gate in this design is reduced to 63,724 tons.

Final Barge Gate Geometry (Prestressed Concrete)			
Parameter		Value	Unit
L_c	Length of the caisson	230	m
W_c	Width of the caisson	36	m
H_c	Height of the caisson	22,5	m
t_w	Thickness of the wall	1,0	m
t_f	Thickness of the floor slab	1,0	m
t_r	Thickness of the roof slab	0,5	m
t_{iw}	Thickness of the compartments	0,3	m
n_x	Number of compartments in width direction	5	-
n_y	Number of compartments in lenth direction	22	-
W_c	Caisson weight (with reinforcement)	63.724	ton

Table 10-7: Final barge gate geometry using the prestressed concrete design

Draught and Ballasting information (Prestressed Concrete)			
Parameter		Value	Unit
d_i	Initial draught of the gate during transportation	7,09	m
$^1h_{ballast}$	Height of the ballast water in caisson normal condition	11,42	m
$^2h_{ballast}$	Height of the ballast water in caisson hurricane condition	15,00	m

Table 10-8: Draught and ballasting information for prestressed concrete design

The design in prestressed concrete has been done roughly in this section. For sure more detailed designs are needed to optimize the dimensions and reinforcement mapping of the caisson structure.

A rough comparison can be made between the cost of the preliminary design with normal reinforced concrete and with the prestressed design. Cost of a prestressed concrete can be estimated roughly as 900 euro per cubic meter (1250 $\$/m^3$) [74]. The concrete needed for the prestressed barge gate is equal to 35,402 m^3. Considering 10% safety in material amount, the comparison of the costs are as follow:

- Normal reinforced concrete barge gate cost ≈ *30,277,940 euros (41,783,557 $)*
- Prestressed concrete barge gate cost ≈ *35,047,980 euros (48,677,750 $)*

The prestressed concrete gate costs more while has the advantages of the less total weight and better durability specially in terms of the cracks. Recently many offshore structures are in prestressed concrete and it seems to be a better option for the barge gate structure. In the rest of this report the design of the barge gate with prestressed concrete is considered for further analysis.

10.5 Stability Against Negative Head (DS6.4.)

After the hurricane it would be a situation when the sea level in the Bay side is higher than in the Gulf side. Results of analysis show that the negative head up to 3.5 meter is also possible when the landfall location is on the east of the Bolivar Roads. It would be interesting to calculate that up to which level the barge gate would be stable against the negative head. The system of the barge gate with negative head is shown in Figure 10-4.

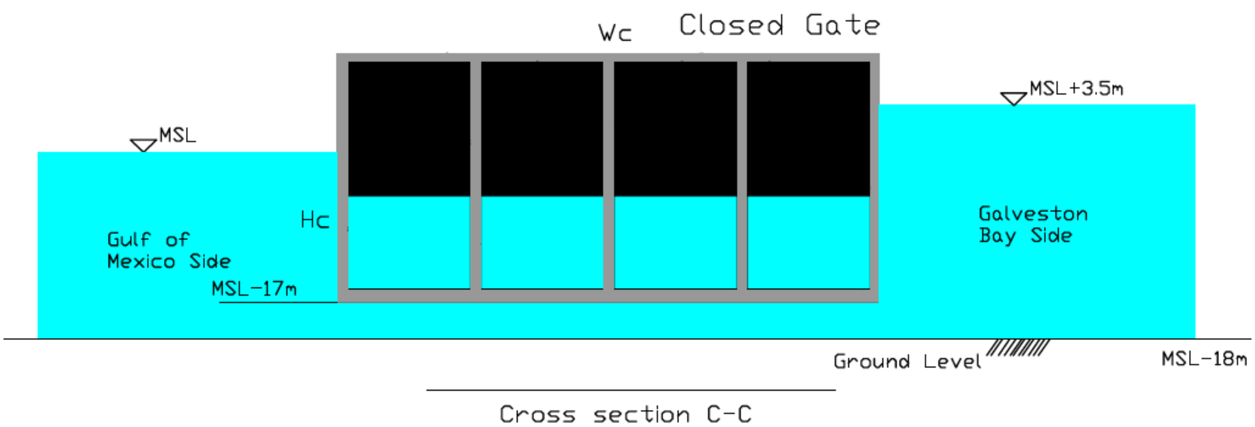


Figure 10-4: Negative head (after the hurricane condition)

Considering the information of the prestressed concrete barge gate design, the rough calculations have been done regarding the horizontal stability of the gate against the negative head (See Section 14.6 for detailed calculations). The horizontal stability can be derived from equation below:

$$\Sigma H \leq f.\Sigma V \qquad \text{Eq. 10-1}$$

In which:

- ΣH= Sum of horizontal forces
- ΣV= Sum of vertical forces
- f= friction coefficient between barge gate and the foundations (rubber damping fabrics)

The assumptions of the calculations are as follow:

- The gate has the total weight of 63.724 tons (derived from prestressed concrete design)
- There is 15 m of ballasting water inside the gate
- The hydrostatic forces are acting on the gate
- The friction coefficient (f) between the barge gate and the foundations (rubber damping fabrics) is assumed as 0.35
- The suction forces are not considered which is on the conservative side

The results indicate that the gate is stable up to the negative head of MSL+2.8 m. This might be surprising because the gate is not supported against the negative head. The possible reasons for clarifying the horizontal stability are as follow:

- The concrete gate has the high self-weight and it is also ballasted with extra ballasting water which makes the vertical downward forces considerable
- It is assumed that the resulting vertical forces are transferred to the foundations at two ends of the barrier and provide the horizontal stability
- Possible wave forces and dynamic forces are not considered
- The foundations should be strong enough to provide such a high horizontal friction and interaction with the gate

In case the negative head exceeds the MSL+2.8 m extra measures should be considered such as:

- Opening the environmental barrier to prevent the huge negative head on the navigational barrier
- Implementing openings and valves in the barge gate and opening them during the negative head to reduce the horizontal forces which is an expensive solution
- Using a mechanical lock or a civil structure (maybe from concrete material) for bearing the extra forces (or even all the forces). (Figure 10-5)

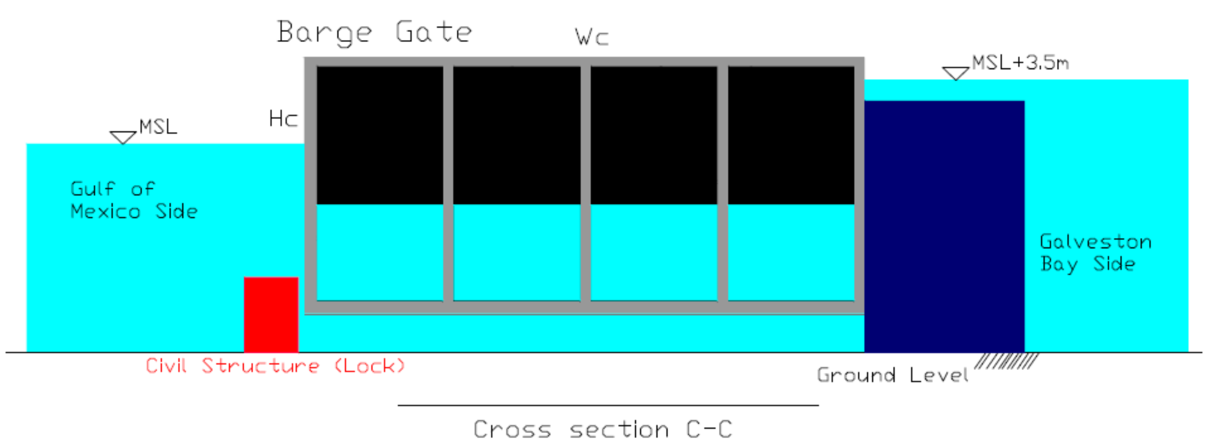


Figure 10-5: Location of civil structure (lock) for negative head

If the last option, using the a lock or a civil structure, is considered, it should be able to withstand around 185 kN/m of the length of the gate force for MSL+3.5 m negative head. Using this option is more safe because maybe the rubber damping material on the foundations wouldn’t provide enough friction with the barge gate and then the lock or the civil structure would support the gate horizontally against the negative head.

Further investigations and precise analysis of the situation for the negative head is recommended in the future research.

10.6 Initial Hydrodynamic Analysis (DS6.5.)

The design of the barge gate has been done in the previous sections by considering the static loads acting on the structure. According to the hydraulic structures design knowledge, through the risk analysis and design evaluations, it is found out that the hydrodynamic loads and motions would affect the design of the structure considerably. Because of the time limits of the current research the complete hydrodynamic analysis of the structure is not possible. However, in this section the introduction to the analysis and the recommendations for further research are provided.

In general the hydrodynamic loads and dynamic motions of the structure under the influence of the incoming waves are important. In the project area, the incoming waves can be summarized as below:

- Wind waves which are irregular waves and can be divided as sea waves (generated by local winds) and swell waves (long period waves generated by local wind but which have travelled away from the local wind area)
- Astronomical tides with a very long period
- Short waves generated by passing ships

The hydrodynamic analysis of the structure is particularly important during the closure phase of the gate when the gate is afloat. The closure of the gate should be done during the normal condition and prior to the hurricane. Indeed, the boundary conditions in this situation are the normal boundary conditions.

It can be expected that the long waves such as tides cannot affect the barge gate motions strongly because the closure procedure is fast and the barge gate have enough time to follow the movements of the water plane. Thus, the hydrodynamic forces and motions due to the energy of shorter wind waves are more important for the initial analysis.

The analysis can be divided to three different parts including:

- Analysis of the articulation system (swing point) under the hydrodynamic loads and motions transferred from the barge gate to it
- Analysis of the barge gate itself under the hydrodynamic loads and motions
- Vibration analysis of the barge gate due to underflow at the closed position or during the immersion

10.6.1 Loads on the Articulation System

During the closure, the barge gate is affected by environmental loads including wave, current and wind loads. According to the required degrees of freedom described in section 0, the articulation system should be able to resist (restrict) the forces in horizontal directions (surge and sway), mainly in sway direction (Figure 10-6). It has been assumed that the other degrees of freedom are free in the articulation system. The horizontal loads on the sway direction will help the gate to be closed during the closure procedure and the gate finally rests on the abutments and these forces will resist by the abutments.

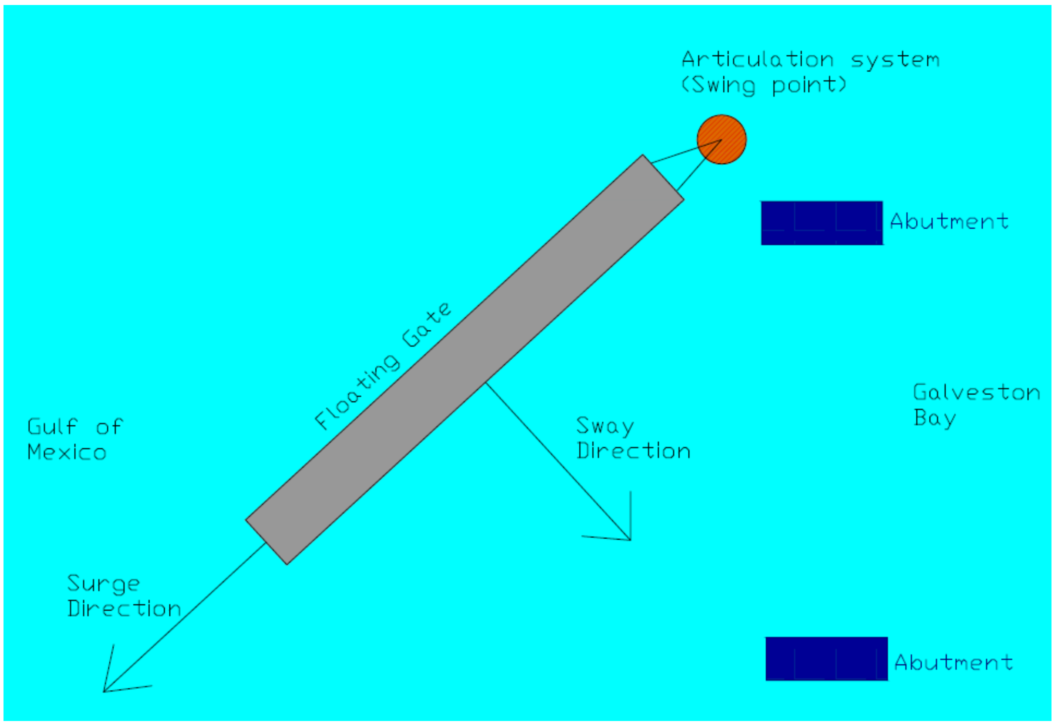


Figure 10-6: Loads on the articulation system in sway and surge directions

In this section, the forces acting on the floating barge gate are roughly calculated or estimated in some cases. Then an indication of the magnitude of the forces acting on the articulation system can be achieved.

In general, the forces acting on the floating barge gate can be summarized as below:

Steady forces:

Mean wave drift forces:

This force can be roughly estimated by the formula below [83]:

$$F_{md} = (1/16).\rho_w.g.L.[(1-C_T).H_{sig}]^2 \qquad \text{Eq. 10-2}$$

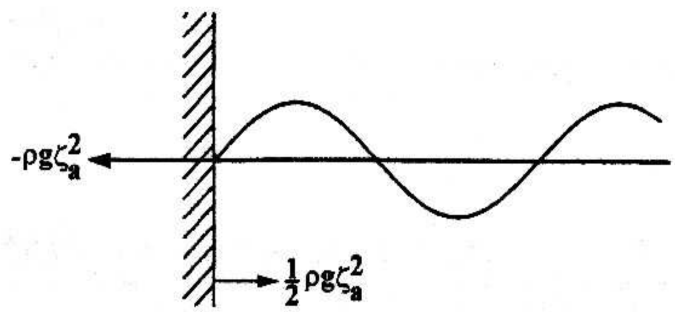


Figure 10-7: Wave drift forces

Current forces [83]:

$$F_c = (1/2).\rho_w.C_s.A.u_c^2 \qquad \text{Eq. 10-3}$$

Wind loads [83]:

$$F_w = (1/2).\rho_{air}.C_s.A.u_w^2 \qquad \text{Eq. 10-4}$$

In which:

- C_T = wave height transmission coefficient
- L = Length of the terminal in the direction of the force
- ρ_w = Water Density
- ρ_{air} = Air density
- H_{sig} = Significant wave height of the incoming wave
- A = Area of the structure in the direction of the force (different for wind loads and current forces)
- u_c = Current velocity
- u_w = Wind velocity
- C_s = Shape coefficient

Oscillatory motion induced forces:

These forces are as a result of the oscillatory motions of the barge gate including:

- 1st order wave frequency forces
- 2nd order low frequency wave forces

The magnitude of these forces can be derived after the hydrodynamic analysis of the barge gate and calculating the motions of the gate during its closure. Also, stiffness of the articulation system plays an important role here. At this stage of the research the information about the motions of the terminal is not available. Just as the first estimate the sum of the oscillatory forces are assumed as half (1/2) of the sum of the steady forces.

The calculations of the above mentioned forces can be realized considering the normal condition during the closure and the perpendicular forces to the structure. The results of the calculations have been shown in Table 10-9. The details of the calculations can be found in section 14.7.

Steady Forces			
Force	**Surge direction**	**Sway direction**	**Unit**
F_{md}	178	1137	kN
F_c	221	1412	kN
F_w	139	886	kN
Total	**538**	**3436**	**kN**

Table 10-9: Steady forces results

Conclusions of the initial analysis:

On the basis of the previous assumptions regarding the oscillatory motion induced forces, the total forces acting are as follow:

- Total acting force in surge direction: =1.5*538 = 807 kN
- Total acting force in sway direction: =1.5*3436 = 5154 kN

As it can be seen the forces in the sway direction are bigger because of the bigger projected area of the structure in the y direction (length of the structure). These rough forces can be used for the initial design of the articulation system which needs to provide the restrictions in surge and sway directions.

The exact and precise amount of the forces estimated above can be derived after the hydrodynamic analysis of the structure considering different motion effects and waves spectrums. This issue is out of the scope of the current research project.

10.6.2 Hydrodynamic Analysis of the Barge Gate

Hydrodynamic analysis of the barge gate is an important aspect of the project. In this section an introduction to the hydrodynamic behavior of the barge gate is provided. The extended analysis is left for the future research.

The hydrodynamic behavior of the barge gate is important in different operational phases:

- **During the closure;** when the gate is floated from the open position to close position. The hydrodynamic loads and motions would be normative for the design of the barge gate itself and it affects the other complementary structures such as articulation system.
- **During the immersion of the gate;** when it rests on the abutments and it is immersed using the water ballasting the gate can experience extra hydrodynamic forces due to immersion. Also, shaking of the gate is possible which put extra hydrodynamic loads to the structure.
- **During the closed situation;** when the gate is closed the underflow makes hydrodynamic forces to the structure and the gate will vibrate because of the hydrodynamic forces.

In the static analysis of the structure the natural oscillation period of the barge gate was calculated and it was checked that the gate is dynamically stable in the floating condition (during the transportation). However, in that stage the hydrodynamic mass (additional water mass) and damping were ignored.

Considering ***during the closure phase***, the oscillatory motions in the barge gate are caused by the wave forces on the floating body. These forces can be divided to forces with frequencies equal to the incoming wave frequency and forces with low frequencies (low frequency wave drift forces). Each of these two forces can result in all 6 dynamic modes of motions (sway, surge, heave, roll, pitch and yaw). The definitions of the six modes of motions can be seen in Figure 8-11.

The most important motion of the structure during the closure is the ***roll motion*** which occurs because of the wave actions [84] (Figure 10-8). This motion will be maximum when the gate axis and the wave direction are perpendicular. The maximum roll angle is the most important characteristics in the design of the articulation system and the swing arm which connects the articulation system to the gate.

Heave motion will also be significant during the closure phase. Heave and roll only make displacements in the transverse plane of the barrier and have no effect on the longitudinal motion.

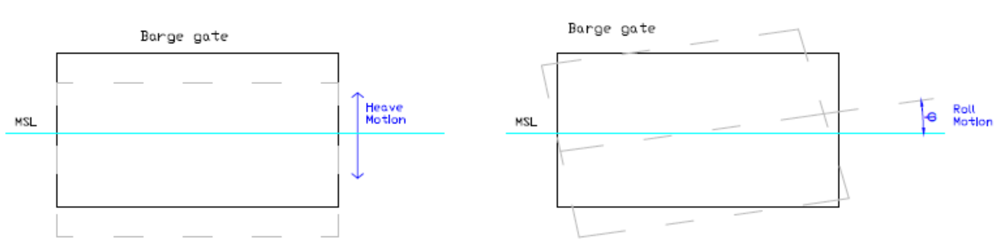


Figure 10-8: Roll and heave motions of the barge gate during the closure phase

In general it is expected that the motions of the barge gate during closure wouldn't be problematic for the operational or structural aspects because of the following reasons:

- Closure is during the normal conditions before the hurricane while the environmental loads are not severe
- The barge gate has the huge dimensions and the floats implemented in it make it more stable
- Guiding columns and cables help the structure to close safely and under control

Studying the motions of the barge gate is not in the scope of this report but is recommended through analytical or experimental studies.

10.6.3 Analysis of the Barge Gate on Vibrations

The barge gate will act like a vibrating system in the closed position because of the underflow. It is important to analyze the vibrating system of the gate. In this section a preliminary analysis in this regard will be performed. This analysis will consider the simplified approach.

The barge gate has a natural frequency and it can be loaded dynamically because of the following reasons [79]:

- Instability induced excitation; due to for example vortex shedding because of the underflow or water flow separation
- Extraneous excitation; due to for example an unsteady flow or wave action
- Movement induced excitation; due to for example when the water flow enhance the movement to which the gate is already subjected

In general, the gate has 6 degrees of freedom as it has been discussed in previous sections. Mostly the ***vertical vibration*** results in worst problems from a gate structure collision point of view [79]. Indeed, for the initial vibration analysis the vertical vibration is going to be considered in this report. The interesting situations for analysis are as follow:

- During the immersion of the gate at its final location (the vibration behavior of the gate will change with the immersed depth)
- When the gate is completely immersed to the final position

In this report the second situation, the immersed gate (Figure 10-9), will be considered and the other situation is recommended for future research.

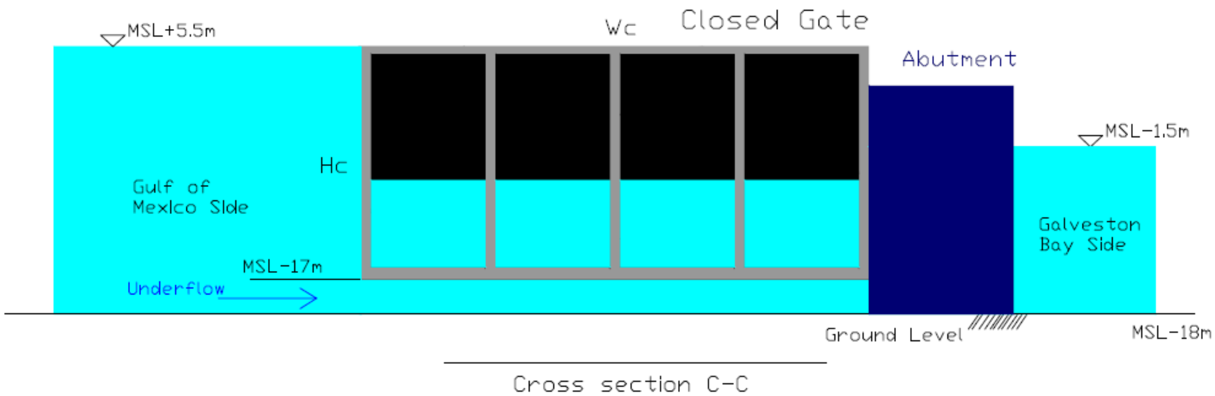
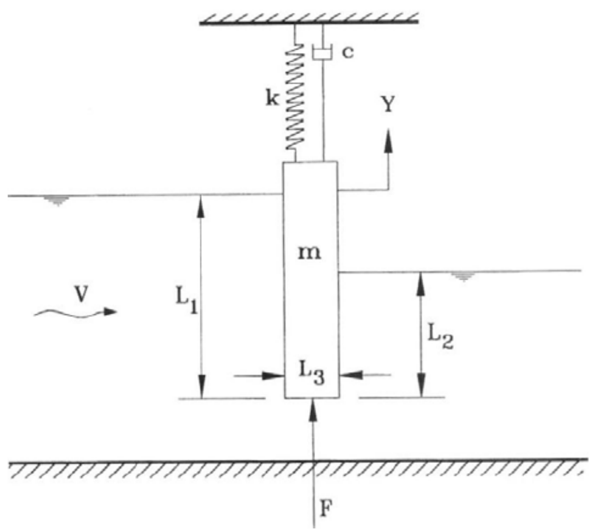


Figure 10-9: The situation considered for the vertical vibration analysis

The vibrating system can be modeled as the mass-spring-damper system (Figure 10-10) with the following equation:

$$(m+m_w)\frac{d^2y}{dt^2}+(c+c_w)\frac{dy}{dt}+(k+k_w)y=F_w(t,y) \qquad \text{Eq. 10-5}$$



m	=	mass	[kg]
m_w	=	added water mass	
c	=	damping	[Ns/m]
c_w	=	added water damping	
k	=	spring stiffness	[N/m]
k_w	=	added water stiffness	
L	=	representative length	[m]
v	=	flow velocity	
F or Fw	=	all static or dynamic forces by water	[N]
y	=	direction of analysis, here vertical	[-]

Figure 10-10: Simple mass-spring-damper system for the barge gate

The different values in the equation above should be determined first. The resonance frequency of the barge gate (f_r) and the excitation frequency (f) should be then calculated.

It is advised that the resonance frequency should be at least a ***factor of 3*** higher than the excitation frequency [60]. If the excitation frequency is smaller than the eigenfrequency (f_n) and well below the resonance peak, the system may be described as quasi-static which means that the structure may show minor response to the excitation source [79] (Figure 10-11).

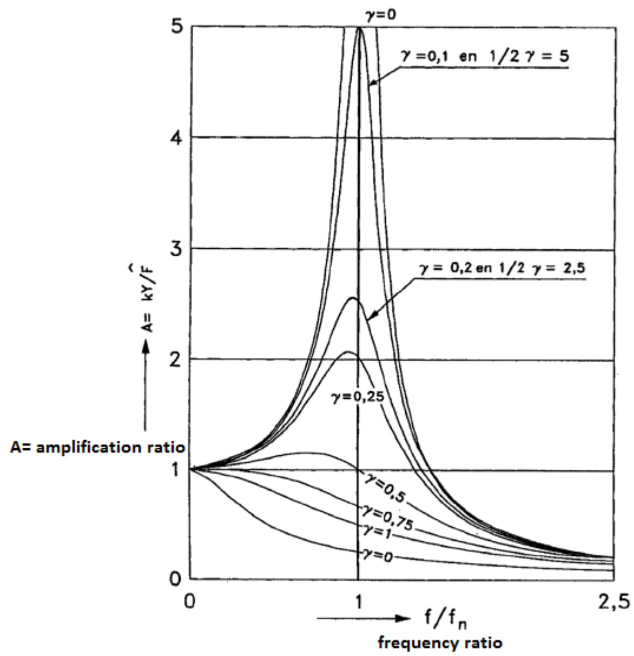


Figure 10-11: Response curve of a single degree of freedom mass spring system in the time domain [60]

It should be mentioned that the higher eigenfrequency modes of the barge gate should be checked in further steps of the design which should not to be close to the excitation frequency while generally they do not govern the design of a gate. In the case that f is larger than f_n, a ***ratio of 3*** is recommended and then the structure will not respond to the excitation.

On the basis of the calculations the following conclusions can be derived for the vertical vibration of the barge gate:

- The excitation frequency (f=0.74) is significantly higher compared to the 1st eigenfrequency (f_n=0.12) with the factor of more than 3 as it is advised (factor 6).
- The system is positively damped, the occurring oscillations have diminishing amplitude and the system is stable.

The above mentioned conclusions show a desirable situation in terms of the vertical vibration and resonance doesn't occur. It should be mentioned that the calculations here are based on assumptions and some simplifications. The complete analysis of the vibrating system considering the higher eigenfrequency modes of the structure is recommended for future research. The calculations regarding this section are provided in appendices section 14.8.

10.7 Conclusions

In this section different design iterations and revisions have been performed to check the barge gate design from different perspectives.

Considering the construction material, steel will offer less structural weight while concrete reduce the cost of the project and is better for the environment.

The retaining height of the barrier is the other aspect. The analysis has been done regarding the reduction of the retaining height from 22.5 m to 18.5 m. The barge gate is designed for this reduced retaining height and the total weight of structure was decreased to 6,890 tons or almost 10% less weight.

Also, the barge gate can be designed with prestressed concrete. The rough calculations, with considering the full retaining height, for the prestressed concrete has been done and the designed gate with the modified dimensions was derived. The total weight of the structure is reduced in this case to 63,724 tons which is almost 10% reduction compared to the original design with normal reinforced concrete.

It can be concluded that using the prestressed concrete and less retaining height will reduce the total weight of the structure. The design of the gate in prestressed concrete with the partially retaining height is recommended for gaining better view about the different design aspects.

For the current project, the final design of the gate is selected as the barge gate with prestressed concrete design and the height of the 22.5 m which is the fully retention height. This selection is just intended for the rest of the current research project. For the selection of the best option, considering different criteria such as construction material, normal or prestressed concrete and retaining height, a MCA is needed on the basis of the project needs and requirements and the authorities points of view.

Hydrodynamic analysis of the barge gate is the other important issue of this project. Initial hydrodynamic analysis is done in this chapter. The rough estimation is made for the forces acting on the articulation system. Also, during the closure of the barge gate the roll motion and heave motion are introduced as important motions of the structure. It is expected that the hydrodynamic motions wouldn't cause problems during the closure.

On the other hand, initial dynamic vibration analysis is done for the closed gate because of the underflow. It is found out that the gate is positively damped and the resonance wouldn't occur for the vertical vibration which is the most important vibration situation.

For the future research, extensive hydrodynamic analysis of the structure in different operational phases is recommended.

11 Supporting Structures

11.1 Introduction

In the previous chapters the preliminary design of the barge gate and the complementary structures have been done. In this chapter, the supporting structures are going to be considered (Figure 11-1). The information from the previous design steps is going to be used for the first estimation of the abutments design. Due to the time limits, design of the foundations is not treated in this report and only an overview of possible options is provided.

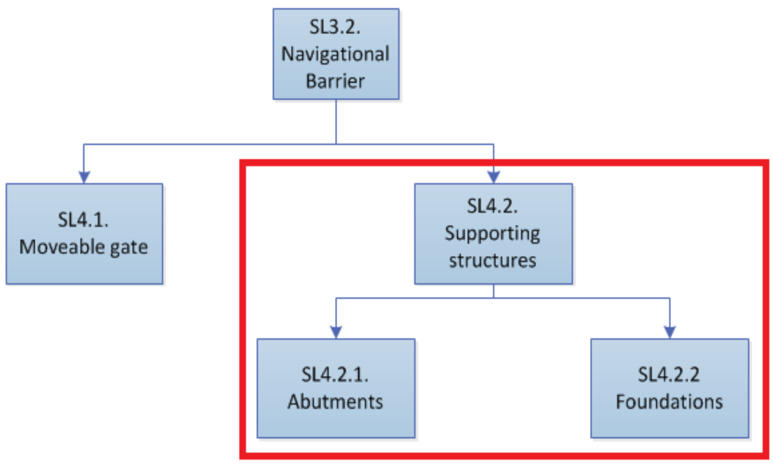


Figure 11-1: Supporting structures including abutments and foundations

11.2 Preliminary Design of the Abutments

The abutments are the important parts of the barrier. The design of the abutments should be done considering the requirements from the barge gate from the previous design steps. The preliminary design of the abutments is treated in this section. Firstly, the major loads acting on the abutments are described and then the conceptual design of the abutments and related calculations and structural design are going to be performed.

11.2.1 Loads

The qualitative description of the loads acting on the abutments are described here. The calculations in this regard can be found in section 14.5. The main loads acting on the abutments are as follow:

Hydraulic loads:

The load due to the head differences and wave pressures under critical design circumstances acting on the barge gate are transferred to the abutments. These loads should be calculated considering the reaction forces of the gate, which acts like a beam, on the horizontal supports (abutments).

Berthing loads:

During the closure of the gate before the hurricane condition, the gate will berth on the abutments. The berthing force acting on the abutments should be considered in designing of the abutments (See Section 0 for more information on calculation of this force).

Construction phase:

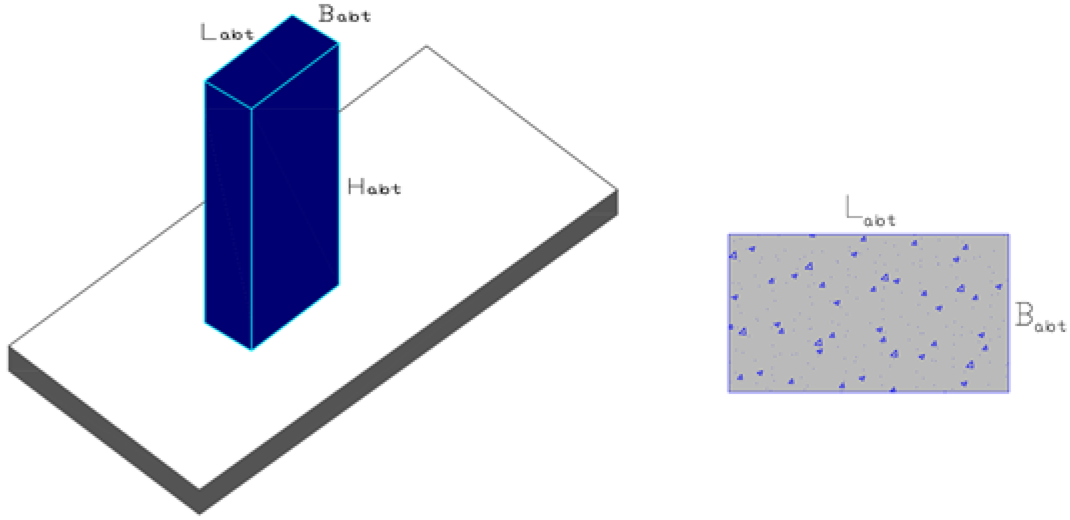
During the execution of the barrier, elements of the barrier might be loaded in a completely different way than after completion of the structure. Thinking about these loads is important. In this step of the design, the loads on the construction phase are not considered and the design can be adjusted later on for these loads. However, it is expected that the construction loads would not be normative if the appropriate construction method would be chosen.

11.2.2 Initial Design

For the initial design, the abutments can be considered as piers. Piers are gravity structures which be made of concrete. Piers can be constructed in a dry dock and then transferred to the location by a vessel. The important aspect here is the weight of the pier. The crane should be able to lift the pier and also maybe the special vessel is needed for the transportation. It can be concluded that the maximum dead weight of the piers should be derived in the early stages of the design and maybe adjusted on the basis of the crane and vessel capacity. Otherwise special lifting and transportation equipment should be built.

According to the previous design steps, heavy loads from the barge gate will act on the abutments. The initial method in the design can be with the normal reinforcement concrete. If the design of the abutment in this method leads to high weight of the structure, then the design can be adjusted by using the prestressed concrete.

For the abutments, a vertical element that transfers all the loads from the barge gate to the foundations is required. As a rough first estimation of the shape of the structure for the preliminary design, the abutments can be presented as a shaft (pier) on a base slab (Figure 11-2). For sure this assumption is just the initial estimation for the shape of the structure which can be adjusted in the later steps of the design. Due to the time constraints the design of this simple model is performed in this report.



3D view of the abutment Cross section of the abutment

Figure 11-2: Abutment schematization for initial design

Calculations have been done considering two different load cases:

- During the hurricane (with maximum positive head)
- During the barge gate closure (including berthing forces of the barge gate)

In the hurricane conditions the wave and hydrostatic loads should be resisted by the abutments and in the closure process the berthing forces of the gate transferred to the abutment. The results of calculations can be seen in section 14.5.1. From calculations it is derived that the hurricane condition is normative because of the higher bending moment acting on the abutment.

The design of the abutment has been done for this configuration of the pier by using the normal reinforcement concrete. Using the concrete with γ_c =25 kN/m^3, the abutment with the dimensions as follow has been found as the optimal option:

- H_{abt} = 24 m
- L_{abt} = 10 m
- B_{abt} = 5 m

Using the reinforcement B500B, 504 ϕ40 are required. See Section 14.5.1 for complete design calculations. The abutment in this case has the weight of 3,000 tons. Using the lightweight concrete with γ_c =17 kN/m^3 will reduce the weight of the pier to 2,040 tons which is a considerable reduction. In the following step, the design can be optimized by using the prestressed concrete.

11.2.3 Abutment Design Optimization

According to the previous section, because of the high acting loads on the abutments, a heavy structure for the abutments is designed. In this section, the design can be optimized by using the prestressed concrete which leads to less weight for the structure next to other advantages.

The design and calculations have been done considering the prestressed concrete. See section 14.5.2 for the details of the calculations. Using the Y1860 prestressed tendons with each 30 strands of ϕ18, in total 49 tendons are required. The weight of the pier is reduced to 2,100 tons in case of using concrete with γ_c =25 kN/m^3 and to 1,428 tons in case of using lightweight concrete with γ_c =17 kN/m^3. The dimensions of the abutments are as follow:

- H_{abt} = 24 m
- L_{abt} = 7 m
- B_{abt} = 5 m

Next to the reduction of the total weight of the structure, the crack width in the concrete structure is also minimized and the reinforcement is better protected against corrosion in case of using the prestressed concrete.

11.2.4 Conclusions

In this section the preliminary design of the abutments have been realized. Two cases of normal reinforcement and prestressed concrete have been used. Abutments have been considered as the piers founded on foundations. The summary of the designs can be seen in Table 11-1.

Abutments design summary (each pier)					
Design Method	**Height (m)**	**Length (m)**	**Width (m)**	**Weight (t)**	**Reinforcement used**
Normal reinforcement	24	10	5	2.040	504 ϕ40
Prestressed method	24	7	5	1.428	49 tendons, 30 strands ϕ18

Table 11-1: Abutments design summary

Due to the advantages of the prestressed concrete regarding the weight of the structure and resistance in terms of corrosion, using the prestressed method is proposed for the abutments.

11.3 Foundations Overview

The foundations are the other part of the supporting structures. From the previous studies regarding the Bolivar Roads barrier, for example the study of de Vries (2014) [11], it is known that the soil characteristics of the project site is not suitable. However, the information regarding the subsoil situation is not complete and further survey and investigations are needed.

The dense layer of sand is located from almost MSL-40 m under the structure. This makes the options for foundations limited to the following choices:

- Preloading the subsoil to realize most of the expected settlements before construction of the barrier
- Replacing the clay layers by dense sand layers to make it strong (dense) enough
- Using deep pile foundations (for example steel tubular piles filled with concrete)
- Using pneumatic caisson or a cellular cofferdam which penetrated through most of the clay layers and reaches the dense layers

The first two options seem to be expensive and not appropriate because of the deep layers of weak soil. Also, the pneumatic caissons are hard to realize because the dense sand layers are located in a very deep layer. Indeed, deep pile foundations would be the best option for the foundation of the barrier. Figure 11-3 shows the deep foundations under the abutments. The design of the foundations is not considered in this report because of the time limits and it is recommended for the future research.

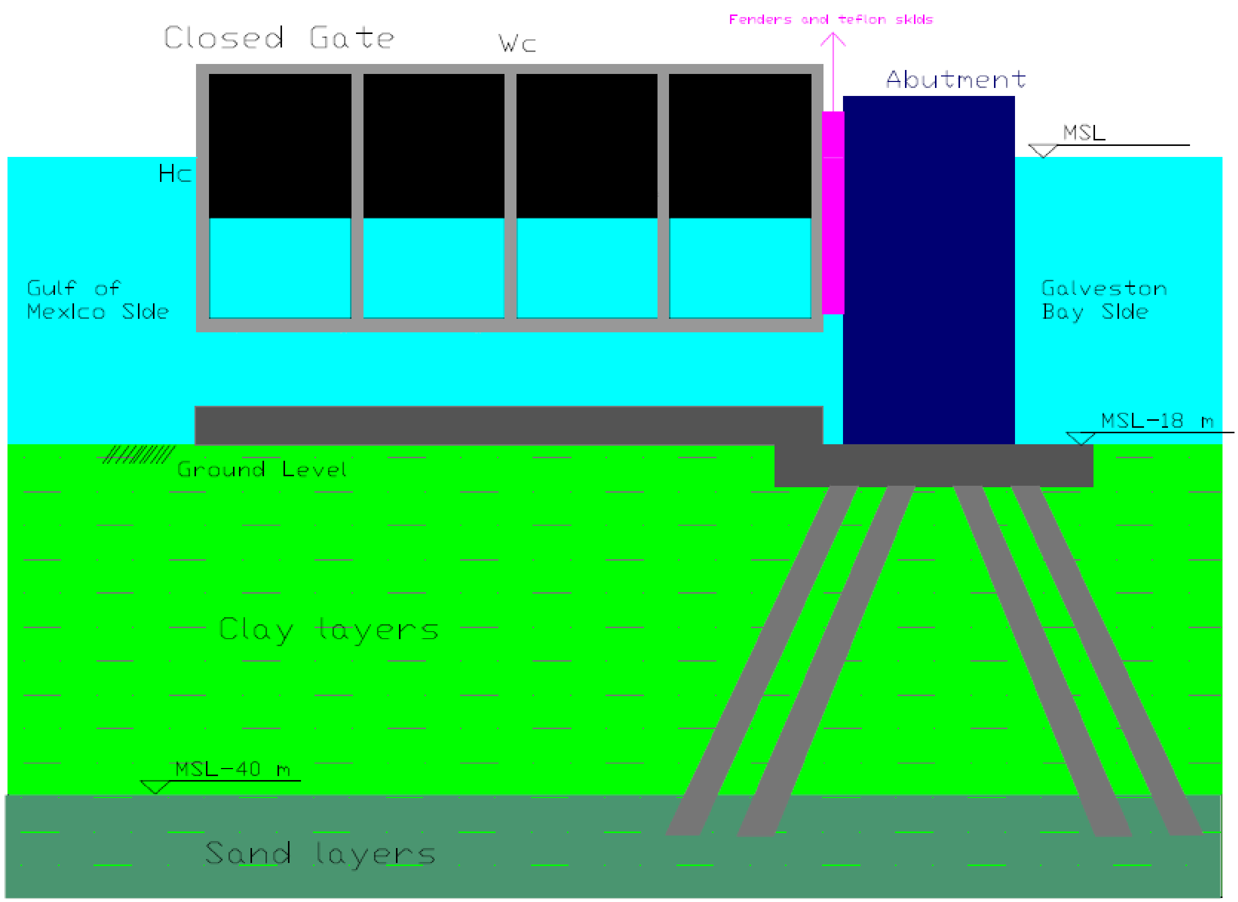


Figure 11-3: Deep foundations cross-section

11.4 Conclusions

In this chapter supporting structures are discussed. The abutments are designed roughly considering the pier with concrete material. Normal reinforcement concrete and prestressed concrete are used where the prestressed concrete offers lighter structure. The foundations are proposed as the deep foundations because of the deep layers of the dense sand. The design of the foundations should be done in the future research.

Up to this chapter the most important elements of the navigational storm surge barrier have been investigated. In the rest of this research some project management aspects are going to be discussed to provide an overview about the realization, maintenance and financial aspects of the project.

12 Project Management Considerations

12.1 Construction and Execution Considerations

12.1.1 Introduction

In this section construction considerations of the navigational barrier of the Bolivar Roads Pass are going to be described. In general, sometimes the construction method dictates the design in headlines [79]. The main decision regarding the construction is choosing between the two methods:

- **Building in the dry:** with big components or even complete subsystems prefabricated elsewhere and brought them into the building location by special equipment or afloat
- **Building in the wet:** building at the location of the project (in situ)

According to the construction guidelines, building in the dry is preferable [80]. In general, the overall tendency is towards more and bigger prefabricated elements. However, in a project mostly the combination of the above mentioned methods can be seen. The important aspect here is that design and construction cannot be considered separately. The construction method in this chapter is proposed according to the design which has been described before. However, it is proposed that later on construction methods would be considered earlier in the final design of the structure.

For construction of the navigational barrier both construction methods mentioned above should be used. The construction method consists of the following main elements:

- Construction of the dry dock
- Construction of the barge gate
- Construction of the abutments and resting chamber
- Construction of the articulation system
- Preparation of the project site
- Construction of the foundations
- Transportation and placement of the prefabricated elements
- Construction of the bed protection

The construction methods for some detail elements such as mooring system, resting chamber, ballasting system and etc. are not considered here. In the following sections more information is provided regarding the construction methods for each of the above mentioned main elements.

12.1.2 Construction of the Dry Dock

For construction of some of the elements, building in the dry is preferred. Indeed, a dry dock is needed prior to the construction of the prefabricated elements can be started. For the location of the dry dock different options are available. The location should be chosen considering different issues such as the distance to the project site and the availability of enough draught for transportation of the barge gate. Also, the location should be close to the highways for supply of the construction materials. It is better if the location of the dry dock provides the opportunity to deposit the excavated soil just next to the dry dock. The possible locations for the construction of the dry dock can be seen in Figure 12-1.

Figure 12-1: Dry dock location options

For construction of the dry dock, firstly the dewatering system should be installed (for example vertical drains system) because in the dry dock the groundwater level should be lowered to beneath the bottom of the dry dock during the construction period of the barge gate. Sheet piles should be used around the dry dock. Then the excavation should be performed. Because of the high weight of the barge gate, large loads would be transferred to the ground. Considering the subsoil strength of the dry dock location, a spread foundation or deep foundations should be constructed.

The dry dock can be separated from the surrounding area by using the impermeable ground layers or another water retaining structure. This water retaining structure completely or partly should be removed when the barge gate is constructed. Then the dry dock is flooded by surrounding water and the barge gate can be transported to the project location. The transportation channel should provide enough draught for the transportation of the barge gate and maybe some dredging work would be needed.

The dry dock should contain the concrete batching plant and enough space for reinforcement bars storage, formworks assembling and storage of other construction materials. Also, the buildings or containers for accommodation of the workers and engineers are necessary. The abutments can also be constructed in the dry dock. Indeed, enough space for all the construction activities should be available in the dry dock.

12.1.3 Construction of the Barge Gate

As it is mentioned before for the barge gate building in the dry dock is the best option. Although transportation costs would be high, this method is preferred due to the large dimensions of the barge gate and the difficulties of the building in the wet. Also, the barge gate is designed in a way that it can be transported in the floating condition by using the tugboats and its own buoyancy.

There are already some examples of the large hydraulic structures which have been built, prefabricated and transported to the location. For example, the floating breakwater of Monaco with the length of 350 meter was built in Spain in a dry dock and towed to Monaco and then positioned at the final location [79]. The cost of transportation for this structure was about one million euro per meter which was significantly expensive. However, due to the short distance between the dry dock and the final position of the barrier in the Bolivar Roads Pass, transportation costs would be considerably cheaper for this project.

For construction of the barge gate, firstly the formwork for the floor slab should be placed. Reinforcement bars of the floor slab then should be placed and fixed. The normal reinforcement or prestressed reinforcement would have different working methods. Also, the vertical starts of reinforcements of the walls should be positioned. Then concrete can be poured in place. Discontinuities in pouring the concrete specially for the floor slab is not appreciated. The quality control of the concrete prior to pouring is necessary and should not be neglected.

When the concrete of the floor slab has enough stiffness, the construction of the walls and compartments can be started. The walls of the caissons have the dimensions of 22.5 m height. This makes casting the walls at once hard. Indeed, the concrete should be casted in shifts by using the climbing formworks (Figure 12-2). Special attention is needed for placement of opening of the walls (and probably in floor if needed). In the construction of the internal walls (compartments) extra attention should be made on the connections of the internal walls, external walls and slabs that they would be watertight.

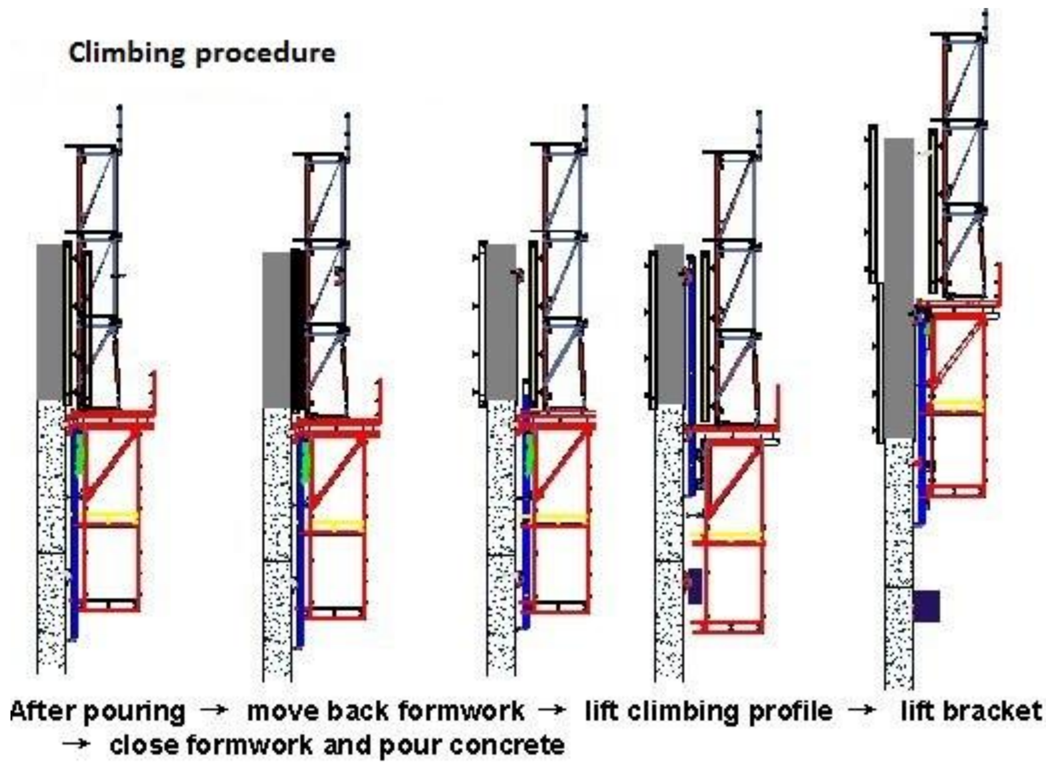


Figure 12-2: Climbing formwork procedure

The roof slab can be constructed afterwards. The detail construction method for the barge gate can be determined later when the final design is clear which is out of the scope of current report.

The construction of additional details such as ballasting system and fenders should also be realized considering the appropriate time and method.

12.1.4 Construction of the Abutments

The abutments can be constructed in the dry dock and then transported to the project location. The construction can be started parallel to the barge gate but it will finish before completion of the barge gate because of the small amount of work which is needed for the abutments compared to the gate.

In the design of the abutments it has been tried to make the total weight of the abutments as low as possible by using the lightweight concrete and also the prestressed reinforcement. Then the normal transportation equipment can be used. If the weight of the abutments is too high then a special transportation equipment should be built parallel to construction of the abutments.

For example, in the Easternscheldt barrier, 65 piers were constructed in the dry docks and brought to their final position by a special designed U-shaped self-propelled hoisting equipment named Ostrea [79] because they were heavy. The smaller elements of this barrier such as sill beams and steel gates were transported by an ordinary floating crane named Taklift 4.

Figure 12-3: Floating crane Ostrea transporting a pier of the Easternschelt barrier [80]

Due to the low weight of the abutments, normal handling equipment can be used for transportation of them from the dry dock to the final position. It is important to make the weight of the abutments as low as possible because special equipment for transportation of only two abutments wouldn't be economical.

12.1.5 Construction of the Articulation System

In the previous sections the ball-joint system or steel swing arms have been proposed as the possible articulation systems at the swing point. The ball-joint system may look like the system of Maeslant barrier. For this barrier the ball-joint system is not a complete sphere but it consists of various segments connected to a core [81]. This system rotates in ten pedestals that are connected to the concrete foundation. Firstly the pedestals were positioned very accurately and then the sphere was placed on them and connected to the gate (Figure 12-4).

The ball-joint system in the current project can also have the same system. The system can be constructed elsewhere and transported to the final position. The system should finally be placed on a foundation and connected to the gate (by help of the wings arms or directly). Final design of the ball-joint system, its foundation and its connection to the barge gate is not treated in this report. The construction method of this system can be determined later when more information about the design of it is available. In case of the steel swing arms, the system can be constructed elsewhere and transported to the project area easily.

Figure 12-4: Ball-joint system of the Maeslant barrier; Left: ball-joint system, Right: pedestals

12.1.6 Preparation of the Project Site

Parallel to the construction of the prefabricated elements, the project site should be prepared. Excavation, dredging and sand dumping in some cases should be realized to prepare the navigational channel and makes the seabed ready for execution of the foundations.

Dredging the navigational channel should provide enough depth for the navigation of ships and also positioning of the barge gate in the closed situation. Thus, in the length and width of minimum 300 m dredging up to MSL-18 m is needed (considering the need for the bottom protection also). When the building site is ready, the construction of the foundations can be started.

12.1.7 Construction of the Foundations

Due to the weak subsoil strength of the Bolivar Roads Pass, it can be expected that the deep foundation should be used for the abutments and the barge gate foundations. The appropriate soil layer for transferring the vertical and horizontal loads by using the pile foundations is located at MSL-40m. Above this layer there are weak clay layers which are not suitable. For the deep foundation, steel tubular piles can be used which is common in the offshore structures where there is a deep water situation. For the durability reasons, the steel tubular piles can be constructed closed-ended and filled with concrete. By this way the piles are less vulnerable for corrosion and more resistance for buckling.

For the construction of the deep foundations, firstly the piles are driven and then the parts of the piles that project above the blinding, such as reinforcement after removal of the pile heads are cast into the concrete floor.

After execution of the deep foundations, the spread (shallow) concrete foundations should be placed on top of them. These spread foundations are used for placement of the abutments and also the barge gate will rest on them in the closed situation. These foundations can be constructed in the wet (in situ).

For the spread foundations, firstly a concrete blinding is laid on the bottom and connected to the deep foundations and then the bottom slab is cast. Then the walls of the foundations can be constructed.

The design of these foundations is out of the scope of current report. When the design of the foundations is done, appropriate construction methods can be chosen for them. So, instead of the in situ construction of the spread foundations maybe prefabricated elements would be more favorable.

12.1.8 Transportation and Placement of the Prefabricated Elements

After preparation of the project site and execution of the foundation, the transportation of the prefabricated elements should be performed. Firstly, the abutments should be transported to the final position. For transportation of the abutments floating cranes can be used. In recent years lifting capacities up to 100.000 kN have been used in the construction of the structures in Denmark and Canada [80]. An example of the floating crane can be seen in Figure 12-5.

Figure 12-5: Floating crane Svanen (Ballast Nedam) with bridge girder for the Westbridge over the Great Belt, Denmark [80]

The placement of the abutments can be done by the guidance of the cranes. The floating crane would be moored to a second vessel or a pontoon to ensure an accurate positioning of the abutments. Also, the deposited sand at the spot of the abutment should be removed before the placement. The positing of the abutments should be performed during the slack tide when the tidal current is minimum. The anchorage of the vessels should be accurately done in order to make the positioning accurate enough. The placement of the piers in Easternscheldt barrier would be a useful guideline for the current project.

Figure 12-6: Placement of the piers in Easternscheldt barrier

The articulation system should be then transported to the final location and placed on its foundation. Floating cranes can be used for the transportation and placement of this system.

When the construction of the barge gate is finished, it will floated to the project location by using the tugboats. In this situation the dry dock is flooded up to the surrounding sea level. Then the gate is floated due to the buoyant forces. The transportation should be done preferably when there is a high water level. The gate is designed in a way that without the water ballast it will have enough draught and stability during the transportation. As it is mentioned in the design stage, sway consideration should be satisfied under the control of the tugboats (Figure 12-7).

When the barge gate arrived in the final position, it can be connected to the articulation system, mooring system and guiding columns under the control of the tugboats and preferably extra pontoons or vessels.

Figure 12-7: An example of the transportation of a caisson by using the tugboats

12.1.9 Construction of the Bed Protection

Bed protection can be constructed after the placement of the foundations and abutments and prior to installation of the barge gate. Compaction of the subsoil before placement of the bed protection might be needed.

Two different bed protection systems have been proposed in the design phase namely armourstone or rip-rap and the box gabions or gabion mattresses. These systems have been considered as interlocked blocks and cabled blockmats. Also, the concrete block mattresses be used. On both sides of the barrier the bed protection should be applied.

The mattresses can be built in a special built factory. The concrete blocks can be attached to the mattress in order to provide the sinking of the mattress as well as stability at the bottom. For the installation of the bed protection a special equipped vessel would be needed. When the main mattresses are placed on the bottom, filter mattresses, rocks and concrete blocks should be placed for the bed protection. The weight of these rocks has been estimated as 1 to 3 tons in the design phase.

12.1.10 Construction and Execution Conclusions

In this section construction methods of the navigational barrier elements are described. For different elements of the barrier specific method should be selected. Construction in the dry or in the wet can be considered while construction in the dry is preferable.

The barge gate and the abutments can be built in the dry dock. The deep foundations and spread foundations can be built in situ. For construction of the articulation system and bed protection special factory is needed.

In this section only the construction method in general and for the main elements are depicted. When the design goes further in more details in later steps of the project, better description and determination of the construction methods can be provided. It is recommended that design and construction would be treated simultaneously in the future research.

12.2 Maintenance Considerations

12.2.1 Introduction

This section deals with the maintenance considerations of the navigational barrier. Maintenance is an important aspect in the life cycle of a structure. For the maintenance purposes inspection is needed which may vary from a rough visual inspection of the general behavior of the structure up to detailed measurements of the specific elements. When the present situation of the structure is not acceptable anymore, different measures can be taken with various costs and effects.

In this section, firstly the general theoretical information about the degradation mechanism is provided. Then a short description of the maintenance strategies and inspection methods are going to be described. Finally, the maintenance activities needed for the barge gate is depicted shortly. For comprehensive description of the theoretical background the reader is referred to the reference [79].

12.2.2 Degradation Mechanisms

In general different degradation mechanisms can be considered for the hydraulic structures [79]:

- Ageing mechanism in general including chloride penetration, corrosion, fatigue, erosion and etc.
- Overloading
- Settlement and deformation
- Damage by external causes
- Influence of human errors

12.2.3 Maintenance and Inspection Strategies

A set of minimum five requirements should be fulfilled during the maintenance process including [79]:

- Information about the present situation or state of the structure
- Goals or a wished situation or minimal a direction to go
- Measures to change the present situation into the desired situation
- Models to check beforehand if these measures will really work
- Capacity to really organize the work in such a way

Generally, following strategies can be considered for the maintenance:

- Pro-active, preventive and corrective maintenance
- Use based maintenance

- Condition based maintenance
- Corrective maintenance
- Risk based inspection and maintenance
- Value driven maintenance

The inspection can be realized in the following forms:

- Visual inspection
- Functional inspection
- Special inspection
- Post-event inspection
- Measurements
- Monitoring

In the next section, the maintenance considerations for the navigational barrier is going to be presented.

12.2.4 Navigational Barrier Maintenance Overview

Due to the importance of the appropriate functionality of the barrier in its life cycle, maintenance activities are very important and vital. All the degradation mechanisms mentioned in the previous sections might happen in the life cycle of the navigational barrier.

The barrier should be inspected by using different methods as follow:

- For the parts which are outside of the water, the visual inspection can be used in specific time intervals.
- For checking the safety aspects and functionality, the barrier can be closed once in a year to control its operational phases and check the safe functionality of the barrier.
- In some cases special inspections are required for example for the barge gate. The barge gate can be disassemble from the system during the calm season (not during the hurricane season), transported to the dry dock and be checked for possible damages.
- In case of happening a hurricane and closing the barrier, the post-event inspection should be performed in after the hurricane condition to check the possible damages to the system during the hurricane. Possible damaged elements should be repaired or replaced in this situation.
- Some measurements and monitoring system are required for example for the articulation system or ballasting system.

In this section only a rough overview of the maintenance aspects is presented. The detailed maintenance instructions should be determined in the future steps of the design.

12.3 Construction and Maintenance Costs

In this section, a rough estimation is presented about the construction cost of the navigational storm surge barrier. Due to the lack of information about the final design of the structure and all its components, only a first estimation about the construction costs can be provided by using some assumptions. A detailed costs estimation needs more in depth design of different elements of the navigational barrier. Table 12-1 shows the total investments costs required for the construction of the navigational storm surge barrier of the Bolivar Roads Pass considering the prestressed concrete barge gate. For the detailed calculations regarding costs see section 14.9.

Total Costs of the navigational barrier	
Component	**Costs (USD $)**
Direct costs	
Barge gate	$85.672.840
Ground preparation	$19.998.000
Abutments	$4.250.400
Foundations	$29.019.343
Other systems	$51.832.068
Total Direct Construction Costs	**$190.772.652**
Indirect costs	
Design, supervision, engineering and administration (10%)	$19.077.265
Overhead (5%)	$9.538.633
Risk (5%)	$9.538.633
Profit (15%)	$34.339.077
Total Indirect Costs	**$72.493.608**
Total Construction Costs	
Total Construction Costs	**$263.266.259**
Total Investment Costs	
Project unforeseens (15%)	$39.489.939
Total Investment Costs (USD $)	**$302.756.198**

Table 12-1: Total costs of the navigational storm surge barrier of the Bolivar Roads Pass

As it can be seen in the table, the total investment cost of the ***prestressed concrete*** navigational barrier is estimated at about ***303 million USD***. If only the cost of the barge gate would be replaced by ***steel*** material instead of concrete then the total cost of the navigational barrier will be about ***538 million USD***. Indeed, the concrete material would be a more economical choice.

A comparison between the current estimation of the costs and the related previous studies of the storm surge barrier for the Bolivar Roads Pass would be useful. Van Breukelen (2013) [53] has estimated the total construction costs of the inflatable barrier for the entire span of the Bolivar Roads Pass at about 1,200 million USD (869 million euros*1.38). It means around 0.48

million USD per meter barrier length. Also, de Vries (2014) [11] has estimated the total investment cost for the environmental barrier of the Bolivar Roads Pass as 1,927 million USD which leads to around 0.85 million USD per meter barrier length (Table 12-2).

The current costs estimation for the navigational barrier will result in a 1.32 million USD per meter length of the navigational barrier in case of the concrete material and 2.33 million USD in case of the steel material. This could be expected because of the complex system of the barge gate and the higher depth in the span of the navigational barrier compared to the entire span of the Bolivar Roads Pass.

Comparison of construction costs for Bolivar Roads Barrier			
Barrier Part	**Barrier Type**	**Million USD /meter barrier**	**Project**
Entire span	Inflatable	0,48	Van Breukelen (2013)
Environemental	Caisson structure	0,85	de Vries (2014)
Navigational (concrete)	Barge gate	1,32	Current research project
Navigational (steel)	Barge gate	2,33	Current research project

Table 12-2: Comparison of construction costs for Bolivar Roads Barrier

In comparison with the estimated unit cost of 40,000 (USD/m^3) for the navigational barrier proposed by van der Toorn (2012) [52], the current calculated unit cost (10,645 USD/m^3) for the concrete barge gate navigational barrier and for the steel barge gate navigational barrier (18,900 USD/m^3) are quite lower. The reason behind this could be the wrong estimation of the unit costs per cubic meters in reference [52] because the dataset used in this reference contains few samples with variation between 20,000 (USD/m^3) to 43,000 (USD/m^3).

However, in general the final costs of the project might be higher than current estimated amount because of the different unit costs in reality or different prices for the cost components.

It should be mentioned that the realized barge gates in the United States have almost the same unit costs with the current project. The bayou Lafourche Saltwater Control Structure costs 13,000 USD/m^3 and Bayou Dularge Floodgate costs 10,000 USD/m^3.

Considering the information regarding the environmental barrier from the other studies, the total construction costs of the Bolivar Roads Storm Surge barrier (including the environmental and navigational barriers) would be between 2.3 billion USD to 4 billion USD.

The cost estimation in this section is on the basis of rough estimations but can provide a first overview about the investment costs of the navigational barrier. The cost estimations in detail can be derived in the future studies when more information about the different components and systems of the barrier is available.

Regarding the ***maintenance costs***, the concrete structure would have maintenance costs as 0.5% to 1% of the initial construction costs per year [74]. In case of the steel structure, this percentage will increase to 1% to 1.5% of the initial construction costs per year. This means that the concrete barge gate not only is cheaper in the initial investment, but also requires less maintenance costs during its life-cycle. In short, concrete barge gate would be a more economical option compared to the steel barge gate.

13 Final Conclusions and Recommendations

13.1 Final Conclusions

In this report the conceptual design of the navigational storm surge barrier of the Bolivar Roads Pass has been considered. The main research question was as below:

"What is the most suitable, realizable, reliable and economical option and its conceptual design for the navigational storm surge barrier in the Bolivar Roads Pass?"

Following conclusions are derived from this research project in response to the question.

Selection of the gate type:

Considering the program of requirements and boundary conditions, different moveable gate options have been considered and evaluated for the navigational barrier. Between all the options, by using the MCA, the barge gate has been selected as the best choice which fulfills the requirements of the project. Barge gate is suitable for the wide openings, provide unlimited air draft, has reasonable construction and maintenance costs and in the floating situation doesn't transfer too much loads to the foundations. (See chapter 5 for more information)

Conceptual design of the barrier:

The system of the barrier has been defined. The barrier is selected as the floating type. This is chosen because then the minimum loads would be transferred to the weak subsoil of the structure. The barrier in the open situation provides the navigation without hindrance and in the closed situation protects the Galveston Bay against the storm surges. There is also 1 m opening under the gate in the closed situation.

The HPLC has been selected as the appropriate material for design of the barge gate. This material is environmentally friendly, durable and a reliable choice for the construction of a complex system such as the current barge gate. Permeability, strength and the light weight are the other advantages of the HPLC while it costs more than normal concrete and has a complex mix design procedure.

Also, it has been decided that the barrier should be designed initially for retaining the full surge (MSL+5.5 m). Different operational phases of the barrier have been discussed and the situations of the gate in each phase have been depicted to provide enough overview for the design of the barrier. (See chapter 6 for more information)

Geometry design of the gate:

Barge gate has been designed as a caisson considering the structural and stability requirements. The design process is an iterative process and the final design should fulfill all the operational phases of the barrier. The design considered the hydrostatic loads mainly. For this situation the normative case is when the barrier is closed and there is a hurricane condition which provides the highest positive head on the barrier. The dimensions of the barrier are derived as 230 m length, 36 m width and 22.5 height. The initial draught of the structure is equal to 8.02 m. The weight of the gate is 70,779 tons. (See chapter 7 for more information)

Complementary structures design:

The initial designs of the complementary structures have been done. Bed protection is needed under the gate because of the high flow velocity and the large scour depth. The design shows that the rock material with 1 to 3 tons is required with the approximate length of 70 m from each side of the gate and the filter layers as supporting system.

Also, gate berthing system has been designed using the wheel fenders. This fender type absorbs the berthing energy of the gate when it wants to rest on the abutments during the closure and provide free vertical movement of the gate during immersion at final location.

Articulation system (swing point) has been proposed to be a ball-joint system or the steel swing arms. On the basis of the system design of the barrier, the articulation system should provide free degrees of freedom in all the directions except surge and sway. Considering the reference projects, the above mentioned systems are the best options. The choice between these two systems should be made in the future. (See chapter 8 for more information)

Design evaluations and risk analysis:

The design evaluations have determined the key design parameters of the project. Considering these key parameters would be useful for the design iterations. Construction material, layout and location (within the Bolivar Roads Pass), prestressed concrete or normal reinforced concrete design method, retaining height and opening under the gate are the most important design parameters which affect the final design aspects significantly.

Also, risk analysis has been depicted which shows various risks in different phases of the project. Risk register matrices and fault trees for the barge gate are the outcomes of the risk analysis. The most important issues here are hydrodynamic analysis, articulation system design and operation and stability against the negative head. (See chapter 9 for more information)

Design revisions:

Some design revisions have been performed in the process of design iterations. The comparison has been made between the steel barge gate and the concrete barge gate. The steel barge gate has lower weight (31,715 tons) compared to the concrete barge gate but it is more expensive. Also, the maintenance of the concrete gate costs less and it is more durable and environmental friendly.

The barrier has been redesigned for less retaining height (MSL+1.5 m). The results of the analysis show that the barge gate obviously has lower weight compared to the full retaining height which reduces the costs of the project. The barge gate in this situation with the dimensions of 230 m* 36 m* 18.5 m has the weight of 63,889 tons.

Design with prestressed concrete instead of the normal reinforced concrete has been realized for the full retaining height. The weight of the structure is reduced to 63,724 tons in this case but the structure costs more because of the higher prices of the prestressed concrete. However, the prestressed concrete has the advantages of higher strength and better performance in terms of cracks.

Stability against the negative head considering the prestressed concrete barge gate has been analyzed. The calculations show that the gate is stable against negative head up to MSL+2.8 m sea level in the Galveston Bay side which is due to the high weight of the barge gate and ballasting water inside it. A civil structure or a mechanical lock can be realized to resist against the negative head. However, other measures for reducing the negative head amount on the barrier are opening the environmental barrier or implementing openings and valves in the barge gate.

In addition, initial hydrodynamic analyses have been performed. The loads on the articulation system have been roughly estimated and important motions of the barge gate during its closure (roll and heave motions) have been described. Also, the results of the vertical vibration analysis of the barge gate in the closed situation show that the gate is positively damped. Also, The excitation frequency (f=0.74) is significantly higher compared to the 1st eigenfrequency (f_n=0.12) with the factor of more than 3 as it is advised (factor 6). Indeed, the resonance wouldn't occur for the first eigenfrequency mode of the structure due to the underflow excitations. (See chapter 10 for more information)

Supporting structures:

The abutments have been designed with the normal reinforcement and prestressed concrete. The prestressed concrete abutment with the dimensions of 24 m* 7 m* 5 m has been found as the best option. Also, due to the weak subsoil of the project site, deep foundations in the format of steel tubular piles filled with concrete have been proposed as the suitable foundation type for the structure. (See chapter 11 for more information)

Project management considerations:

Construction methods have been discussed extensively. The barge gate and abutments can be constructed in the dry dock and transported to the location. The barge gate can be transported afloat and the abutments with the transportation vessels. Also, the shallow foundations can be built in the wet.

Maintenance aspects have been discussed and it is proposed that the barge gate can be disassemble from the system during the calm season (not during the hurricane season), transported to the dry dock and be checked for possible damages.

Construction costs have been roughly estimated. It is expected that the navigational barrier with the concrete barge gate costs 303 million USD while with the steel gate costs 538 million USD. Also, the maintenance costs of the concrete option is cheaper which makes the concrete barge gate a more economical option compared to the steel gate. Also, the total cost of the barrier of the Bolivar Roads Pass (including the environmental barrier and the navigational barrier) is estimated in the range of 2.3 billion USD to 4 billion USD (See chapter 12 for more information).

13.2 Recommendations

This section provides the recommendations regarding the missing design tasks and analyses in this research project for the navigational barrier and also the possible opportunities for the improvement and optimizations in the future studies. The following issues can be mentioned in this regard:

- **Design storm** and **required safety level** should be investigated in more details and by using the probabilistic design approaches. Cost-benefit analysis for determining the appropriate safety level would be useful.
- **Laboratory models and tests** should be realized to find out more about the barrier system and operational phases in reality. Motions of the gate can be checked in this way.
- In the current design mainly the hydrostatic loads have been used in the design of the barge gate. An extensive design by considering the **hydrodynamic loads** is recommended.
- A **hydrodynamic analysis** is recommended for the loads and motions of the structure during different operational phases and particularly in the closing and immersing phase of the barrier. The effects of the gate motions on the articulation system should be also investigated more.
- Design of the gate in this report with the concrete material has been done roughly. Dimensions of the elements and required reinforcement are derived as the first estimation. **Optimizations** regarding the dimensions of the caissons elements, internal walls and determining the map of reinforcement should be done in the future studies.
- There are some **other optimizations options** such as implementing openings in the walls of the barge gate to reduce the forces acting on the barrier (Figure 13-1), implementing bottom recess in the gate to reduce the underflow speed and the suction forces on the floor slab of the gate (Figure 13-2), implementing the corner recesses in the walls of the barge gate to reduce the hydrostatic loads on the structure (Figure 13-3).

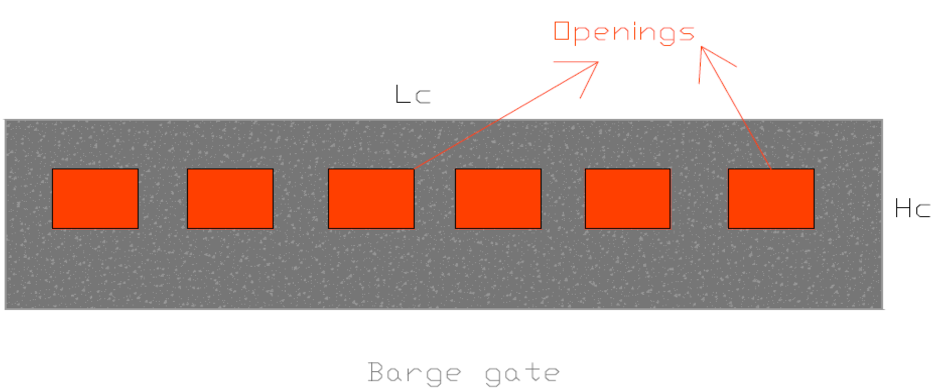


Figure 13-1: Implementing openings in the barge gate, Front view of the barge gate

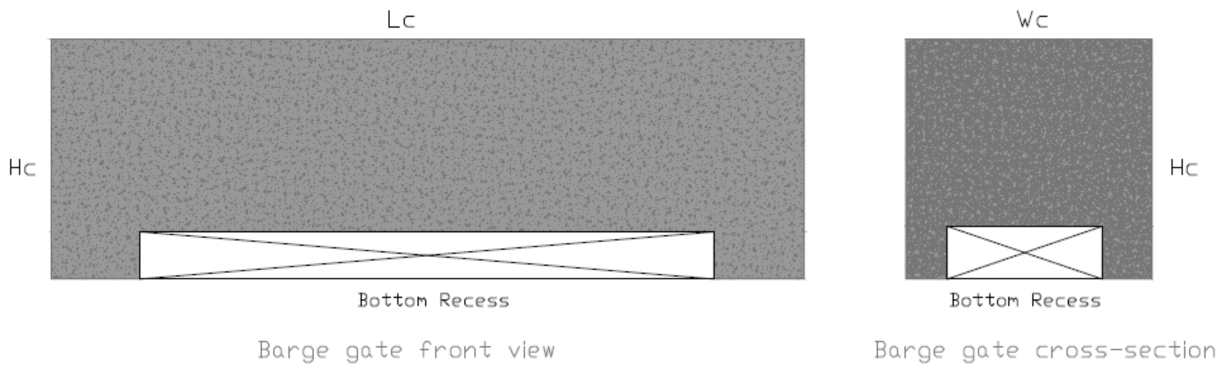


Figure 13-2: Implementing bottom recess in the barge gate

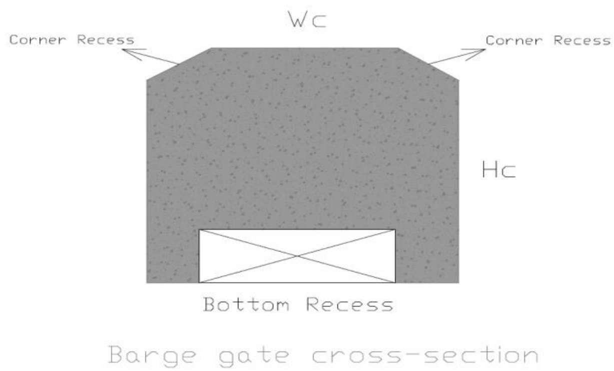


Figure 13-3: Implementing corner recesses in the barge gate

- **Optimizations** regarding the configuration and shape of the internal walls are recommended.
- The **complementary structures and supporting structures** have been initially designed and introduced in this report. Further analyses are needed for the final design of these structures. The foundations are not designed in this report and should be designed in the future studies.
- The **articulation system** (swing point) has been discussed shortly in this report and the conceptual design for it has been proposed. This system should be further analyzed and the best option should be chosen and designed in more details.
- The **cost estimations** are derived roughly because of the lack of information regarding different elements of the navigational barrier. The financial analysis is necessary in the future to find out about the life-cycle costs of the project.

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14 Appendices

14.1 Gate Design Calculations

	General Parameters		
H_1	22,5	m	Water level upstream
H_2	15,5	m	Water level downstream
γ_w	10,25	kN/m^3	
concrete type	B65		
γ_c	17	kN/m^3	
f_{ck}	67	N/mm^2	Characteristic compressive cube strength
γ_{cm}	1,2		Material factor compression
$f_{ctk0,05}$	3	N/mm^2	Characteristic axial tensile strength of concrete
γ_{tm}	1,4		Material factor tension
F_{cd}	55,8	N/mm^2	Design value of concrete compressive strength
F_{ctd}	2,1	N/mm^2	Design value of concrete tensile strength

	Gate (Caisson) Dimensions		
H_c	22,5	m	Height
B_c	36	m	Width
L_c	230	m	Length
t_w	1,1	m	Thickness of the wall
t_f	1,4	m	Thickness of the floor
t_r	0,5	m	Thickness of the roof
n_x	5		Number of compartments in x direction, width direction
n_y	22		Number of compartments in y direction, length direction
t_{iw}	0,3	m	Thickness of the internal walls (compartments)

14.1.1 Results of Calculations Design check 1: Floating Caisson

Initial Draught Calculations				
$W_1 =$	3.167.100	kN	+	The whole section weight
$W_2 =$	2.696.414	kN	-	Empty section weight
$W_3 =$	119.663	kN	+	Compartments in x direction weight
$W_4 =$	78.123	kN	+	Compartments in y direction weight
$W_c =$	668.472	kN	Weight of the caisson	
$d_i =$	7,88	m	Initial Draught	

	Hydrostatic forces							
	P_{wwall} =	$d_i * \gamma_w$	=	80,73	kN/m^2	Hydrostatic pressure		
	F_{wwall} =	$0.5 * P_1 * d_i$	=	317,94	kN/m	Hydrostatic force		
	P_{wfloor} =	80,73	kN/m^2	Rectangular water pressure to the floor				
	F_{wfloor} =	2.906,40	kN/m	Force on the floor				

	Shear Forces Calculations with hydrostatic loads							
	$V_{wall,max}$ =	317,94	kN/m	Max shear force on wall				
	$V_{floor,max}$ =	Weight of side wall + Half of the weight of the roof - half of the Max floor force						
		(weight of the floor is not considered which is conservative)						
	W_w =	420,75	kN/m	Weight of the wall				
	W_{roof} =	306	kN/m	Weight of the roof				
	W_{floor} =	856,8	kN/m	Weight of the floor				
	$V_{floor,max}$ =	879,45	kN/m	Max shear force floor				
	$V_{floor,max}$ =	451,05	kN/m	Shear force by considering the weight of the floor itself				
	$V_{roof,max}$ =	153	kN/m					

	Bending Moments Calculations with hydrostatic loads							
	Wall							
	$M_{max, wall}$ =	835	kNm/m	Max bending moment on wall				
	Floor							
	$M_{floor, wall side}$ =	835	kNm/m					
	$M_{floor, center}$ =	8.388	kNm/m					
	$M_{max, floor}$ =	8.388	kNm/m	MAX bending moment of floor slab				
	Roof							
	$M_{max, roof}$ =	1.377	kNm/m	MAX bending moment of roof slab				

Structural Design checks for hydrostatic loads

Shear stress criteria

τ_{max} =	$0{,}4 * f_{ctd} + 0.15\, f_{cd}$	N/mm^2	Max allowed shear stress
τ_{max} =	9,23	N/mm^2	
τ =	$(3/2) * (V/(bt))$	N/mm^2	Occuring shear stress
τ_w =	0,482	N/mm^2	
τ_f =	1,05	N/mm^3	
τ_r =	0,51	N/mm^4	
τ_{max} / τ_w =	19,16	should be > 1	
τ_{max} / τ_f =	8,82	should be > 1	
τ_{max} / τ_r =	18,10	should be > 1	

Moment criteria

x =	$M / (bd^2 f_{cd})$		
c =	0,04	m	Concrete cover
ϕ =	32	mm	Reinforcement steel
d_w =	1,028	m	Effective height of the cross section of wall
d_f =	1,328	m	Effective height of the cross section of floor
d_r =	0,428	m	Effective height of the cross section of roof
x_w =	14		
x_f =	85		
x_r =	135		
x_w =	0,05	should be < 1	x is divided by 300 almost the maximum alowable reinforcement
x_f =	0,28	should be < 1	
x_r =	0,45	should be < 1	

On the basis of the values for x, the reinforcement percentage should be chosen from the table

ρ_w =	0,3	%	A_{sw} =	3.084	mm^2/m
ρ_f =	1,1	%	A_{sf} =	14.608	mm^2/m
ρ_r =	1,75	%	A_{sr} =	7.490	mm^2/m
f_{yd} =	434,78	N/mm^2	B500B reinforcement		
M_{uw} =	1.241	kNm/m			
M_{uf} =	8.435	kNm/m			
M_{ur} =	1.394	kNm/m			
M_{uw}/M_{ed} =	1,49				
M_{uf}/M_{ed} =	1,01				
M_{ur}/M_{ed} =	1,01				
ρ_w/ρ_{max} =	0,08				
ρ_f/ρ_{max} =	0,30				
ρ_r/ρ_{max} =	0,48				

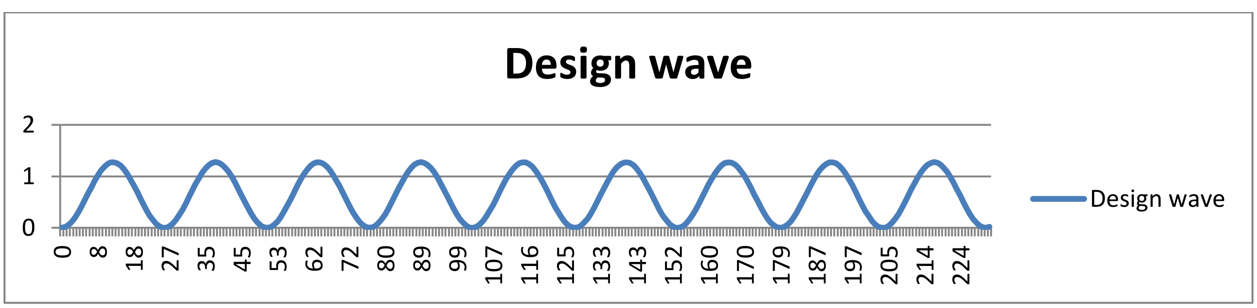
Longitudinal wave loads

The transportation should be done in the normal condition.

L_w =	$gT_{reg}^2/2\pi$	m	Wave Length
T_{reg} =	4	s	Peak wave period normal conditions
L_w =	25,48	m	Wave Length normal conditions
H_w =	1,27	m	Wave Height normal conditions (assumed as L/20)

$$x = \frac{L}{2\pi}\theta - \frac{L}{40}\sin\theta$$

$$z = \frac{L}{40}(1-\cos\theta)$$



q_w =	$d_i * \gamma_w$ =	82,20	kN/m^2		
a =	$L_w/2$ =	12,74	m	Halft of the wave length	Simplified for Sinus shape
b =	$H_w * \gamma_w$ =	13,02	kN/m^2	Max water level difference	
x =	a/3,14 =	4,06	m	Point of force component	
F =	a*b / 3,14	52,81	kN/m	Max force beacuse of the wave	
V_{max} =	52,81	kN/m	Max shear force		
M_{max} =	2 * F * x =	428,51	kNm/m	Max bending moment	
The shear force and bending moment in this case are not critical because of the small length of the wave in normal condition. Indeed, they are not normative in					

Static Stability Calculations

Centre of Gravity Position

Ve_1 =	2.095.875		+	The whole section
Ve_2 =	1.855.767		-	Empty section
Ve_3 =	82.357		+	Compartments in x direction
V_1 =	186.300		+	The whole section
V_2 =	158.613		-	Empty section
V_3 =	7.039		+	Compartments in x direction
KG =	9,29	m	Above the underside	

Draught of the Caisson

d_c =	7,88	m	Draught
KB =	3,94	m	Centre of Buoyancy

Moment of Inertia

I_{yy} =	894.240	m^4

Volume of displaced Fluid

V_{disw} =	65.217	m^3

	KB calculation						
	BM =	13,71	m				
	Metacentric Height Calculations						
	h_m = GM=	KB+BM-KG					
	GM =	8,36	m	Should be > 0.5 m			

Dynamic Stability Calculations								
	Sway Considerations							
	L_w =	$gT_{reg}^2/2\pi$	m	Wave Length				
	T_{reg} =	4	s	Peak wave period regular circumstances				
	L_w =	25,48	m					
	$L_w/0,7L_c$ =	0,16		should be < 1				
	$L_w/0,7B_c$ =	1,01		should be < 1				

The sway consideration in the width direction might be problematic because the criteria is almost equal to 1. By considering extra measures in the transportation phase there would be no problem in this regard.

	Natural Oscillation							
	I_{xx1} =	87.480	m^4	+	The whole section			
	I_{xx2} =	66.288	m^4	-	Empty section			
	I_{xx3} =	0	m^4	+	Compartments in x direction (too small)			
	I_{xx} =	21.192	m^4	Moment of inertia xx				
	I_{zz1} =	34.172	m^4	+	The whole section			
	I_{zz2} =	24.623	m^4	-	Empty section			
	I_{zz3} =	1.093	m^4	+	Compartments in x direction (too small)			
	I_{zz} =	10.642	m^4	Moment of inertia zz				

	I_{polar} =	$I_{xx} + I_{zz}$	m^4	
	I_{polar} =	31.834	m^4	Polar Inertia
	A =	144,62	m^2	The area of the concrete in a vertical cross section
	j =	(I_{polar} / A)^0,5		Polar inertia radius
	j =	14,84	m	
	T_0 =	$2\pi j / ((h_m g)$^0,5)		Natural ocillation period of the floating caisson
	h_m =	8,36	m	From Static stability sheet
	T_0 =	10,19	s	
	T_{reg} =	4	s	Peak wave period regular circumstances
	T_0 / T_{reg} =	2,55		should be > 2

14.1.2 Results of Calculations Design Check 2: Gate During the Immersion at Final Location

Desired draught			
D_{cdes} =	MSL - 17 m		Desired caisson draught
D_{cdes} =	17	m	
W_{cdes} =	1.442.790	kN	Desired total weight of caisson and ballast
Wc =	668.472	kN	Weight of the caisson
$W_{wballast}$ =	774.318	kN	Weight of the ballast water needed
$h_{wballast}$ =	10,59	m	Height of the ballast water needed in the caisson in each compartment
$Ve\gamma_{concrete}$	5.481.893		
$Ve\gamma_{ballast}$	5.593.542		
$V\gamma_{concrete}$	590.349		
$V\gamma_{ballast}$	835.611		
KG =	7,77		
KB =	8,5	m	
I_{yy} =	864.919	m^4	
V_{disw} =	140.760	m^3	
BM =	6,14	m	
h_m = GM=	KB+BM-KG		
h_m = GM=	6,88	m	Should be > 0.5 m

The stability checks have been done for different ballasting situations and draughts. The gate is stable in all the cases.

14.1.3 Results of Calculations Design Check 3: Gate immersed at Final Location During Normal Conditions

The gate is going to be designed for the vertical supports on the foundations.

Forces calculations			
P_{max} =	50	kN/m^2	
L=	220	m	
t =	22,5	m	
d =	20.600	mm	
$V_{max, wall}$ =	5.500	kN/m	
$M_{max, wall}$ =	302.500	kNm/m	

Bending moment Reinforcement			
$A_{s,req}$ =	M_{max} / $0,9f_{yd}d$		
$A_{s,req}$ =	37.527	mm^2/m	
A_s =	$\rho bd*10^4$	mm^2	
Bars assumed as ϕ32			
A_{bar} =	803,84	mm^2	
N_{rei} =	47	Number of required bars	
hth =	100	mm	Heart to heart of the bars
Rows =	4,7		Number of rows of the bars in each meter
ρ =	2,83	%	Should be 0,27 < < 3,63
ρ/ρ_{max} =	0,78	should be < 1	
Final check			
f_{yd} =	434,78	N/mm^2	B500B reinforcement
M_u =	338.382		
M_{ed} =	302.500		
M_u/M_{ed} =	1,12	should be > 1	

	Shear Reinforcement		
	$\cot \theta$ =	2,5	
	z =	1195,2	mm
	s =	200	mm
	f_{ywd} =	434,78	N/mm^2
	$A_{sw, req}$ =	847	mm^2/m
	Stirrups assumed as ϕ16		
	A_{bar} =	200,96	mm^2
	N_{rei} =	5	Number of required bars

14.1.4 Results of Calculations Design Check 4: Gate Immersed at Final Location During Hurricane Conditions

	Hydrostatic force Upstream				
	P_1 =	$H_1 * \gamma_w$	=	230,63	kN/m^2
	F_1 =	$0.5 * P_1 * H_1$	=	*2.594,53*	kN/m
	Hydrostatic force Downstream				
	P_2 =	$H_2 * \gamma_w$	=	158,875	kN/m^2
	F_2 =	$0.5 * P_2 * H_2$	=	*1231,2813*	kN/m

	Wave Load					
	P_{wave} =	$\gamma_w * H_i * [\{Cosh(k(d+z)\}/Cosh(kd)]$		for	$-d < z < 0$	
	P_{wave} =	$[1-(z/H_i)] * \gamma_w * H_i$		for	$0 < z < H_i$	
	The wave load from -d < z < 0 is important for us because the highest level of the gate is up to MSL + 5.5 m					
	H_i =	5,4	m			
	d =	23,5	m			
	T =	7,9	s			
	L =	$gT^2 / 2\pi$	=	99,38	m	wave Length during the hurricane
	K =	$2\pi / L$	=	0,063		wave Number during the hurricane
	$P_{wave\ top}$ =	55,35	kN/m^2	for	z = 0 m	
	$P_{wave\ bottom}$ =	23,90	kN/m^3	for	z = -22,5 m	
	F_{wave} =	$0,5 * (P_{wave\ top} + P_{wave\ bottom}) * H_1$				
	F_{wave} =	891,53	kN/m			
	*The wave load is almost 35% of the hydrostatic load					

	Desired draught calculations					
d_{dis} =	17	m				
W_{dis} =	1.442.790					
W_c =	668.472	kN				
$W_{ballast}$ =	774.318	kN				
$h_{ballast}$ =	10,59	m	Initial ballast normal condition for MSL-17m			
$P_{wall, ballast}$ =	108,53	kN/m^2	Ballasting pressure to the wall			
$F_{wall, ballast}$ =	379,19	kN/m	Max Ballasting load to the wall			
$P_{floor, ballast}$ =	108,53	kN/m^2	Ballasting pressure to the floor			
For having the security, more water is used for ballast, then the structure can rest on its supports.						
$h_{ballast, des}$ =	15,00	m	Ballasting the gate with more water			
$P_{floor, ballast}$ =	153,75	kN/m^2	Ballasting pressure to the floor			
$F_{wall, ballast}$ =	1670,625	kN/m	Max Ballasting load to the wall			
P_s =	71,75	kN/m^2	Suction force during the hurricane (conservative)			
Total pressure on bottom of the structure downstream =		P_2 =		158,88	kN/m2	
Total pressure on bottom of the structure upstream =		$P_{wave\ bottom} + P_1$ =		254,52	kN/m2	
$P_{ver, sum}$ =	39,20	kN/m^2	Sum of vertical pressure (downward +) including weight of the structure (*1.2 safety)			

Suction force:

In the critical situation, when the water level is the highest in the Gulf side and the lowest in the Bay side, the highest discharge and consequently flow velocity is occurred. In this condition:

$\Delta H = 23.5 - 16.5 = 7$ m

Considering C_s equal to 1, this suction pressure (P_s) for the critical condition is equal to 71.75 kN/m^2.

	Shear Forces Calculations						
	Gulf side wall						
	$V_{bottom, gulf}$ =	3106,87	kN/m	Max shear force on Gulf side wall (conservative with minimum ballast)			
	Bay Side wall						
	$V_{bottom, bay}$ =	852,09	kN/m	Max shear force on Bay side wall			
	Floor						
	V_{floor} =	Weight of side wall + Half of the weight of the roof + half weight of the floor + half weight of the ballast - water pressure from bottom					
	W_w =	421	kN/m	-			
	W_r =	306	kN/m	-			
	W_f =	857	kN/m	-			
	$F_{ballast, floor}$ =	5.535	kN/m	-	force of ballast		
	$F_{suction, floor}$ =	2.583	kN/m	-	force of suction		
	$F_{w, floor}$ =	7.441	kN/m	+	force of water pressure from bottom of the caisson		
	V_{floor} =	1.089	kN/m	Conservative value			
	Roof						
	V_{roof} =	153	kN/m				

	Bending Moments Calculations						
	Gulf side wall						
	$M_{b, gulfwall}$ =	9.805	kNm/m	Max bending moment on Gulf side wall (conservative with minimum ballast)			
	Floor						
	$M_{floor, wall side}$ =	9.805	kNm/m				
	$M_{floor, center}$ =	6.351	kNm/m				
	$M_{floor, max}$ =	3.454	kNm/m	Max bending moment on floor slab			
	Roof						
	$M_{middle, roof}$ =	1.377	kNm/m	Max bending moment on roof slab			

Structural Design checks				
Shear stress criteria				
τ_{max} =	$0{,}4 * f_{ctd} + 0.15\, f_{cd}$		N/mm^2	Max allowed shear stress
τ_{max} =	9,23		N/mm^2	
τ =	(3/2) * (V/(bt))		N/mm^2	Occuring shear stress
τ_w =	4,707		N/mm^2	
τ_f =	1,17		N/mm^3	
τ_r =	0,459		N/mm^4	
τ_{max} / τ_w =	1,96	should be > 1		
τ_{max} / τ_f =	7,92	should be > 1		
τ_{max} / τ_r =	20,11	should be > 1		
Moment criteria				
x =	$M / (bd^2 f_{cd})$			Page 111, caisson design Manual, reinforcement percentage table
c =	0,04	m	Concrete cover	
ϕ =	32	mm	reinforcement steel	
d_w =	1,028	m	effective height of the cross section	
d_f =	1,328	m		
d_r =	0,428	m		
x_w =	166			
x_f =	35			
x_r =	135			
x_w =	0,54	x is divided by 300 almost the maximum alowable reinforcement and it should be less than 1		
x_f =	0,11			
x_r =	0,43			
On the basis of the values for x, the reinforcement percentage should be chosen from the table				

	Moment Strength check wall						
	b =	1000	mm				
	h = t_w =	1100	mm				
	d =	1028	mm				
	M_{Ed} =	9.805	kNm				
	ρ =	3	%	Should be 0,27 < < 3,63			
	f_{cd} =	55,8	N/mm^2				
	f_{yd} =	434,78	N/mm^2	B500B reinforcement			
	A_s =	30.840	mm^2/m				
	k =	7,79					
	ρ.k =	0,23					
	M_u =	12.110	kNm				
	M_u -M_{ed} =	2.305	should be > 0				
	M_u /M_{ed} =	1,24	should be > 1				
	Bars assumed as ϕ32						
	A_{bar} =	803,84	mm^2				
	N_{rei} =	39	Number of required bars /m				
	hth =	100	mm	Heart to heart of the bars			
	Rows =	3,9		Number of rows of the bars in each meter			
	ρ =	3,00	%	Should be 0,27 < < 3,63			
	ρ/$ρ_{max}$ =	0,83	should be < 1				

	Moment Strength check floor						
	b =	1000	mm				
	h = t_f =	1400	mm				
	d =	1328	mm				
	M_{Ed} =	3.454	kNm				
	ρ =	1,31	%	Should be 0,27 < < 3,63			
	f_{cd} =	55,8	N/mm^2				
	f_{yd} =	434,78	N/mm^2	B500B reinforcement (page 181 manual)			
	A_s =	17.397	mm^2/m				
	k =	7,79					
	ρ.k =	0,10					
	M_u =	9.512	kNm				

$M_u - M_{ed}$ =	6.058	should be > 0				
M_u / M_{ed} =	2,75	should be > 1				
Bars assumed as ϕ32						
A_{bar} =	803,84	mm^2				
N_{rei} =	22	Number of required bars /m				
hth =	100	mm	Heart to heart of the bars			
Rows =	2,2		Number of rows of the bars in each meter			
ρ =	1,31	%	Should be 0,27 << 3,63			
ρ/ρ_{max} =	0,36	should be < 1				

Shear Strength check wall						
d =	1028	mm				
$C_{RD,c}$ =	0,12					
k =	1,44					
ρ_1 =	0,02					
f_{ck} =	67					
b_w =	1000					
$V_{Rd,c}$ =	910	kN	MAX shear capacity			
$V_{max, wall}$ =	3.119	kN	upstream			
$V_{max, wall}$ =	864	kN	Downstream			
cot θ =	2,5					
z =	925,2	mm				
s =	200	mm				
f_{ywd} =	434,78					
A_{sw} =	620,23	mm^2	=	4	ϕ 16	

Moment Strength check Roof

b =	1000	mm	
$h = t_r$ =	500	mm	
d =	428	mm	
M_{Ed} =	1.377	kNm	
ρ =	1,9	%	Should be 0,27 < < 3,63
f_{cd} =	55,8	N/mm^2	
f_{yd} =	434,78	N/mm^2	B500B reinforcement (page 181 manual)
A_s =	8.132	mm^2/m	
k =	7,79		
$\rho.k$ =	0,15		
M_u =	1.397	kNm	
$M_u - M_{ed}$ =	20	kNm	should be > 0
M_u / M_{ed} =	1,01		should be > 1
Bars assumed as ϕ32			
A_{bar} =	803,84	mm^2	
N_{rei} =	11	Number of required bars /m	
hth =	100	mm	Heart to heart of the bars
Rows =	1,1		Number of rows of the bars in each meter
ρ =	1,90	%	Should be 0,27 < < 3,63
ρ/ρ_{max} =	0,52	should be < 1	

Shear Strength check wall

d =	1028	mm	
$C_{RD,c}$ =	0,12		
k =	1,44		
ρ_1 =	0,02		
f_{ck} =	67		
b_w =	1000		
$V_{Rd,c}$ =	910	kN	MAX shear capacity
$V_{max,\ wall}$ =	3.107	kN	upstream
$V_{max,\ wall}$ =	852	kN	Downstream

	cot θ =	2,5					
	z =	925,2	mm				
	s =	200	mm				
	f_{ywd} =	434,78					
	A_{sw} =	617,88	mm^2	=	4	ϕ 16	

	Shear Strength check Floor						
	d =	1328	mm				
	$C_{RD,c}$ =	0,12					
	k =	1,39					
	ρ_1 =	0,02					
	f_{ck} =	67					
	b_w =	1000					
	$V_{Rd,c}$ =	1.132	kN	MAX shear capacity			
	$V_{max,\ floor}$ =	1.089	kN	its ok			
	cot θ =	2,5					
	z =	1195,2	mm				
	s =	200	mm				
	f_{ywd} =	434,78					
	A_{sw} =	167,57	mm^2	=	1	ϕ 16	

	Shear Strength check Roof						
	d =	428	mm				
	$C_{RD,c}$ =	0,12					
	k =	1,68					
	ρ_1 =	0,02					
	f_{ck} =	67					
	b_w =	1000					
	$V_{Rd,c}$ =	442	kN	MAX shear capacity			
	$V_{max,\ roof}$ =	153	kN	its ok			

	Shear Reinforcement						
	$A_{sw, req}$ =	1.727	mm^2				
	$A_{sw, add}$ =	1.106	mm^2	=	6	ϕ 16	

Forces Calculationson horizontal supports

Critical Load is equal to highest pressure of the upstream including hydrostatic pressure and wave pressure minus hydrostatic pressure of the downstream. This load occurs only in parts of the gate. For the initial design of the gate the distributed load for all the section is considered which is less than the critical load.

q=	78,93	kN/m2	Uniform distributed load on the gate = ΔH * γ_w * 1,1 (safety factor)
L =	220	m	Length of the beam on supports
t =	36	m	Height of the beam on supports = B_c
d =	33.800	mm	
$V_{max, wall}$ =	8.682	kN/m	MAX shear force
$M_{max, wall}$ =	477.496	kNm/m	Max bending moment
$A_{s,req}$ =	M_{max} / $0,9f_{yd}d$		Required reinforcement
$A_{s,req}$ =	36.103	mm^2	
A_s =	$\rho bd*10^4$	mm^2	
Bars assumed as ϕ40			
A_{bar} =	1256	mm^2	
N_{rei} =	29	Number of required bars	
hth =	100	mm	Heart to heart of the bars
Rows =	2,9		Number of rows of the bars in each meter
ρ =	3,51	%	Should be 0,27 < < 3,63
ρ/ρ_{max} =	0,97	should be < 1	

Final check							
f_{yd} =	434,78	N/mm^2	B500B reinforcement				
M_u =	535.274						
M_{ed} =	477.496						
M_u/M_{ed} =	1,12	should be > 1					
Shear Reinforcement							
$A_{sw, req}$ =	1.727	mm^2					
$A_{sw, add}$ =	1.109	mm^2	=	6	ϕ 16		

Vertical support hurricane condition

Forces calculations							
P_{max} =	60	kN/m^2					
L=	220	m					
t =	22,5	m					
d =	20.600	mm					
$V_{max, wall}$ =	6.600	kN/m					
$M_{max, wall}$ =	363.000	kNm/m					
Bending moment Reinforcement							
$A_{s,req}$ =	$M_{max}/0{,}9f_{yd}d$						
$A_{s,req}$ =	45.032	mm^2/m					
A_s =	$\rho bd*10^4$	mm^2					
Bars assumed as ϕ32							
A_{bar} =	803,84	mm^2					
N_{rei} =	57	Number of required bars					
hth =	100	mm	Heart to heart of the bars				
Rows =	5,7		Number of rows of the bars in each meter				
ρ =	3,39	%	Should be 0,27 < < 3,63				
ρ/ρ_{max} =	0,93	should be < 1					

Final check				
f_{yd} =	434,78	N/mm^2	B500B reinforcement	
M_u =	410.378			
M_{ed} =	363.000			
M_u / M_{ed} =	1,13	should be > 1		
Shear Reinforcement				
cot θ =	2,5			
z =	1195,2	mm		
s =	200	mm		
f_{ywd} =	434,78	N/mm^2		
$A_{sw, req}$ =	1.016	mm^2/m		
Stirrups assumed as ϕ16				
A_{bar} =	200,96	mm^2		
N_{rei} =	6	Number of required bars		

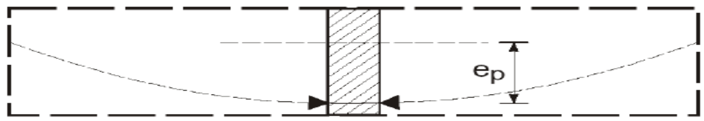
14.2 Gate Design Calculations with Prestressed Concrete

	Caisson geometry			
	H_c	22,5	m	Height
	B_c	36	m	Width
	L_c	230	m	Length
	t_w	1	m	Thickness of the wall
	t_f	1	m	Thickness of the floor
	t_r	0,5	m	Thickness of the roof
	n_x	5	Number of compartments in x direction, width direction	
	n_y	22	Number of compartments in y direction, length direction	
	t_{iw}	0,3	m	Thickness of the internal walls (compartments)

Mechanical properties of prestressing steel.

steel type	type	tensile strength		fracture strain	0,1% proof-stress	maximum tensile stress: during pre-stressing	maximum tensile stress: during pre-stressing with accurate jack	maximum tensile stress: initial stress	slope discontinuity in the σ-ε diagram (ULS)	modulus of elasticity
		f_{pk}	f_{pk}/γ_s	ε_{pu}	$f_{p0,1k}$	$\sigma_{p,max}$	$\sigma_{p,max}$	σ_{pm0}	f_{pd}	E_p
		MPa	MPa	‰	MPa	MPa	MPa	MPa	MPa	GPa
Y1030H	bar	1030	936	35	927	773	773	773	843	205 or 170
Y1670C	wire	1670	1518	35	1503	1336	1428	1253	1366	205
Y1770C	wire	1770	1609	35	1593	1416	1513	1328	1448	205
Y1860S7	strand	1860	1691	35	1674	1488	1590	1395	1522	195

	Prestressing steel		Y1860	
	$\sigma_{pmo} \leq$	1395	N/mm^2	
	$\sigma_{p,max} \leq$	1488	N/mm^2	no overstressing
	Total time-dependent losses =			20 %
	$\sigma_{pm, inf} =$	1116	N/mm^2	
	Requirement:	No concrete tensile stress		



Load as a result of prestressing (cross-sectional method)

P_m =	external axial prestressinng force
q_p =	unifromly distributed upward load

$$q_p = \frac{P}{R} = \frac{P}{\frac{l^2}{8e_p}} = \frac{8Pe_p}{l^2}$$

M_p =	parabolic bending moment with max at midspan

$$M_p = \tfrac{1}{8} q_p l^2 = P\, e_p$$

σ_{ct} =	Concrete stress at the top fibre of cross-section as a result of prestressing

$$\sigma_{ct} = -\frac{P_m}{A_c} + \frac{P_m\, e_p}{W_{ct}}$$

σ_{cb} =	Concrete stress at the bottom fibre of cross-section as a result of prestressing

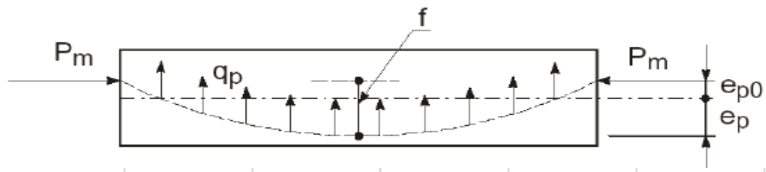
$$\sigma_{cb} = -\frac{P_m}{A_c} - \frac{P_m\, e_p}{W_{cb}}$$

Considering the effect of bending moment

$$\sigma_{ct} = -\frac{P}{A_c} + \frac{P\ e_p}{W_{ct}} - \frac{M}{W_{ct}} \leq 0$$

$$\sigma_{cb} = -\frac{P}{A_c} - \frac{P\ e_p}{W_{cb}} + \frac{M}{W_{cb}} \leq 0$$

or with equivalent prestressing load method



$$q_p = \frac{P_m}{R} = \frac{8\,P_m\,f}{l^2}$$

$$M = -P_m\,e_{p0} + \frac{1}{8} q_p\,l^2 = -P_m\,e_{p0} + \frac{1}{8}\frac{8 P_m\,f}{l^2} l^2 = -P_m\,e_{p0} + P_m\,f$$

$$M = P_m\,e_p$$

Design Check 1: Floating Caisson

Floating condition (empty caisson)

Floor

e_p =	0,8	m	
q_p ≈	55,69	kN/m	
R =	203	m	
P_m =	11.276	kN	

Using Y1860S7

σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	7	number of required tendons per meter	

Wall

e_p =	0,8	m	
q_p ≈	70,00	kN/m	
R =	79	m	
P_m =	5.537	kN	

Using Y1860S7

σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	4	number of required tendons per meter	

Roof			
e_p =	0,3	m	
$q_p \approx$	8,50	kN/m	
R =	540	m	
P_m =	4.590	kN	
Using Y1860S7			
σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	3	number of required tendons per meter	

Longitudinal wave loads

Floor			
e_p =	0,8	m	
$q_p \approx$	13,02	kN/m	
R =	101	m	
P_m =	1.320	kN	
Using Y1860S7			
σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	2	number of required tendons per meter	

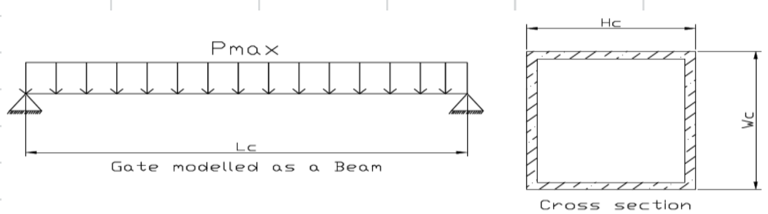
Design Check 4: Gate Immersed at Final Location During Hurricane Conditions

Local stresses

Floor		
e_p =	0,7	m
$q_p \approx$	120,00	kN/m
R =	231	m
P_m =	27.771	kN
Using Y1860S7		

σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	15	number of required tendons per meter	
Wall			
e_p =	0,8	m	
q_p ≈	137,00	kN/m	
R =	79	m	
P_m =	10.837	kN	
Using Y1860S7			
σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	7	number of required tendons per meter	
Roof			
e_p =	0,3	m	
q_p ≈	8,50	kN/m	
R =	540	m	
P_m =	4.590	kN	
Using Y1860S7			
σ_{pmo} =	1395	N/mm^2	
A_p =	1400	mm^2	per tendon (strand 7 ϕ18)
N_{ten} =	3	number of required tendons per meter	

Gate rests on Horizontal Supports Calculatior



e_p =	17	m		
q_p ≈	80 * H_c =	1800	kN/m	
R =	356	m		
P_m =	640.588	kN		

Considering the floor and roof for placing prestressing tendons

P_m /2 =	320.294	kN

Using Y1860S7

σ_{pmo} =	1395	N/mm^2	
A_p =	3000	mm^2	per tendon
N_{ten} =	77	number of required tendons in roof and floor	

Gate rests on Vertical Supports Calculations

e_p =	10	m	
q_p ≈	50 * W_c =	1800	kN/m
R =	605	m	
P_m =	1.089.000	kN	

Considering the walls for tendons (external walls and internal walls)

P_m /7 =	155.571	kN

Using Y1860S7

σ_{pmo} =	1395	N/mm^2	
A_p =	3000	mm^2	per tendon
N_{ten} =	37	number of required tendons per wall	

14.3 Bed Protection Calculations

14.3.1 Scour Depth Calculations

	Scour depth			
$m =$	0,6	contraction coefficient		
$B =$	220			
$q =$	7	$m^2/s\ /m$	underflow critical condition	
$h_t =$	16,5	m		
$H =$	7	m		
$d_{90} =$	0,5	mm		First layer caly will gone, sand will resist
$q^{0.4}$	2,18			
$H^{0.22}$	1,53			
$h_t^{0.4}$	3,07			
$g^{0.2}$	1,58			
$d_{90}^{0.22}$	0,86			
$y_{m,e} =$	7,57	m	Critical condition + 3 m clay layer	

14.3.2 Bed Protection Calculations

	Velocity under the gate		
$u_g =$	7,10	m/s	Under the gate velocity
$u_g =$	1,3	m/s	normal condition

	Bed protection, rip-rap or armourstone			
$D =$	$D_{n50} = 0{,}84 D_{50}$		m	Armourstone / rip-rap characteristic size
$\Phi_{sc} =$	0,5		Stability factor	
$\Delta =$	1,73		Relative buoyant density	
$\rho_r =$	2800	kg/m^3	Rock density (page 87 rock mechanics book)	
$\rho_w =$	1025	kg/m^3	Water density	
$\Psi_{cr} =$	0,035		Mobility parameter	
$k_t^2 =$	1,5	non-uniform flow, increase turbulance in outer bends		
$k_h =$	1		Velovcity profile factor (initial estimation)	
$k_{sl} =$	1		Side slope factor (page 549 rock mechanics), initial estimation	
$D =$	1,09	m		

	Bed protection, Box gabions			
	$D =$	$D_{n50} = 0,84D_{50}$	m	Armourstone / rip-rap characteristic size
	$\phi_{sc} =$	0,5		Stability factor
	$\Delta =$	1,04		Relative buoyant density
	$\rho_r =$	2800	kg/m^3	Rock density (page 87 rock mechanics book)
	$\rho_w =$	1025	kg/m^3	Water density
	$n_v =$	0,4		layer prosity
	$\psi_{cr} =$	0,07		Mobility parameter
	$k_t^2 =$	1,5	non-uniform flow, increase turbulance in outer bends	
	$k_h =$	1		Velocity profile factor (initial estimation)
	$k_{sl} =$	1		Side slope factor (page 549 rock mechanics), initial estimation
	$D =$	0,91	m	

14.4 Gate Berthing System Calculations

	Structure stiffness (Fender)			
	$K_{fender} =$	843	kN/m	Initial assumption (cell-fender)
	Velocity of the gate			
	$W_c =$	60.184	ton	weight of the caisson without reinforcement
	$W_{sc} =$	3.540	ton	weight of the reinforcement in caisson
	$W_g =$	63.724	ton	Total weight of the gate
	$v_g =$	0,2	m/s	Design velocity aon the basis of the guidlines

Hydrodynamic coefficient				
ρ_w	1025	kg/m^3	density of sea water	
L_g	230	m	length of the gate	
D_g	7,09	m	draught of the gate during the berthing	
m_w =	9.306.282	kg		
C_b	0,785		block coefficient	
B_g	36	m		
m_s =	47.244.503	kg		
C_H =	1,20		Hydrodynamic coeffcient	

Coefficient of eccentricity				
C_b	0,785			
k =	59,60			
ɣ	45	degree		
cos ɣ	0,53			
r	116,40			
C_{E1} =	0,43			
C_{E2} =	0,5		Recommended value (not less than)	
C_E =	0,50			

Softness coefficient				
C_s =	0,9		stiff structure	
Configuration coefficient				
C_c =	1		for safety	
Kinetic energy				
E_{kin} =	509	kNm	total	
E_{kin} =	170	each fender		
Force of berthing				
$F_{berthing}$ =	926	kN	total	
Assuming 3 fenders in the height of 8 meters of the abutment				
F_{fender} =	309	kN	Force acted on each fender	

14.5 Design Calculation of Abutments

14.5.1 Initial Design Calculations

Design loads 1 : Hurricane condition						
$q \approx$	80	kN/m^2	Unifrom distributed load from Gulf Side on Barge gate			
$q \approx$	1800	kN/m	Consideirng the height of the barge gate			
$R_{abt} =$	0,5 * q * L	=	198.000	kN	Reaction force on each abutment	
e =	12,25	m	Distance of the reaction force to the base			
$M_{max1} =$	2.425.500	kNm				
Design loads 2: Berthing condition during the closure						
$F_{berth} =$	309	kN				
$e_1 =$	20,84	m	Fender 1 distance to base			
$e_2 =$	18,18	m	Fender 2 distance to base			
$e_3 =$	15,52	m	Fender 3 distance to base			
$M_{max2} =$	16.853	kNm	Not normative compared to M_{max1}			

Initial design			
Using noraml reinforcement			
$H_{abt} =$	24	m	
$L_{abt} =$	10	m	
$B_{abt} =$	5	m	
$\gamma_c =$	17	kN/m^3	
$A_{s,req} =$	$M_{max} / 0,9 f_{yd} d$		Required reinforcement
d =	9800	mm	
$f_{ywd} =$	435	N/mm^2	B500B reinforcement
$A_{s,req} =$	632.184	mm^2	

Bars assumed as ϕ40				
A_{bar} =	1256	mm^2		
N_{rei} =	504	Number of required bars		
hth =	100	mm	Heart to heart of the bars	
Rows =	15,8		Number of rows of the bars in each meter	
A_s =	$\rho bd*10^4$	mm^2		
ρ =	1,29		Should be < 3,63	
W_{abt} =	2.040	Tons	weight of the pier	

14.5.2 Optimization Design Calculations

Optimization design				
Using prestressed concrete				
H_{abt} =	24	m		
L_{abt} =	7	m		
B_{abt} =	5	m		
γ_c =	17	kN/m^3		
M_{max1} =	2.425.500	kNm		
e =	6	m		
P_m =	404.250	kN		
Using Y1860				
σ_{pmo} =	1395	N/mm^2		
A_p =	6.000	mm^2	per tendon (strand 30 ϕ18)	
A_{req} =	289.785	mm^2		
N_{rei} =	49	Number of required tendons		
W_{abt} =	1.428	Tons	weight of the pier	

14.6 Stability Against Negative Head

	H_c	22,5	m	Height		
	B_c	36	m	Width		
	ΔH	2,8	m	Negative head		
	H_b	18,4	m	Effective buoyancy height		
	γ_w	10,25	kN/m^3			

Downward force

The previous calculations shows that 15 m of ballasting water secure the stability of the gate during the hurricane condition. This information is used here for calculation of the downward vertical force

$V_g =$	2771	kN/m	Force of the gate downward
$V_{wb}=$	5535	kN/m	Force of the ballasting water downward
$V_{buo} =$	6790	kN/m	Buoyancy force upward
$V_{res} =$	1516	kN/m	Resulting Vertical force downward

* The calculations are without considering the suction force which is on the conservative side

Horizontal force due to negative head

$F_1 =$	$0.5 * P_1 * H_1 =$	1481	kN/m	Hydrostatic force on Gulf side
$F_2 =$	$0.5 * P_2 * H_2 =$	2009	kN/m	Hydrostatic force on Bay side

$H_{res} =$	528	kN/m	Resulting Horizontal force due to the negative head

Horizontal stability requirement

$$\Sigma H < f \cdot \Sigma V$$

f =	0,35	assumed	Friction coefficient
H/fV=	1,00	Should be < 1 for horizontal stability	

14.7 Forces Acting on the Articulation System (Swing Point):

Inputs

C_T	0,15	Assumptions		
H_c	22,5	m	Height	in Z direction
B_c	36	m	Width	in Y direction
L_c	230	m	Length	in X direction
ρ_w	1,025	ton/m^3		
ρ_{air}	1,25	kg/m^3		
C_s	1		For rectangular barges	
u_c	1,3	m/s	Current velocity in normal condition	
u_w	20	m/s	Initial assumption for wind velocity in normal condition	
H_{sig}	3,3	m		
d	7,09	m	Initial draught	

Mean wave drift forces

$$F_{md} = (1/16).\rho_w.g.L.[(1-C_T).H_{sig}]^2$$

In X direction (surge direction)

$F_{md,surge}$ =	178	kN

In Y direction (sway direction)

$F_{md,sway}$ =	1137	kN

Current forces

$$F_c = (1/2).\rho_w.C_s.A.u_c^2$$

In X direction (surge direction)

A =	255	m^2
$F_{c,\ surge}$ =	221	kN

In Y direction (sway direction)

A =	1631	m^2
$F_{c,\ sway}$ =	1412	kN

Wind forces

$F_w = (1/2).\rho_{air}.C_s.A.u_w^2$

In X direction (surge direction)

A =	555	m^2
$F_{w,\ surge}$ =	139	kN

In Y direction (sway direction)

A =	3544	m^2
$F_{w,\ sway}$ =	886	kN

Summary

Steady Forces			
Force	**Surge direction**	**Sway direction**	**Unit**
F_{md}	178	1137	kN
F_c	221	1412	kN
F_w	139	886	kN
Total	**538**	**3436**	**kN**

14.8 Vertical Vibration Analysis

Total suspended mass (m)

m_{gt}	63.724	ton	Mass of the gate
m_g	28.322	ton	*Mass of the gate considering buoyancy with approximation
$m_{wballast}$	124.200	ton	Mass of the ballasted water inside the gate (15 m)
m_t	152.522	ton	Total effective mass of the gate and ballast underwater
m_t	1,53E+08	kg	

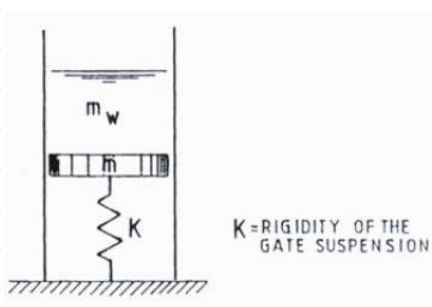
*By using the underwater mass for the concrete gate the buoyancy force has been taken into the account for the calculations

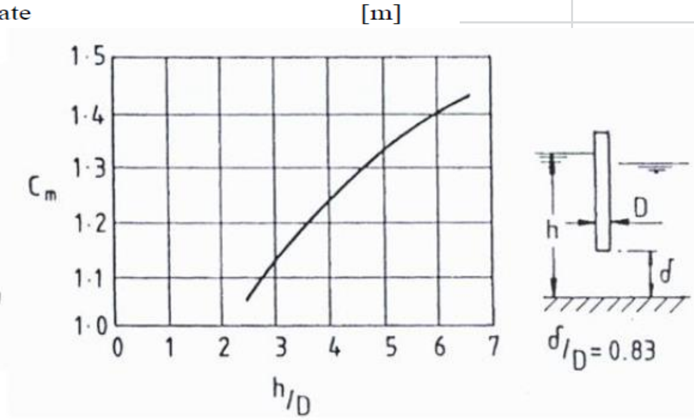
Added mass (m_w)

$$m_w = C_L \rho D^2 \quad \text{or} \quad C_m = \frac{m_w}{\rho D^2 L}$$

$$C_L = m_w / \rho D^2 \quad \text{hence} \quad C_L = L * C_m$$

m_w	=	added water mass	[kg]
C_M	=	added mass coefficient	[-]
C_L	=	$L*C_m$	[m]
ρ	=	density of water	[kg/m³]
D	=	characteristic immersed gate dimension	[m]
L	=	width of the gate	[m]





Trial and Error step 1

D	36	m	Overall thickness of the gate
h	18	m	In normal condition
h/D	0,5		not in the graph

Trial and Error step 2

D	36	m	Overall thickness of the gate
h	23,5	m	In Hurricane condition
h/D	0,65		not in the graph

Due to the huge dimensions of the gate (D), it is hard to estimate the added mass using this method. For the initial estimation the added mass is considered as zero.

m_w	0

Spring rigidity (k) of the suspension system

The system has the freedom in heave direction

k	0	Rigidity in vertical direction (heave)

Added water stiffness (k_w)

Added water stiffness is resulted by immersion, stationary flow and sudden movement of the gate. The simple formula is used here for the calculation [60]:

$$k_w = \rho g A_{intersection}$$

$A_{intersection}$	8.280	m^2	the area of the gate cutting through the water surface
k_w	8,33E+07	N/m	Added water stiffness

Damping (c)

Damping in a structure is because of different types of damping and different parts of the structure can contribute to it. The damper and spring of the gate system may be frequency dependent. For the initial estimation it is assumed that the damper is frequency-independent for every separate motion (vertical, horizontal and rotation).

Assumption: the total damping coefficient for the gate is assumed equal to 0,015 - 0,02

ζ	0,015

$$\varsigma = c/(2\sqrt{km})$$

c	3,38E+06	Ns/m	Damping

Added damping (c_w)

The gate moves vertically and perpendicular to the flow of water. The resulting damping will be almost zero in this case.

c_w	0	

Resonance frequency (f_r) / eigenfrequency (f_n)

The resonance frequency is not entirely equal to the natural or eigenfrequency. In the resonance frequency the added mass and added stiffness should be taken into account. However, in these calculations because of the assumptions we only consider the resonance frequency.

$$f_r = \frac{1}{2\pi}\sqrt{\frac{(k+k_w)}{(m+m_w)}}$$

f_r	0,12	Hz	Resonance frequency
T	8,50	s	

Excitation frequency (f)

Assumption: Excitation force is flow

Strouhal number: $S = f(L/V)$

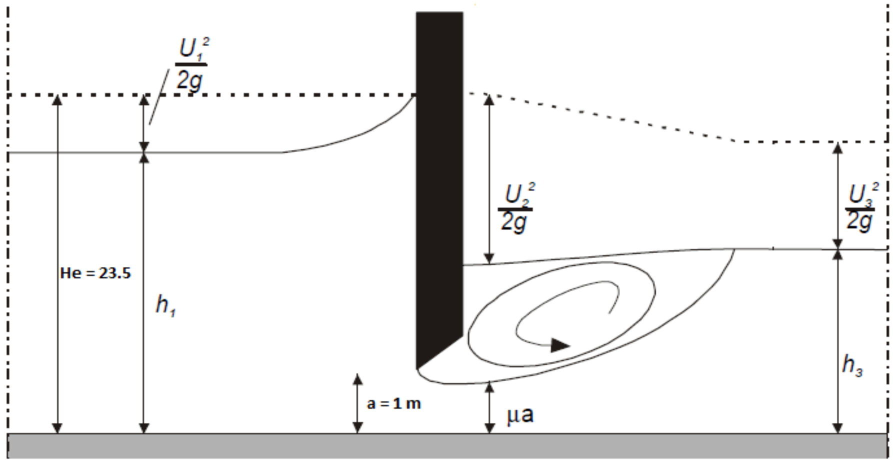
$V = \sqrt{(2gH_e)}$

H_e	23,5	m	the energy head

To determine the frequency of a vortex trail shed from the bottom edge of a partly open arge gate (figure below), the equation below can be used [60]:

$f = \sqrt{(2gH_e)} / 7L$

L =	7*h	length of the eddy behind the gate	
h	17	m	submerged depth of the gate
L	119	m	
f	0,74	Hz	
T	1,34	s	



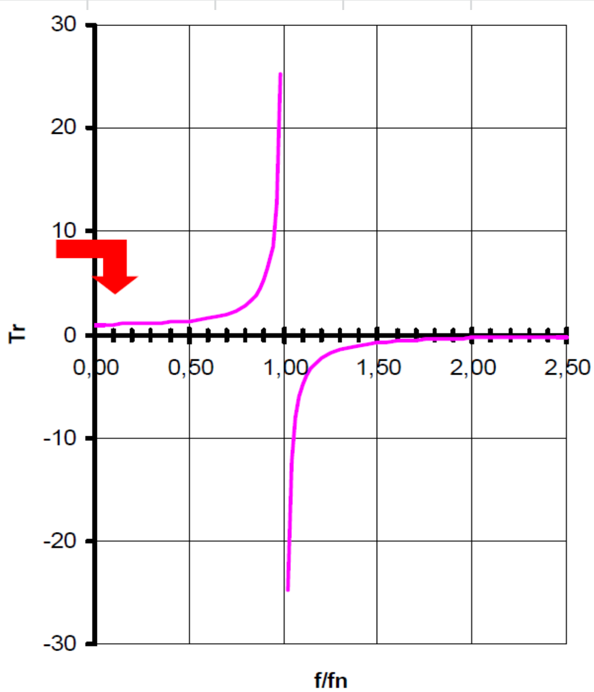
Transmissibility ratio (f/f_r) and Transmission coefficients (TR)

f/f_r	6,32	Factor 6	Advised: more than 3

$$TR = \frac{1}{1-(f/f_r)^2}$$

TR	-0,026	Advised: TR < -0,125

According to the graph below, it is advised that the excitation frequency (f) should be at least 3 times higher than the resonance frequency (f_r) of the structure which can be seen in this case. So the structure will not respond to the excitation.



	Critical damping coefficient (C_c) and damping ratio (ζ)			
	$C_c = 2\sqrt{km} = 2m\omega_n$			
	ω_n	$2*\pi*f_n$		
	ω_n	0,73883196		
	C_c	2,25E+08	from	$2\sqrt{km}$
	C_c	2,25E+08	from	$2m\omega_n$
	then obviously			
	ζ	0,015		
	Because of the fact that $\zeta << 1$ then oscillations occur with diminishing amplitude and the system is stable or positively damped.			

Conclusions:

- The excitation frequency (f=0.74) is significantly higher compared to the 1st eigenfrequency (f_n=0.12) with the factor of more than 3 as it is advised (factor 6).
- The system is positively damped, the occurring oscillations have diminishing amplitude and the system is stable.

14.9 Construction Costs Calculations

The estimation of the costs can be done very roughly because many of the cost components are not designed and determined yet.

Barge Gate Costs:

Assumptions:

- Barge gate is considered as the designed prestressed concrete gate in section 10.4
- Other assumptions are shown in the table below

Barge Gate Construction Costs					
Component	**Quantity**	**Unit cost (USD $)**	**Total Costs (USD $)**	**Remarks**	**Reference**
Prestressed Concrete (m^3)	38.942	1.250	48.677.750	Unit cost includes concrete, formework and rebars	[74]
Labor costs	1	-	9.735.550	Assumed as 20% of the concrete costs	[74]
Dry dock construction	1	-	7.301.663	Assumed as 15% of the concrete costs	[74]
Transportation	1	-	4.867.775	Assumed as 10% of the concrete costs	[74]
Installation	1	-	7.301.663	Assumed as 15% of the concrete costs	[74]
Unforseen costs	1	-	7.788.440	10% of the total above costs	[74]
Total barge gate Costs			**85.672.840**	**USD $**	

Ground preparation costs:

Assumptions:

- Dredging is assumed to be needed in the dimensions of 300 m length, 300 m width and 1.1 m depth in the place of the barrier
- Sand dumping with the same dimensions is assumed
- Bed protection is assumed to be needed in the same dimensions

Ground preparing of the barrier location costs					
Component	**Quantity**	**Unit cost (USD $)**	**Total Costs (USD $)**	**Remarks**	**Reference**
Dredging (m^3)	99.000	10	990.000		[74]
Sand dumping (m^3)	99.000	10	990.000		[74]
Bed protection (m^2)	99.000	180	17.820.000		[74]
Unforseen costs	1	-	198.000	10% of the total above costs	[74]
Total ground preparation Costs			**19.998.000**	**USD $**	

Abutments costs:

Assumptions:

- Abutments design in section 11.2 are used
- Other assumptions are shown in the table below

Abutments Costs					
Component	**Quantity**	**Unit cost (USD $)**	**Total Costs (USD $)**	**Remarks**	**Reference**
Prestressed Concrete (m3)	1.932	1.250	2.415.000	Unit cost includes concrete, formework and rebars	[74]
Labor costs	1	-	483.000	Assumed as 20% of the concrete costs	[74]
Dry dock construction	1	-	362.250	Assumed as 15% of the concrete costs	[74]
Transportation	1	-	241.500	Assumed as 10% of the concrete costs	[74]
Installation	1	-	362.250	Assumed as 15% of the concrete costs	[74]
Unforseen costs	1	-	386.400	10% of the total above costs	[74]
Total abutments Costs			**4.250.400**	**USD $**	

Foundations costs:

Assumptions:

- Foundations are not designed and treated in this report and the following assumptions have been considered
- Four shallow foundations with the dimensions of 40 m length, 5 m width and 1.5 m height from concrete are required for under two ends of the gate in the open condition and closed conditions
- In total 50 steel tubular piles (filled with concrete) with the dimensions of 1.5 m diameter, 80 mm thickness and 26 m length are required under the abutments and shallow foundations. This assumption is roughly derived from the calculations of reference [11]

Foundations Costs					
Component	**Quantity**	**Unit cost (USD $)**	**Total Costs (USD $)**	**Remarks**	**Reference**
Shallow foundations Prestressed Concrete (m3)	1.200	1.250	1.500.000	Unit cost includes concrete, formework and rebars	[74]
Labor costs shallow foundation	1	-	300.000	Assumed as 20% of the concrete costs	[74]
Deep foundations concrete (m^3)	2.296	450	1.033.256		[74]
Deep foundations steel (kg)	3.845.244	4	15.919.310		[74]
Labor costs deep foundation	1	-	3.390.513	Assumed as 20% of the deep pile construction costs	[74]
Piles transportation	1	-	1.695.257	Assumed as 10% of the concrete costs	[74]
Piles installations	1	-	2.542.885	Assumed as 15% of the concrete costs	[74]
Unforseen costs	1	-	2.638.122	10% of the total above costs	[74]
Total foundations Costs			**29.019.343**	**USD $**	

Other systems costs:

The costs of the systems which are not designed in this report and the information about them is unknown are roughly estimated in table below. This is just an initial estimation and it should e investigated more in the future researches.

Other Systems					
Component	**Quantity**	**Unit cost (USD $)**	**Total Costs (USD $)**	**Remarks**	**Reference**
Ballast system	1	-	8.567.284	Assumed as 10% of the barge gate costs	[82]
Articulation system	1	-	17.134.568	Assumed as 20% of the barge gate costs	Self estimation
Guiding columns & cables and propellers	1	-	8.567.284	Assumed as 10% of the barge gate costs	Self estimation
Berthing system	1	-	8.567.284	Assumed as 10% of the barge gate costs	Self estimation
Negative head lock / civil structure & water measurements system	1	-	4.283.642	Assumed as 5% of the barge gate costs	Self estimation
Unforseen costs	1	-	4.712.006	10% of the total above costs	Self estimation
Total other systems Costs			**51.832.068**	**USD $**	

Total Costs:

Total Costs Concrete	
Component	**Costs (USD $)**
Direct costs	
Barge gate	$85.672.840
Ground preparation	$19.998.000
Abutments	$4.250.400
Foundations	$29.019.343
Other systems	$51.832.068
Total Direct Construction Costs	**$190.772.652**
Indirect costs	
Design, supervision, engineering and administration (10%)	$19.077.265
Overhead (5%)	$9.538.633
Risk (5%)	$9.538.633
Profit (15%)	$34.339.077
Total Indirect Costs	**$72.493.608**
Total Construction Costs	
Total Construction Costs	**$263.266.259**
Total Investment Costs	
Project unforeseens (15%)	$39.489.939
Total Investment Costs (USD $)	**$302.756.198**

(Percentages of indirect costs and project unforeseen from reference [79].)

In case of using the steel material (assuming the other costs the same as the concrete case), the cost of the project would be as follow.

Total Costs Steel	
Component	**Costs (USD $)**
Direct costs	
Barge gate	$233.945.240
Ground preparation	$19.998.000
Abutments	$4.250.400
Foundations	$29.019.343
Other systems	$51.832.068
Total Direct Construction Costs	**$339.045.052**
Indirect costs	
Design, supervision, engineering and administration (10%)	$33.904.505
Overhead (5%)	$16.952.253
Risk (5%)	$16.952.253
Profit (15%)	$61.028.109
Total Indirect Costs	**$128.837.120**
Total Construction Costs	
Total Construction Costs	**$467.882.171**
Total Investment Costs	
Project unforeseens (15%)	$70.182.326
Total Investment Costs (USD $)	**$538.064.497**