The Houston Ship Channel Barrier
A Conceptual Design of the Dynamic Navigational Storm Surge Barrier. Houston, Texas, USA.

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Author
M.H. Schleppers BSc
ID: 1517236
E: mhschlepers@gmail.com
T: +31645119729
Pletterijstraat 43
2515AV Den Haag
The Netherlands

University
Delft University of Technology
Faculty of Civil Engineering & Geosciences
Stevinweg 1
2628CN Delft
The Netherlands

Graduation committee
Prof.dr.ir. S.N. Jonkman : Delft University of Technology
Ir. A. van der Toorn : Delft University of Technology
Dr.ir.drs. C.R. Braam : Delft University of Technology
A. Sebastian, EIT : Rice University, Houston, TX
Preface

This is the final report of the MSc Thesis titled *The Houston Ship Channel Barrier: A Conceptual Design of the Dynamic Navigational Barrier*. It represents the final part of the MSc program *Hydraulic Engineering - Field Hydraulic Structures*, containing the final research and design assignment I have elaborated to obtain the degree of Master of Science in Civil Engineering at Delft University of Technology.

The objective of this thesis was to develop a conceptual design of the movable storm surge barrier near Houston, Texas. A prestigious project on which several academics and students have dedicated their work, since 2008. I am happy with the result and thanks to the great relevance and importance of this subject it has been a pleasure working on. During the work, local stakeholders have clearly demonstrated the need of such a barrier, giving a feeling of gratitude to contribute to the process. Additionally, I really enjoyed using all knowledge obtained in the last couple of years in soil and fluid mechanics, engineering, probabilistics and expertise in planning and designing. Multidisciplinary international projects have grabbed my attention for several years already, this thesis has confirmed that I really want to make this my profession.

I could not have finished this project without the help of several people. First of all I would like to thank my graduation committee for their valuable input and support during the past nine months. I would like to thank Prof. dr. ir. S.N. Jonkman and Ir. A. van der Toorn for the formulation of the thesis content and their inspiring ideas. Special thanks goes to PhD candidate A. Sebastian for all help and provision of useful contact details and documents. I would thank Dr. ir. drs. C.R. Braam for his flexibility and effort being in my graduation committee. Furthermore I would like to thank the delegates from Houston, with in particular Dr. P.B. Bedient, C.M. Penland and J. Torres for their enthusiasm for my work and willingness to help.

In addition to representing the end of my graduation, this report marks the end of being a student in Delft. I would like to thank my family and friends who have supported me for all these years. My friends of the *Dampende Doctijiefs, R.A.S.P.* and *The Engelenbak* for providing the ideal environment for a student. Marjolijn for being the best support I can imagine and last but not least I would like to say thanks to my parents for always supporting me.
Abstract

The frequency of hurricanes along any 80 km segment of the Coast of Texas is about 1 in 6 years (Roth, 2009). On average, 4 major hurricanes (category 3 or higher) occur every century in the Houston-Galveston Region (Needham and Keim, 2011). In addition, the region is very vulnerable. Houston is the fourth biggest city in the US, the Houston Ship Channel (HSC) accommodates the largest petrochemical complex in the US and Galveston Bay provides large environmental value. Together, the population, industry and the environment make this region a very flood prone area. Action is required in the near future to protect Society, Economy and Environment.

Figure 1: The location of interest: (a) Houston-Galveston region with proposed protection measures (SSPEED 2013); (b) Alignment of the global protection system with three different levee sections, one environmental section and the navigational section; and (c) The final location of the HSC Barrier, indication of the navigational fairways and other relevant locations.

The main objective of this thesis is to develop an alternative design for a dynamic barrier to protect the HSC from flooding. By the use of a number of specific objectives and research questions, an optimized design has been developed. The objectives are based on previous studies and the identified local design and boundary conditions. The optimization methodology used, has an iterative character:
as the level of detail increases, the work converges towards an optimal solution based on available data and knowledge. First a global design of the protection system is made consisting of multiple barriers and levee sections. Subsequently the possible alternatives for the navigational barrier are summarized and one best suitable gate type is selected, based on its characteristics and the local conditions. A barrier of this type is designed in detail, reviewed and optimized where possible.

**Previous Studies**

Based out of Rice University, the SSPEED Center is closely involved in the development and research of a protection system for the Houston-Galveston region, aiming to be implemented in a reasonable time frame. A number of possible structures have been proposed by local parties and academic organizations since 2008, to protect both inside and outside Galveston Bay, as shown by Figure 1-(a). The structures presented in black (numbers 1 and 2) already exist and were built during the 20th century. This thesis focuses on a structure at the HSC, the possible alignments are shown in blue (A and B). The HSC itself is located just northwest of symbol A in Figure 1-(a). Recent studies by Sebastian et al. (2014), Bedient (2014) and Penland (2014) conclude that even with structures outside of Galveston Bay in place, a significant residual surge can occur in the Bay as a result of local wind set up. Therefore it is likely required to have a structure inside of Galveston Bay, protecting the most vulnerable areas.

**Design and Boundary Conditions**

The Houston Ship Channel Barrier will be designed for a lifetime of 100 years, in which it has to provide sufficient protection. A simplified cost-benefit analysis, in which the Benefit/Cost-ratio has been determined for different protection levels, resulted in an economical optimal design storm of 1/10,000 years. However, to adapt to local design trends, which use lower design standards, a 1/1,000 year storm event is used as the design storm. For the purpose of this thesis, a 1/1,000 year storm will be represented by $Ike + 30\%$ increased wind speed, which in combination with a westwards shifted track (landfall location P7, as defined by Sebastian et al. (2014)) is an often used ‘worst-case’ scenario in the literature. Design conditions associated to this storm are a wind velocity $u_{wind} = 65 \text{ m/s}$, a significant wave height $H_s = 6 \text{ m}$ and peak period $T_p = 9 \text{ s}$, and a water level elevation (storm surge) at the HSC entrance near Morgan’s Point of $8.0 \text{ m}$.

As a result of estimates given by existing literature and expected population and economic growth, the estimate amount of potential damages for the design storm is 105 Billion USD for the HSC, as a result of $100 \text{ Billion industrial damages } (\text{SSPEED} \ 2013), \ 4.0 \text{ Billion environmental damages } (\text{Rifai and Burleson} \ 2013) \ \text{and } 1.0 \text{ Billion in social damages } (\text{Viscusi} \ 2005). \ \text{Although the investment costs are estimated very roughly (7.0 Billion USD, for a 1/1,000 year protection level } P_f), \ \text{it can be concluded that the benefits of a storm surge barrier in terms of risk reduction are much higher than the investment costs } (I) \ \text{of this measure. The benefit in terms of risk reduction for an infinite time period is determined as the Net Present Value } (\text{NPV}), \ \text{where } D \ \text{is the associated amount of damages and the rate of interest } r = 0.05. \ \text{This results in a NPV of the risk reduction of } 51.5 \text{ Billion USD for a 1/1,000 year protection level.} \ \text{NPV} = \left( P_{f_0} - P_{f_{new}} \right)_D > I \ 

**Global Design of the Protection System**

The designed protection system stretches from the Highway 146, west of Morgan’s Point, to Highway 99, northeast of Cedar Crossing Industrial Park. The chosen alignment variates slightly from line B in Figure 1-(a), using existing infrastructure to minimize the length of levees to be constructed and impact on private property, which is a requirement.
As presented in Figure 1-(b), the protection system consists of three levee sections; an environmental section, which contains dunes and a small caisson barrier to allow environmental flow; and a navigational section, which contains a movable barrier. The levee section at the west end of the protection system makes use of the existing Barbour’s Cut Boulevard, which is an elevated road, reducing the required amount of space and materials for construction. It stretches to Morgan’s Point, where it connects to the navigational barrier. On the other side of the navigational barrier the Wildlife Management Area of Atkinson Island is located.

In order to protect the hinterland from flooding during a 1/1,000 year storm event, levee sections 1 and 3 will have the crest height at $MSL + 8.5\ m$ to limit overtopping. The local ground surface at levee sections 1 and 3 is at $MSL + 6.0\ m$ and the 2.5 m high levees on top of this need to have 1:3 slopes. Due to the less shallow foreshore at Atkinson Island, the crest level of Levee section 2 need to be at $MSL + 12.5\ m$. This levee will have 1:5 outer slopes and a 12.0 m wide berm at $MSL + 7.5\ m$. All levee slopes will be grass covered to enhance the natural character of the area.

East of Atkinson Island, levee sections 2 and 3 are connected to the Environmental section, which crosses the 1.5 km wide Cedar Bayou as presented in Figure 1-(c). This environmental section will maintain the natural character by allowing tidal movement and a dynamic coastline, while it can close off Cedar Bayou to protect the hinterland from flooding during storm events. It will consist of dunes for the major part, constructed out of dredged spoils. In the center it will contain a caisson structure to allow in and outflow, following the tidal prism. The design of this environmental barrier contains a road to improve the accessibility of Atkinson Island.

Technical Design of the Navigational Barrier: Horizontally Sliding Concrete Gate

A Multi Criteria Analysis (MCA) has shown that a horizontally sliding gate is the most suitable option for the navigational barrier. Based on existing works by Penland and Cibor (2013), the first technically designed navigational barrier alternative is a horizontally sliding concrete gate with a rectangular shape. Under normal conditions, this gate is stored at a dry dock at the side of Morgan’s Point. During operation (a closed gate) the stability is maintained by the great dead weight for the major part and partly by the sill on the bottom of the channel, which transmits the loads to the foundation.

While moving, stability will be provided by a guidance rail which is integrated in the shape of a sill at the channel bottom. Unlike a floating gate, the transversal forces due to the side current will be directly transmitted to the foundation. However, sedimentation of the sill has to be eliminated and variations of the sill surface elevation need to be minimized. During movement, the direction is addressed using hydrofenders of which a pair is installed every 50 m of gate length, creating a thin water layer between the gate and the sill to reduce friction and flush away sediments. The driving mechanism makes use of multiple pulleys, enabling gate movement either way by cable pulling.

The worst case scenario for stability is found for a hydraulic head difference of $8.0\ m$ between the water level at the bay side ($MSL + 8.0\ m$) and the water level at the HSC side ($MSL$). This scenario results in a gate width of 18 $m$, required to maintain stability. With the channel bottom level at $MSL − 22\ m$ the total gate height is 35 $m$, as a result of the surge, wave height (6.0 $m$) and 2.0 $m$ freeboard. Initial steps in optimization of the cross section of the gate has resulted in a configuration of 6 hollow spaces so that a minimum amount of concrete is required while sufficient strength of all elements has been guaranteed. The hollow spaces can be filled with water to increase the dead weight of the gate to ensure stability. The final design of the cross-section of the gate is presented in Figure 2-(a), an indication of the loads acting on the gate is given as well.

The bearing soil layer is present at depths of $MSL − 50$ to $-60\ m$. For this situation a foundation consisting of inclined close-ended steel tubular piles turns out to be the best option. The final pile configuration is presented in Figure 2-(c), where piles with diameter $\phi_{t} = 2.5\ m$, steel thickness $t_{1} = 0.025\ m$ and an inclination of 1:3 is used to provide sufficient bearing capacity. On top of the pile foundation the sill is installed, which is constructed out of prefabricated prestressed concrete slabs.
of 20 by 30 meters, as presented in Figure 2(b). The guidance rail is integrated in the shape of the sill, where by the use of prefabrication a maximum quality of the sill profile can be realized, avoiding problems related to height variations. Since the 30 m wide foundation slabs are constructed on top of a clay layer, it is expected to have no issues related to seepage.

On each side of the channel an abutment is constructed for load transmission and leakage reduction. At the most accessible western side, near Morgan's Point, the dry dock is located for storage of the gate. All critical systems and mechanisms are grouped on this side. The dry dock has the shape of a rectangular concrete box, with an open top. The sides consist of diaphragm walls, which bear on the same soil layer as the pile foundation. Halfway the height of these walls is the concrete bottom slab of the dry dock constructed, which has the same shape as the concrete slabs at the channel, but with extended sides in order to provide space for construction of the gate, maintenance and inspection.

Figure 2: Elements of the concrete gate alternative: (a) Overview of the final gate cross-section and present loads; (b) Cross-section of the foundation slabs; and (c) Pile configuration (plan view).

Alternative Design of the Navigational Barrier: Horizontally Sliding Steel Truss Gate

The technical design of the concrete gate alternative has resulted in a robust construction that is expected to meet the strength and stability requirements, where issues related to gate movement have been addressed and maintenance and inspection are enabled. However, the concrete gate is rather heavy and there are some more aspects in the MCA for which the gate does not provide a perfect performance. The purpose of the Alternative Design is to improve these aspects. It will also be sliding horizontally at a guidance structure, integrated in a sill which is installed on top of a pile foundation consisting of inclined close-ended steel tubular piles. This alternative design however, will have a triangular shape like a truss and is constructed out of steel tubes which has a number of advantages compared to a rectangular concrete wall. This steel alternative is presented in Figure 3.

The triangular shape greatly decreases the loads on the foundation and improves the load transmission to the foundation while it guarantees the overall stability of the structure. Additionally, the sloping water retaining wall will receive less wave impact and cause less wave reflection. Besides the gate design, the designs of the abutments and dry dock are improved as well, contributing to the landscape integration and spatial quality. By including a museum in the design of the barrier, extra value has been added in terms of recreational quality and educational facilities.
Conclusions

The main objective of the thesis is met by the presented Technical Design of a Horizontally Sliding Concrete Door, located at the mouth of the HSC. An Alternative Design, which contains a Steel Truss Gate, is presented aiming on improving the original design. Both alternatives enable a solid protection for the HSC against a 1/1,000 years storm event, where the design of the Steel Truss Gate has improved aspects related to foundation requirements, dynamics and aesthetics. In addition to the protection of Society, Economy and Environment, the steel truss barrier could offer added value as a "lifestyle attraction" which is integrated in the landscape.

<table>
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<tr>
<th>Alternative:</th>
<th>Concrete Door</th>
<th>Steel Truss</th>
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<tbody>
<tr>
<td>1. Height of the gate (and top level)</td>
<td>$35 \text{ m (MSL + 13 m)}$</td>
<td>$33 \text{ m (MSL + 11 m)}$</td>
</tr>
<tr>
<td>2. Width of the gate</td>
<td>$18.0 \text{ m}$</td>
<td>$49.5 \text{ m}$</td>
</tr>
<tr>
<td>3. Selfweight of the gate (dry)</td>
<td>$8,250 - 11,550 \text{ kN/m}$</td>
<td>$1,250 \text{ kN/m}$</td>
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<tr>
<td>4. Volume of concrete foundation slab</td>
<td>$130 \text{ m}^3/m$</td>
<td>$200 \text{ m}^3/m$</td>
</tr>
<tr>
<td>5. Pile configuration ($n_piles \cdot \Phi_t - \Delta x$)</td>
<td>$12 \cdot \Phi_t 2.5 - 20 [m]$</td>
<td>$12 \cdot \Phi_t 2.0 - 16.7 [m]$</td>
</tr>
<tr>
<td>6. Movement mechanism</td>
<td>Hydrofenders with pulleys</td>
<td>Rack and pinion</td>
</tr>
<tr>
<td>7. Gate head area (movement resistance)</td>
<td>$630 \text{ m}^2$</td>
<td>$400 \text{ m}^2$</td>
</tr>
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</table>

The majority of the objectives has been met, although some aspects are still improvable. It is recommended to investigate alternatives related to Gate Movement, the Construction Method and Planning and to make a Global Cost Estimate. It is also recommended to investigate additional non-structural measures, determine the required response time for different actions and to develop a detailed protocol for gate operation.
# Contents

1 Problem Description 21
   1.1 Introduction . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 21
   1.2 Historic storm events . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 23
   1.3 Actors . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 26
      1.3.1 Society . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 26
      1.3.2 Economy . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 26
      1.3.3 Environment . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 27
   1.4 Problem definition . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 27
      1.4.1 Main problem . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 27
      1.4.2 Objectives . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 27
      1.4.3 Research questions . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 28
   1.5 Methodology . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 28
      1.5.1 Design methodology . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 28
      1.5.2 Structure of the report . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 28

2 System Description and Boundary Conditions 31
   2.1 Introduction . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 31
   2.2 Geography . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 31
      2.2.1 Greater Houston and Houston Ship Channel Description . . . . . . . . . . . . . 32
      2.2.2 Galveston Bay . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 32
      2.2.3 Climate . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 33
      2.2.4 Freshwater Inflow . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 34
   2.3 Infrastructure . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 38
      2.3.1 Land use . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 38
      2.3.2 The Houston Ship Channel . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 39
      2.3.3 Vessels . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 40
   2.4 Geology . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 42
      2.4.1 Surface Elevation . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 42
      2.4.2 Geo-technical Information . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 42
      2.4.3 Land Subsidence . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 43
   2.5 Coastal processes . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 44
      2.5.1 Normal conditions . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 44
      2.5.2 Storm conditions . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 46
   2.6 Flood risk . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 53
      2.6.1 Basics of Risk-Based Flood Protection . . . . . . . . . . . . . . . . . . . . . . . . . 53
      2.6.2 Probability of Occurrence . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 54
      2.6.3 Consequences of Flooding . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 55
      2.6.4 Risk Reduction . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 57
   2.7 Summary . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . 59
2.7.1 Assumptions and Boundary Conditions .............................................. 59
2.7.2 Design Conditions ............................................................................. 60
2.7.3 Requirements and Design Criteria .................................................... 61

3 Global Protection System Design ......................................................... 63

3.1 Introduction ......................................................................................... 63
3.2 The Presence of Other Structures .......................................................... 64
3.3 Barrier Alignment ............................................................................... 65
  3.3.1 Comparison of Alignments ............................................................. 65
  3.3.2 Selection ......................................................................................... 66
3.4 Design of the levee sections ................................................................. 66
  3.4.1 Crest height .................................................................................... 67
  3.4.2 Dike Optimization by wave Run-up and Overtopping criteria .......... 68
3.5 Environmental section ......................................................................... 70
3.6 Navigational section ........................................................................... 71
3.7 Additional protection methods ............................................................. 72
  3.7.1 Early warning systems ................................................................. 72
  3.7.2 Education ....................................................................................... 72
  3.7.3 Evacuation .................................................................................... 72
  3.7.4 Landuse planning .......................................................................... 73
3.8 Conclusions ......................................................................................... 73

4 Navigational Gate Selection ................................................................. 75

4.1 Introduction ......................................................................................... 75
4.2 Gate alternatives .................................................................................. 76
  4.2.1 Open channel .................................................................................. 76
  4.2.2 Horizontally moving gates ............................................................... 77
  4.2.3 Vertically moving gates ................................................................... 80
  4.2.4 Costs of Existing Storm Surge Barriers ........................................... 84
  4.2.5 Summary ....................................................................................... 85
4.3 Multi Criteria Analysis ......................................................................... 86
  4.3.1 Criteria .......................................................................................... 86
  4.3.2 Weight factors ............................................................................... 87
  4.3.3 Results ......................................................................................... 88
4.4 Conclusions ......................................................................................... 89

5 Technical Design: Horizontally Sliding Gate Barrier .......................... 91

5.1 Introduction ........................................................................................ 91
5.2 Concept .............................................................................................. 93
  5.2.1 Movement mechanisms ................................................................. 93
  5.2.2 Failure mechanisms ...................................................................... 95
5.3 Load definition ..................................................................................... 96
  5.3.1 Loads on the barrier ...................................................................... 96
  5.3.2 Load distribution ........................................................................... 98
5.4 Design of the cross section ................................................................. 100
  5.4.1 Stability ......................................................................................... 101
  5.4.2 Strength ......................................................................................... 102
  5.4.3 Dynamics ...................................................................................... 103
5.5 Design of the Movement mechanism ................................................. 105
5.6 Design of the Foundation ................................................................... 107
  5.6.1 Sill ............................................................................................... 107
  5.6.2 Pile foundation ............................................................................. 110
5.6.3 Protection against erosion .................................................. 112
5.7 Design of the Abutments .......................................................... 114
  5.7.1 Abutment head ............................................................... 115
  5.7.2 Dry Dock .................................................................. 116
5.8 Conclusions .................................................................... 120

6 Alternative Design: Horizontally Sliding Steel Truss 123
  6.1 Introduction .................................................................. 123
  6.2 Concept ....................................................................... 124
  6.3 Load definition ............................................................... 125
  6.4 Design of the Truss Gate ...................................................... 126
    6.4.1 Stability .................................................................. 127
    6.4.2 Strength .................................................................. 129
    6.4.3 Dynamics ................................................................. 132
  6.5 Design of the Movement Mechanism ..................................... 133
  6.6 Design of the Foundation ...................................................... 134
    6.6.1 Sill ........................................................................ 134
    6.6.2 Pile foundation .......................................................... 136
    6.6.3 Protection against erosion ............................................ 137
  6.7 Design of the Abutments ...................................................... 137
    6.7.1 Abutment heads .......................................................... 137
    6.7.2 Dry dock .................................................................. 137
    6.7.3 Additional structures ................................................... 139
    6.7.4 Overview ................................................................. 140
  6.8 Conclusions .................................................................... 141

7 Conclusions and recommendations 145
  7.1 Introduction .................................................................. 145
  7.2 Conclusions .................................................................. 145
  7.3 Recommendations ............................................................ 148

Bibliography ..................................................................... 151

List of Figures .................................................................. 157

List of Tables .................................................................... 165

Appendices .......................................................................... 169
  A Historic storm events .......................................................... 171
  B Consequences of Flooding ..................................................... 179
  C Protection system alignments ............................................... 185
  D Channel requirements ........................................................ 197
  E Concrete barrier design ......................................................... 201
  F Composite barrier design ...................................................... 225
  G Foundations .................................................................. 233
**List of Symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Unit</th>
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Acronyms

API  American Petroleum Institute
ASCE  American Society of Civil Engineers
BMDS  Barchart Market Data Solutions
CBA  Cost-Benefit Analysis
CBS  Columbia Broadcasting System
CCMP  Comprehensive Conservation and Management Plan
CO-OPS  Center of Operational Oceanographic Products and Services
EPA  Environmental Protection Agency
FEMA  Federal Emergency Management Agency
GBNEP  Galveston Bay National Estuary Program
GHPRD  Greater Houston Partnership Research Department
GOM  Gulf Of Mexico
HCFCD  Harris County Flood Control District
HGAPS  Houston Galveston Area Protection System
HGSD  Harris Galveston Subsidence District
HMA  Heavy Mass Aggregate
HOGANSAC  HOuston GAIvestn NAVigation Safetv Advisory Committee
HSC  Houston Ship Channel
IHNC  Inner Harbor Navigation Canal
ISU  Iowa State University of Science and Technology
MCA  Multi Criteria Analysis
MOSE  Modulo Sperimentale Elettromeccanico (Experimental Electromecanical Module)
MSL  Mean Sea Level
NEP  National Environmental Policy
NOAA  National Oceanic and Atmospheric Administration
NPV  Net Present Value
PHA  Port of Houston Authority
PIANC  Permanent International Association of Navigation Congresses
SSPEED  Severe Storm Prediction, Education and Evacuation from Disasters
TAMUG  Texas A & M University Galveston
TCEQ  Texas Council of Environmental Quality
THI  Tropical Hazard Index
TSHA  Texas State Historical Association
USACE  United States Army Corps of Engineers
USCD  United States Climate Data
USCEQ  United States Council of Environmental Quality
USD  United States Dollars
USGS  United States Geological Survey
USIC  United States Inflation Calculator
UT  University of Texas
CONTENTS
Chapter 1

Problem Description

1.1 Introduction

Hurricanes are a common problem in the areas surrounding the Gulf of Mexico, and the Houston-Galveston region is no exception. As one of the largest growing populations in the United States, the vulnerability of the area to flooding has been revealed by Brody (2014), as well as the risk of flooding of the Houston Ship Channel (HSC), accommodating the US’ major petrochemical industries, giving both economical as well environmental risks (Rifai and Burleson, 2013). This has caused a change in strategy from reactive to proactive (Merrell, 2012). This risk recognition has among others led to the establishment of SSPEED Center, based out of Rice University. The major goal of this research center is to develop and implement a protection system for the critical facilities in the Houston-Galveston Region in a reasonable time frame (Bedient, 2014).

The response of the Galveston Bay and the HSC to a hurricane storm surge has been modeled by Dawson et al. (2013) and Sebastian et al. (2014). They discovered that a probable worst case scenario for the area would have occurred if Hurricane Ike (2008) made landfall at a distance of about 40 km west of the original landfall location. Additionally, Hurricane Ike was ‘just’ a category 2 storm (see Sections 1.2 and 2.5.2), whereas category 4 storms have occurred in the Houston-Galveston area during the last century. Increasing wind velocities would result in even higher storm surge at the HSC entrance (Morgan’s Point). The results of Dawson et al. (2013) and Sebastian et al. (2014) can be used to determine the design conditions for a storm surge protection system.

Several different protection concepts are possible and have been developed in the past few years. Figure 1.1 shows the different concepts to protect the Houston-Galveston area. The Galveston Seawall and the Texas City Levee, numbers 1 and 2 respectively, are existing structures. The structures indicated with A–J represent the different structures required for either protection inside or outside the bay. Besides the locations of the structures (inside, outside or a combination) different types of protection are possible as well. For some of the different structures first sketch designs have already been made and for some the level of protection of the Houston-Galveston region has been determined using computational models. Several designs have already been developed for the Ike Dike (Merrell et al., 2011) at the sea side of Galveston Bay (F and G). The design that followed from a workshop at Delft University of Technology (Jonkman et al., 2013) includes a gate at the Bay entrance near Galveston (J), whereas Bedient (2014), Colbert and Shanley (2013) and Penland and Cibor (2013) all located a storm surge barrier at the entrance of the HSC. For the HSC entrance, Bedient (2014) discussed two different locations on a smaller scale as well (A and B). In addition to ‘hard’ structures like levees, dams and barriers, protection concepts are developed which make use of soft materials like banks constructed out of dredged disposals (E) or oyster reefs (D), following the relatively new idea of ‘Building with Nature’ (Wesenbeeck, 2014).
Although some of the ideas about the elaboration of the area its protection system may be in conflict with each other, it has been made clear to all stakeholders that it is necessary to take action in the near future (Waterworth 2014). The purpose of this MSc Thesis is the elaboration of a technical design for the Houston Ship Channel Barrier, at location A or B in Figure 1.1. In the following section some important historic storm events are briefly described after which the objectives of a storm surge barrier will be defined on the basis of identification of the different actors involved and their needs. Subsequently the main problem will be defined after which the different objectives and research questions will be formulated. This chapter concludes with the description of the design methodology and the presentation of the report structure.

Figure 1.1: Different concepts to protect the Houston-Galveston region, obtained from SSPEED (2013). The existing structures are presented in black (numbers 1 and 2), in blue are the two potential locations for the Houston Ship Channel Barrier, on which the main focus will lay on.
1.2 Historic storm events

The occurrence of floods due to extreme weather conditions is well known for the areas surrounding the Gulf of Mexico. Tropical cyclone-generated storm surges are among the most deadly and costly natural disasters to impact the United States (Needham and Keim, 2011). The hurricanes at the coast of Texas have been collected by both Roth (2009) and Blake and Gibney (2011), giving the characteristics of the occurred hurricanes and the resulting damages in both capital loss and loss of life. Most recently, Needham and Keim (2011) gathered characteristics of 195 surge events since 1880 creating the first comprehensive storm surge database for the US Gulf Coast, giving the surge magnitudes in combination with the landfall location and return period. This database, called SURGEDAT, may reduce human and economic losses as the information will improve coastal zone planning, emergency management preparation and public education.

According to Blake and Gibney (2011), 61 tropical storms have hit the Upper Texas Coast since 1854 where the frequency of hurricanes in along any fifty mile (80 km) segment of the Texas coast is about 1 in 6 years (Roth, 2009). According to Roth (2009), the longest hurricane-free period for Texas since 1829 was nearly 10 years, between October 1989 and August 1999. The occurrence of hurricanes and tropical storms is very seasonal, for the Texas coast these kind of storms only occur during the summer months: from the beginning of June until the end of October.

Flooding is by far the most serious threat from these tropical storms, resulting from wind induced set-up (storm surge), excessive river run-off due to intense rainfall or a combination of both. The worst thing about this, in a flood protection point of view, is that the weaker the system is in terms of wind velocities and storm surge, the more intense the rainfall (Roth, 2009). For the Houston Ship Channel, this means that both run-off from the Jacinto River and storm surge at Galveston Bay can cause floods and thus have to be taken very seriously. It can be concluded that any system, no matter what strength, can create major problems of flooding in the region (Torres et al., 2015).

Figure 1.2: Hurricanes at the Texas Coast. The different tracks recorded on the left, the occurrence of storms and hurricanes per month on the right. (Roth, 2009)

As stated, 61 tropical storms have been recorded in the past 150 years. In this section the characteristics of some of the most recent hurricanes that caused major damages to the Houston-Galveston region will be presented, as well will be the resulted damage and number of deaths. A clear increase in damages can be observed with the time, which is of course a result of the fast increasing population and industrial activity in the area. According to Read (2012) hurricane disasters will continue to be more costly, mainly due to people’s inability to set proper land use and building codes in sensitive coastal areas. For a complete overview of historic events see Appendix A.
Galveston Hurricane (1900)

The great Galveston Hurricane of September 1900 is responsible for a death toll of about 8,000 people and is therefore the worst natural disaster in the United States in terms of lives lost. Winds of over 125 mph (200 km/h) caused a storm surge that covered Galveston Island by 15 feet (4.6 m) of water, additionally the 24-hour rainfall record was broken: 10" (25.4 cm) fell in Galveston. More than 2,600 houses were demolished and 3/4 of the city was completely wiped out (Roth, 2009).

At that time Galveston was one of the biggest US cities near the Gulf of Mexico, but the Great Galveston Hurricane of 1900 made people decide to move the harbor more inland to find more protection, resulting in today’s location of the Houston Ship Channel Industrial Complex. An interesting aspect of the Galveston hurricane is that it followed nearly the same path as Hurricane Ike did 108 years later, the most recent hurricane induced flood disaster in the region.

Tropical storm Allison (2001)

June 5-9, 2001: A low, originating off the West coast of Africa, became a tropical storm about 80 miles south of Galveston. For this event storm surge was not the main cause of flooding, but excessive rainfall was. When entering Texas it dropped over 25 cm of rain, at the Port of Houston almost a meter of rainfall was found. Rains continued for one week while Allison was traveling along the coast of the U.S. (Figure 1.3) which set the stage for massive flooding in southeast Texas. Damage to Texas totaled $5 billion and 22 deaths (Roth, 2009), making it the most expensive urban flood event due to a tropical storm in US History. It is ranked in the top 10 of all tropical cyclones or hurricanes in terms of amount of damage.

Hurricane Rita (2005)

Forecasts predicted the landfall location of hurricane Rita very near Houston, which made the authorities decide to evacuate the residents of Galveston and some parts of Houston. However, as presented in Figure 1.4, Hurricane Rita made landfall near the border between Texas and Louisiana, eastwards from Houston, and thus not resulted in any significant storm surge or rainfall in the Houston-Galveston Area (HGA). Still this was a notable historical event: During evacuation, people got stuck with their cars in traffic jams which in combination with very high temperatures resulted in dehydration. Since there was no possibility for emergency services to arrive; 103 deaths occured during the Rita event. The poor response for Hurricane Rita prompted the re-evaluation of hurricane planning, both (flood) mitigation and evacuation planning and preparation.
Hurricane Ike (2008)

With wind speeds up to 175 km/h, Hurricane Ike was a category 2 hurricane, see Table 2.4 in Section 2.5.2 and is the 3th costliest hurricane in the US history with $27 billion in damages. The worst damage was seen in Galveston and Chambers counties (storm surge reached 17 ft [5.2 m] in Chambers City) where some communities faced a total destruction. A number of 63 persons died directly during Ike and a total of 124 deaths were associated with this hurricane. Ike destroyed 52 oil platforms, severely damaged 32 platforms, tossed storage tanks, punctured pipelines, half a million gallons of oil spilled in the Gulf of Mexico, 448 releases reported mostly in Port Arthur and the industrial corridor in Houston and over 1500 sites needed a cleanup (Bedient, 2014; Berg, 2009; Rifai and Burleson, 2013; National Weather Service, 2014; Read, 2012).

Hurricane Ike was one of the best monitored surge events in history. An important observation was the forerunner effect; due to the shallow Texas shelf and the high wind speeds a first flood wave
entered Galveston Bay hours before the peak flood wave arrived, causing a significant increase of water volume in the bay before the actual peak surge arrived. This phenomenon dramatically increased the impact of the peak surge due to the increased volume of water in the bay (Kennedy et al., 2011). Hurricane Ike was an eyeopener for the region. It became apparent that something had to change to avoid future flood disasters.

1.3 Actors

The objective of a storm surge barrier, as defined by Len Waterworth (Waterworth, 2014), is to protect people, businesses and environment (Figure 1.6). This triangle indicates the first three major actors for the Houston Ship Channel case, representing the local residents, the Port of Houston and HSC industries and the environmental organizations respectively. These three main groups represent multiple stakeholders and parties, for whom a brief summary is given in this Section (SSPEED, 2013).

![Figure 1.6: Barrier objectives, by Len Waterworth.](Waterworth, 2014)

1.3.1 Society

This group represents the local residents. Covering the residents of the city of Houston, Harris County and all other surrounding cities located at the floodplains behind the proposed barrier locations (see Chapter 3) as well the people that might not live in the areas with risk of flooding. For example, people that work at the Houston Ship Channel Industrial Complex, for whom it is important that their employment remains guaranteed. And, those whose jobs or livelihood would be impacted by damage to the HSC (e.g. fishers who would be impacted by an oil spill). To conclude, a proposed storm surge barrier as part of the whole flood protection system has to give sufficient protection to the millions of people that live and work in these areas.

1.3.2 Economy

Obviously, the businesses established at the Port of Houston want to protect their properties and economical value, but at the moment the following contradiction can be seen: Companies are looking for the least restrictive government and they do not want to contribute to an integral flood protection system. Currently, Ship Channel Industries protect their own property by taking private flood protection measures like small dikes and drainage systems, which all together is much more expensive than one structure that protects the whole area.

A second issue one has to deal with in this area is that the companies are not willing to make any statements about flood risks. According to Hogendoorn (2014) this can be explained by a number of observations: first openness and acknowledgement of a possible hazard creates legal liability, which is one thing the companies do not want; secondly they think that openness will create political problems; and thirdly the focus of the top of the business is not on the issue since they are busy with other things, while the executive staff - who face the risks - is not allowed to make any independent decisions. A
huge mind switch is required to enable an integral flood protection system, containing a storm surge barrier that protects the whole Houston Ship Channel Industrial Complex.

1.3.3 Environment

Galveston Bay is the 7th largest estuary in the US and recognized by the US Environmental Potrection Agency (EPA) as an estuary of national significance under the National Environmental Policy (NEP). The environmental and ecological quality in the Galveston Bay is great and this has to be preserved at all times. Parties that can be categorized under the title ‘Environment’ are the environmental organizations. Some examples are the EPA, which sets environmental requirements, and the so-called CEQ’s: US Council of Environmental Quality (USCEQ) and Texas Council of Environmental Quality (TCEQ). Their most important principles are: protect and restore natural ecosystems and the environment while encouraging sustainable economic development; avoid unwise use of floodplains, flood-prone areas and other ecologically valuable areas; and give full and equal treatment to nonstructural approaches. [SSPEED, 2013]. Local organizations are also stakeholders like the Galveston Bay Foundation and Galveston Bay Council.

1.4 Problem definition

1.4.1 Main problem

The problem of flooding due to extreme weather conditions in the very vulnerable area of Houston has been made clear in the previous sections. From these sections a clear connection can be made between people, business and environment. This triangle is of major importance to get to a feasible solution, giving the main purposes of the final design: Protect people, protect industry and protect the environment.

1.4.2 Objectives

Main objective

Design an (alternative) dynamic barrier to protect the Houston Ship Channel from flooding.

Specific objectives

1. Map the requirements, boundary- and design conditions.

2. Determine a suitable protection level, based on the benefits in terms of risk reduction.

3. Choose an appropriate alignment based on the existing infrastructure and resulting protection.

4. Deliver a list of suitable barrier types with characteristics.

5. Identify the criteria for the evaluation of the designs and gate type selection.

6. Make a stable and strong technical design that integrates with the landscape.

7. Provide construction method and planning recommendations.

8. Estimate Cost.
1.4.3 Research questions

From the objectives, the research questions can be defined. The answers to the different subquestions will together give the answer to the main question. The subquestions will be answered along the different chapters of the report.

Main question

What will be a feasible design for a dynamic barrier to protect the Houston Ship Channel, based on stakeholder needs, cost-benefits and evaluation criteria?

Subquestions

1. Which previous research and design works have been considered already?
2. What are the local design and boundary conditions?
3. What are the requirements and needs of local parties and stakeholders?
4. What is the best location for the Houston Ship Channel Barrier, giving the most protection for the least structure?
5. What additional protection measures will be required?
6. What alternative solutions are possible for the gate(s) and which gate type will be most ideal following the local design- and boundary conditions?

1.5 Methodology

1.5.1 Design methodology

The design of a large flood protection structure will be executed following the traditional steps: Description of the system; Definition of objectives; Analysis of the system, requirements and boundary conditions; Development of alternatives; Investigation of impacts and costs; And the selection of an alternative to be designed in detail.

In order to end with a good design, the level of detail will increase during the work. Starting with an overall design for the entire protection system containing levees, an environmental "barrier" and the location for the dynamic navigational barrier, in which a global design will be made for the different structures following design rules of thumb. Subsequently the elements of the navigational barrier at the Houston Ship Channel will be designed in a more detailed level, consisting of the cross-section of the gate, the barrier foundation, the abutments and the installations and systems required for movement of the gate. The final design will be evaluated after which some improvements will be made. Finally conclusions will be made and a number of recommendations will be given.

1.5.2 Structure of the report

In this first chapter the location of interest has been introduced, as well has the relevance of the problem by showing some historical events and recent research. Subsequently, the problem was defined giving the project objectives and the list of research questions.

In Chapter 2 the first three subquestions will be answered in order to comply to the first specific objective. The next three subquestions will be answered in the remaining chapters, converging to an optimized solution based on available data, by meeting all specific objectives.

This thesis will mainly focus on the design of a movable storm surge barrier at the entrance of the Houston Ship Channel. Due to the variety of previous and ongoing research a number of different
protection concepts has been introduced, which may be combined with the Houston Ship Channel Barrier. An important assumption will be whether or not an ‘Ike-Dike’ or ‘Coastal Spine’ will be present, since this structure will have great impact on behavior of the bay during storm conditions and thus on the requirements for a barrier at the channel entrance. This issue will be addressed in Chapter 3, after which the design of the global protection system will be presented.

After an analysis of the possible gate types for the navigational barrier in Chapter 4, one gate type is selected based on the results of a Multi Criteria Analysis. Chapter 5 presents the technical design of the selected gate type. After a brief evaluation of the resulting barrier design, an alternative design will be proposed in Chapter 6 introducing some adjustments to improve the gate, based on the criteria of Chapter 4. Finally, Chapter 7 presents the conclusions and recommendations. The appendices contain extensive descriptions of the design steps, computations and characteristics of the system. The elements covered by the different chapters are listed in the framework presented in Figure 1.7.

Figure 1.7: Framework of the report structure.
Chapter 2

System Description and Boundary Conditions

2.1 Introduction

The previous chapter introduced the problem of frequent flooding at the Houston Ship Channel and the objectives of a possible solution for this problem in combination with the needs of the different stakeholders. To be able to design a proper solution, all local boundary conditions, design conditions and barrier requirements have to be known and have been collected in this chapter.

First a geographical overview of the Houston Ship Channel and its surroundings will be given in Section 2.2 Geography, defining the area of interest for this thesis. For this area, the existing infrastructure and important structures will be described in Section 2.3 Infrastructure. These two sections are followed by an analysis of the Geology in Section 2.4. In this section the geotechnical information is presented, considering the different soil layers and the soil characteristics, which will be used to determine the design conditions of the foundation in Chapter 5. Additionally, the problem of land subsidence is introduced in this section. The coastal processes are described in Section 2.5. First the normal conditions are given, which are present the majority of the time, considering the climate, relative sea level rise, tidal movement and the bathymetry. Subsequently the situation during storm conditions will be described, giving the design conditions for surge, wind and waves. These design conditions will be used in Chapter 5 as input for the design loads acting on the barrier.

In Section 2.6 the risk-based flood protection approach is introduced. The volume of residents, businesses and environment at risk will be defined. As well are the protection level and the probability of damage determined. At the end of this section a simplified Cost-Benefit Analysis is executed, evaluating the feasibility of the construction of a new flood protection system.

At the end of this chapter a summary is made of all data obtained in the different sections. A table is given with the input variables, required to perform calculations, and a diagram with all needs and requirements is presented. This data is used as input for the following Chapters, giving boundary conditions, design conditions and barrier requirements.

2.2 Geography

Texas lies in the southern most portion of the United States, adjacent to the Mexican border at the South-West and to the Gulf of Mexico in the South-East. It is the largest state in the U.S. and has a coastline of about 700 km. The state Texas is subdivided in 254 counties, which are smaller areas in which multiple cities can be located. This thesis focuses on the three counties located in Southeast Texas: Galveston, Chambers and Harris. The State of Texas and the different counties
around Houston can be seen in Figure 2.1. The City of Houston and the Houston Ship Channel industrial complex are located in Harris County. The location of the HSC relative to Galveston Bay and the City of Houston is presented in Figure 2.3.

Figure 2.1: Locations of Texas and Houston on the left. At the right the surrounding counties and contributing drainage area to the surge gate. Obtained from Christian et al. (2014).

2.2.1 Greater Houston and Houston Ship Channel Description

Galveston Bay and the Houston Ship Channel are surrounded by several cities, of which the City of Houston is the largest with its 2 million residents. For the purpose of this thesis, the study area will be called ‘Greater Houston’ and has a population to about 6 million residents. It is the most densely populated area of Texas and the number of residents is still increasing rapidly with a growth of about 10 percent during the last decade, making it a nationally-important population and economic center (www.city data.com, 2013; GHPRD, 2014).

Given its strategic position to the petroleum fields in the Gulf of Mexico and Southern US, Houston accommodates the main petrochemical industry of Northern America. The Port of Houston is the busiest port of the United States in terms of foreign tonnage and the thirteenth-busiest of the world according to the Port PHA (2010). It supports the largest petrochemical complex in the US (second largest in the world) (Brody, 2014).

The Houston Ship Channel industrial complex provides over 50,000 direct jobs and the potential damage for floods is about 100 billion USD (SSPEED, 2013). Its economic importance is mainly due to its three major industries: refining, petrochemicals and natural gas processing. About 85 percent of Houston exports are petroleum related.

2.2.2 Galveston Bay

Galveston Bay is separated from the Gulf of Mexico by Galveston Island and Bolivar Peninsula in the southeast. It is fed by multiple rivers and bayous. Galveston Bay can be characterized as a so-called micro-tidal wind dominated estuary (Jonkman et al., 2013). The wet surface area of Galveston Bay is about 1554 km² and it has a total shoreline length of 374 km. The average depth is 3 m and is not varying very much except for the navigation channel, which is about 20 m deep and has a pretty straight alignment through the bay from the channel entrance at Morgan’s Point, in the northwest, to the bay inlet called Bolivar Roads in the south.

Galveston Bay provides valuable ecosystem services, recreational activities and commercial fishing. The authority of the Clean Water Quality Act of 1987 established the Galveston Bay National Estuary
Program (GBNEP) to develop a Comprehensive Conservation and Management Plan (CCMP) for Galveston Bay, which has been described by Whittington et al. [1994]. The purpose of this CCMP is to address threats to the Bay resulting from pollution, development and overuse in order to secure and maintain the ecologic and economic value of the bay. In order to valuate the economic value of the Galveston Bay, Whittington et al. [1994] made estimates of the annual economic value of changes in the environmental quality of Galveston Bay, as presented in Table 2.1.


<table>
<thead>
<tr>
<th>Change in environmental quality</th>
<th>1993 Dollars</th>
<th>2015 Dollars</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Due to implementation of the Management Plan: (Use and nonuse values)</td>
<td>$100 – 150 Million/yr.</td>
<td>$166 – 250 Million/yr.</td>
</tr>
<tr>
<td>2. Losses incurred if the existing environmental quality deteriorated greatly: (Use values)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Recreational Fishing:</td>
<td>$75 – 150 Million/yr.</td>
<td>$125 – 250 Million/yr.</td>
</tr>
<tr>
<td>Boating:</td>
<td>$25 – 50 Million/yr.</td>
<td>$40 – 85 Million/yr.</td>
</tr>
<tr>
<td>Other: (hiking, camping, swimming, bird-watching, etc.)</td>
<td>$15 – 50 Million/yr.</td>
<td>$25 – 85 Million/yr.</td>
</tr>
<tr>
<td>2.2 Commercial fishing</td>
<td>$1 – 2 Million/yr.</td>
<td>$2 – 3 Million/yr.</td>
</tr>
</tbody>
</table>

2.2.3 Climate

Houston lies near the Gulf of Mexico, at about 30° North, and thus has a humid and semitropical climate in summertime, with an average annual temperature of about 20.5 °C. Winters are usually mild, although temperatures below zero do occur sometimes. Average temperature, wind and coastal processes are generally very soft and calm in this region, a graph of the monthly average values is presented in Figure 2.2.

Figure 2.2: Left: Houston climate graph, from USCD [2014]; Right: All year winds in Galveston Bay. Period of record: 31 December 1972 - 07 June 2014. From ISU.
2.2.4 Freshwater Inflow

The Houston Ship Channel is contained within the San Jacinto River Watershed, a large watershed that exceeds the boundaries of Harris County. With a total catchment area of approximately 11,655 m$^2$ consisting of multiple small rivers, bayous and creeks, it drains much of the Greater Houston. From all upstream imports three tributaries can be distinguished that work as the major input of water for the Houston Ship Channel and thus have the most influence on the downstream discharge: Brays Bayou, Buffalo Bayou, and the San Jacinto River, from west to east respectively. Each of these three upstream tributaries is the outflow of an upstream reservoir, which will be the system’s upstream
boundary conditions [Christian et al., 2014]. Control structures are installed at the reservoirs to control the downstream river discharge, provide water supply and enable substantial flood storage.

The upstream reservoir controlling Brays Bayou is George Bush Park, in the west of Houston. It flows mainly through residential areas and passes downtown Houston in the south. It finally confluenes with the bigger Buffalo Bayou. Its drainage area has a size of 127 square miles ($329 \text{ km}^2$) and the population within this area counts 0.7 million residents.

The Buffalo Bayou is the confluence of the outflows of two reservoirs, namely Barker Reservoir and Addicks Reservoir. It flows straight through downtown Houston where it confluences with Whiteoak Bayou. The part downstream of this confluence, from downtown Houston to the Houston Ship Channel, is called Buffalo Bayou as well. On the eastern side of Houston the Brays Bayou joins the Buffalo Bayou as well after which everything flows to the Houston Ship Channel by the name of Buffalo Bayou. The size of its drainage area is about 102 square miles ($265 \text{ km}^2$) and within this area half a million residents can be found. The mean discharge of the Buffalo Bayou is $1,550 \text{ cubic feet per second}$, or $44 \text{ m}^3/\text{s}$. (USGS, 2014)

Both described bayous feed the Houston Ship Channel from the west, but from the north it receives the discharge of the San Jacinto River. The San Jacinto River originates from Lake Houston in the northeast of Houston, well outside of Harris County, and flows through much of the eastern edge of Harris County after which it confluences with the two bayous to form the Houston Ship Channel. The San Jacinto River watershed consists of multiple smaller bayous as well and together with the river itself it contains about 310 miles ($500 \text{ km}$) of open streams. It is responsible for the drainage of a total area of about 487 square miles ($1,261 \text{ km}^2$) in Harris County. Within the watershed the estimated population is about 150,000 in Harris County. (HCFCD, 2010)

Figure 2.4: Upstream hydraulic input of Houston Ship Channel: (a) Buffalo Bayou Watershed; (b) Brays Bayou Watershed; and (c) San Jacinto Watershed. (HCFCD, 2010)

**Precipitation**

The different bayous in the area are very much dependent on the amount of rainfall. Observations and measurements from the US Geological Survey (USGS) give the daily mean values for discharge and gauge heights. Figure 2.5 shows some characteristics of the bayous present at the Greater Houston (USGS, 2014).
River discharge

As presented before, the three upstream drainage areas which are most important for the discharge of the Houston Ship Channel are the Buffalo Bayou, the Brays bayou and the San Jacinto Watershed. A rainfall of 30 cm in 24 hours is on average $3.5 \times 10^{-6} \text{ m/s}$. This gives for the above described drainage areas (keeping in mind that the discharge of some more small Bayous have to be added) an increase in amount of water to be drained and an increased river discharge of:

$$\Delta Q_{\text{Buffalo}} = 3.5 \times 10^{-6} \times 265 \times 10^6 = 925 \text{ m}^3/\text{s}$$  \hspace{1cm} (2.1)

$$\Delta Q_{\text{Brays}} = 3.5 \times 10^{-6} \times 330 \times 10^6 = 1,150 \text{ m}^3/\text{s}$$  \hspace{1cm} (2.2)

$$\Delta Q_{\text{San Jacinto}} = 3.5 \times 10^{-6} \times 1,260 \times 10^6 = 4,375 \text{ m}^3/\text{s}$$  \hspace{1cm} (2.3)

Where equations (2.1), (2.2) and (2.3) represent the amounts of extra discharge for the Buffalo bayou, Brays bayou and San Jacinto River respectively. The total increase in discharge at the Houston Ship Channel due to these three watersheds then will be 6,450 $\text{ m}^3/\text{s}$. This however, would only be the case for an instant discharge increase when the rainfall increases, in reality some more aspects are of importance like landuse, soil type and topography (Christian et al., 2014) which may give a lag in discharge increase and a somewhat smaller peak discharge.

Figure 2.6: Model calibration for discharge [$\text{ m}^3/\text{s}$] at: (a) Greens Bayou; (b) San Jacinto River; (c) Brays Bayou. (Christian et al., 2014)
To determine the total increased river runoff at the Houston Ship Channel during a hurricane some more aspects have to be taken into account like the delay between moment of rainfall and actual discharge increase and the amount of water that is incorporated into the soil, additionally some more watersheds than the above described contribute to the total runoff. Figure 2.5 gives an overview of watersheds in Harris County, the size of the drainage areas and the population living in each watershed. Note that the results of the above performed hand calculations for the the 100-year rainfall discharges are very equal to the discharges given by HCFCD (2007) in Figure 2.5.

For the design river discharge a combination will be made of the data given in Figures 2.5 and 2.6 and the results of equations 2.1, 2.2 and 2.3. Figure 2.5 gives the 100-year rainfall events for the different watersheds. However, these yearly maximums for the different watersheds do not have to occur at the same time. In order to end up with a barrier design that can withstand all varying combinations of storm surge and river runoff, the design will be challenged number of situations in Chapter 5, using the following river discharges at the Houston Ship Channel:

Situation 1: \[ Q_1 = 2,500 \text{ m}^3/\text{s} \] (8.0 m surge)
Situation 2: \[ Q_2 = 5,000 \text{ m}^3/\text{s} \] (6.0 m surge)
Situation 3: \[ Q_{1/100y} = 25,000 \text{ m}^3/\text{s} \] (1.0 m surge)

Water level elevation at protected side

The wet area protected by a possible barrier is about 40 km². A 1/100 years rainfall event causes a river discharge \( Q_{1/100y} \) of 25,000 m³/s, which in case of a closed barrier results in a water level elevation at the wet protected area of \( 6.25 \cdot 10^{-4} \text{ m/s} \) or \( 2.25 \text{ m/hour} \). This water level elevates very rapidly which makes it impossible to close the barrier for rainfall events of this order of magnitude. The other two rainfall situations \( Q_1 \) and \( Q_2 \) cause water level elevations of 0.625 m/s and 1.250 m/s respectively.

Figure 2.17 shows that the water level elevates somewhat slower than computed above, but it does confirm that the water surface elevation upstream a closed barrier is significant: 2.5 to 3.0 m in 10 hours. To further reduce this elevation a shorter closing or opening duration is required. If this cannot be optimized, the installation of discharge pumps at the barrier may be required to drain the river runoff to the bay, this requirement will be determined by the amount of damage corresponding to a 3.0 m water level elevation which will be further discussed in Sections 2.3, 2.6.3 and 2.4. Strength and stability requirements of the gates will follow from occurring loads for the decisive surge scenario, these checks will be done in Section 5.3.1.

Discussion

The amount of rainfall in the catchment area, the resulting increased river discharge and the size of the storage area in case of a closed barrier, are all very important characteristics for the determination of the requirements on the opening and closure time and reliability of the barrier. If the river discharge is big and the storage area is small, the water level at the protected side of the barrier will elevate very rapidly. In that case the function of the barrier can not be fulfilled. However, the river discharges can be regulated by regulation structures at the described reservoirs, limiting the amount of water in the bayous and therefore limiting the discharge in the Houston Ship Channel. For that reason, the subject considering water level elevations and a negative head will not be an issue for this thesis as long as the operation system of the gate is reliable (Hershfield, 1961; Moore and Riley, 2002; Chen, 2006; CBS/AP, 2012; USDA, 2014; Torres et al., 2015).
2.3 Infrastructure

2.3.1 Land use

As a result of the population and economical growth, mentioned in Chapter [1], the city its residential and industrial areas rapidly expanded to the more flood vulnerable areas, which results in a flood risk increase. Additionally the drainage capacity decreases due to the reduction of green areas and the flow resistance decreases due to the fact that natural flows and meandering rivers get replaced by straighter and shorter human constructed canals. This phenomenon is called floodplain encroachment, presented in Figure 2.8. All above described aspects of an increasing land use will lead to an increase in the probability of flooding and, due to the increasing amount of constructed capital and development density, to an increase in consequences as well. To conclude: due to the encroachment phenomenon the flood risk increases more or less quadratic (see Section 2.6). According to [Brody 2014] the current development pattern can be described as an outward expanding low-density development, which can amplify flooding and increases flood damage. Land use around the Houston Ship Channel is presented in Figure 2.7, where Figure 2.8 shows only the central part of the Houston Ship Channel. Additionally Figure 2.7 indicates some important structures and developed areas.

Figure 2.7: Location of the Houston Ship Channel and other important infrastructure.

Figure 2.8: Encroachment due to increase of industrial areas and reduction of floodplains around the Houston Ship Channel near Texas Terminal, northwest from Pasadena: (a) 1953; (b) 1995; and (c) 2014.
The current development pattern is likely to continue in the near future. Brody (2014) predicts especially for the residential areas a huge increase of volume, as presented in Figure 2.9. With the growing economy, an increase can be expected as well for the Houston Ship Channel Industrial Complex. The amount in US Dollars of this increase is very hard to predict, although qualitatively the statement can be made that the need of flood protection will increase rapidly in time. (Waterworth 2014)

2.3.2 The Houston Ship Channel

The Houston Ship Channel, located in the northwest of Galveston Bay (Figures 2.3 and 2.7), hosts more than 50 ships and 300 barge movements each day and is one of the busiest ports of the Americas. In 2005, the channel was deepened and widened in order to accommodate more and bigger ships. The new main channel has a project depth of 45 feet (14 m) and a width of 530 feet (162 m) and on each side of the channel barge lanes are added. The total width of the channel is 1000 feet. A schematic illustration of the dimensions of the Houston Ship Channel (HSC) its cross section is given in Figure 2.10. In order to maintain a sufficient depth in the channel, dredging works are being executed almost any time. (HOGANSAC 2006).
Structures

The HSC does not contain many crossings with structures except for the Fred Hartman Bridge at the HSC entrance, which was constructed in the 90’s to replace the Baytown Tunnel in order to increase the depth clearance. Since the removal of the Baytown Tunnel in 1999, the channel has no depth limitations due to tunnels anymore. The vertical navigation clearance under the Fred Hartman Bridge is 54 m and the main span is 381 m (Structurae, 2015). Its exact location is indicated Figure 2.7.

2.3.3 Vessels

The vessel dimensions, and thus the types of the vessels entering the Houston Ship Channel, are important for the barrier design as well. Under normal conditions a barrier is opened to allow vessels to navigate through the channel. The minimum width and depth will follow from these dimensions using design rules from Verheij et al. (2008) and Ligteren (2009).

A big increase in size of vessels can be seen during the last century. When making a design for a lifetime of 100 years these developments have to be taken into account, to enable larger vessels to use the channel as well and to allow the port to increase its capacity in the future. Some vessel types which are common in the bigger ports around the world will be discussed.

Suezmax

As the name says, the Suezmax tanker is the biggest type of tanker that is able to pass the Suez Canal in Egypt. After the deepening and widening of the Houston Ship Channel in 2005, as described in Section 2.3.2 (HOGANSAC, 2006, 2011), the channel is able to accommodate the Suezmax tanker. The dimensions of this type of vessel are given in Figure 2.11. The dimensions of the Length, Width and Draught are 899 feet (274 m), 164 feet (50 m) and 66 feet (20 m) respectively.

New Panamax

As a result of current construction works to increase the capacity of the locks at the Panama Canal a new vessel type will have the maximum dimensions to pass the Panama Canal. Obviously, this new limit vessel is called the New Panamax. The dimensions of this type of vessel are given in Figure 2.11. Length, Width and Draught are 366 m, 49 m and 15 m respectively. Recent designs for a barrier in the Galveston Bay entrance Bolivar Roads used this New Panamax as the design vessel (Jonkman et al., 2013, 2014).

Very Large Crude Tanker (VLCC)

The VLCC tanker is one of the biggest crude tankers in the world. It is the biggest vessel able to enter the Malacca Strait in Southeast Asia. At the moment this type of vessel is too big to enter the Houston Ship Channel. However, to respond on possible growth of the vessels in the next 100 years, the barrier will be designed on the dimensions of this vessel. The dimensions of the Length, Width and Draft are 1312 feet (400 m), 193 feet (60 m) and 82 feet (25 m) respectively (Network, 2014).
The Houston Galveston Navigation Safety Advisory Committee (HOGANSAC) is closely involved in matters relating to the transit of vessels and products at the Houston Ship Channel. According to their folders [HOGANSAC (2006)] and [HOGANSAC (2011)], the HSC is currently capable to accommodate the Suezmax tanker. As long as the tanker is not fully loaded and the draught is limited, a 1-lane traffic of the Suezmax tanker would be possible with the current channel-width of 162 m, according to the design rules of [PIANC (1997)]. This is a much smaller type of vessel than some other oil terminals like Rotterdam or Singapore can accommodate. However, as introduced in Section 2.3.2, the channel width is currently limited by the Fred Hartman Bridge, which main span is 381 m wide and the associated clearance height is 54 m. These dimensions limit the maximum channel capacity to a 2-lane traffic for the Suezmax or a 1-lane traffic for the VLCC vessel, as computed in Appendix D.

Keeping in mind that the barrier will be constructed for the next 100 years, an increase in channel capacity during its lifetime may be expected. According to the design rules the capacity may be increased to a 1-lane VLCC or a 2-lane Suezmax tanker by increasing the channel depth to 27 m or 22 m respectively. Since the minimum channel depth at the moment is only 14 m it is not expected in the next 100 years that a depth of 27 m will be feasible, therefore the design vessel and traffic intensity used to determine the navigational clearance of the barrier is a 2-lane traffic of Suezmax tankers. As presented in Section 3.6, the resulting channel dimensions are large enough to accommodate a 1-lane traffic of the VLLC tanker in the future, as long as the tanker is not fully loaded and it draught does not exceed 20 m. [HOGANSAC (2006) 2011] [PIANC (1997)] [F&O 2014].
2.4 Geology

2.4.1 Surface Elevation

The areas around Galveston Bay are wide plains with, especially on the eastern side, low surface elevations of less than 2 m above Mean Sea Level (MSL). Around the banks of the ship channel and Buffalo Bayou the surface quickly takes a height of about 2-5 m above MSL. Most petrochemical industries are located at these surface heights as well, where the city’s business center (Downtown Houston) has been established at a height of about 10-15 m above MSL. As presented in Figure 2.12, the surface area around Galveston Bay is pretty flat. Moving further away from the bay the surface height slowly increases, were the 20 m above MSL line is about 60 to 90 km away from the coastline. At time of construction, Houston Ship Channel Facilities are required to be protected against a FEMA 1/100 storm surge [FEMA, 2014].

According to TSHA [2010] the Houston area has over 150 active faults, including the Long Point-Eureka Heights fault which runs through Downtown Houston. In the past these faults can have caused some earthquakes which may have led to the current surface heights. However, there have been no significant historically recorded earthquakes in Houston. In some communities southeast from Houston land is sinking due to groundwater extraction (see Section 2.4.3), which may be associated with slip around the faults. This slippage, however, is slow and not considered an earthquake since it is not causing any significant seismic waves.

2.4.2 Geo-technical Information

The Houston-Galveston region’s geology developed from river deposits, where available sediment originates from erosion of the Rocky Mountains in the state Colorado, in the center of the United States, containing sands, clays and organic materials. The surface layers consist of these porous materials, which are suitable for rice farming but can cause problematic foundation issues due to their bad bearing capacities [TSHA, 2010]. Soil stratigraphy consists of up to 60 feet of loose sand and soft clay, including dredged materials, overlying stiff to very stiff clays and medium-dense to dense sands to a depth of at least 300 feet. Table 2.2 and Figure 2.13 give the different layers of soil for the Houston Ship Channel entrance.
Table 2.2: Soil layer classification and strength properties. Obtained from Penland and Cibor (2013).

<table>
<thead>
<tr>
<th>Depth [feet]</th>
<th>[m]</th>
<th>Classification</th>
<th>Relative density</th>
<th>Undrained shear strength [kips/sqft]</th>
<th>Density [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>+10 to -40</td>
<td>+3 to -13</td>
<td>Loose sands and soft clays</td>
<td></td>
<td>0.25</td>
<td>12</td>
</tr>
<tr>
<td>-40 to -60</td>
<td>-13 to -20</td>
<td>Medium dense sands</td>
<td>50 %</td>
<td>0.75</td>
<td>36</td>
</tr>
<tr>
<td>-60 to -150</td>
<td>-20 to -50</td>
<td>Stiff to very stiff clays</td>
<td></td>
<td>0.75</td>
<td>36</td>
</tr>
<tr>
<td>-150 to -220</td>
<td>-50 to -70</td>
<td>Dense sands</td>
<td>85 %</td>
<td>1.00</td>
<td>48</td>
</tr>
<tr>
<td>&gt; -220</td>
<td>&gt; -70</td>
<td>Very stiff clays</td>
<td></td>
<td>1.00</td>
<td>48</td>
</tr>
</tbody>
</table>

According to the engineers of McClelland (1985), the layer of dense sands at a depth of approximately -50 m to -60 m has an excellent bearing capacity. For closed end pipes with a diameter of 0.61 m the soil is able to resist a force of about 6,600 kN, which is approximately equal to 5,600 kN/m² or 5.6 MPa.

2.4.3 Land Subsidence

As introduced in Section 2.5.1, the Houston area has been adversely affected by land subsidence, mainly caused by ground-water pumping but also by oil and gas extraction from subsurface reservoirs. The results are settlements, or surface degradation, which may increase the frequency of flooding. Near the coast, the net result of land subsidence directly contributes to the relative sea level rise (Section 2.5.1). The additional relative sea level rise due to land subsidence is about 1 inch (2.54 cm) per year according to Coplin and Galloway (1999). This is quite big, especially when looking to the total lifetime of 100 years. Recently the ground-water extraction is restricted (Sebastian et al., 2014), which resulted in a significant reduction in the land subsidence.

The Harris-Galveston Subsidence District (HGSD) is responsible for the regulation of ground-water extraction. Additionally, HGSD collected measurements and observations of subsidence since the beginning of the 20th century. As presented in Figure 2.14(a) the total subsidence over the last
century for the Houston Ship Channel Complex and the center of Houston is 6 to 10 feet, so about 2 to 3 m. Graphs (b) and (c) in the same Figure show that the total subsidence over the last 40 years is much smaller, where (b) Pasadena and (c) Baytown present results of subsidence measurements just west and east of the channel entrance respectively. This confirms the statement of Sebastian et al. (2014) that the amount of subsidence is expected to be much lower in the future due to the restriction of ground-water extraction in the area. However, for both graphs an increase of subsidence can be seen from 2010 onwards, which can not be explained and therefore increases the uncertainty of subsidence for the next 100 years. (HGSD, 2014; Zilkoski et al., 2014; Kasmarek and Houston, 2014)

Anyway, the observed subsidence for the last 40 years is about 1 foot (0.3 m) and is therefore much lower than the first half of the 20th century. The purpose of estimating the land subsidence for the design life time of the structure is estimate the total relative sea level rise (Section 2.5.1) for this time period, which is the sum of the local sea level rise and the land subsidence. The distributions of both variables are very uncertain and in the literature different estimated values can be found. As presented in Section 2.5.1 the average global sea level rise was 2 mm/year during the 20th century and this is accelerating (Douglas, 1991; Willis et al., 2010). To deal with uncertainties in predictions for both global sea level rise and land subsidence in the future, only the relative sea level rise will be used. For the complete design lifetime a value of 1.0 m will be used, being the sum of Global Sea Level Rise (∼ 20 cm) and Land Subsidence (∼ 80 cm) for 100 years based on literature.

2.5 Coastal processes

2.5.1 Normal conditions

Tide

As mentioned in Section 2.2.2 Galveston Bay is a so-called micro-tidal wind dominated estuary. The tide is of a mixed type with a small tidal range of approximately 2-3 feet (0.3 m), according to NOAA (2014) and Jonkman et al. (2013). Figure 2.15 presents the mean range and diurnal range at the Morgan’s point (the ship channel entrance) of 1.13 and 1.31 feet respectively. The graph also shows that the average mean water level at Morgan’s Point is about MSL −0.2 m (NOAA, 2014).
Relative sea level rise

As a result of global warming a global sea level rise can be found. The rate of sea level rise has accelerated considerably relative to the pre-industrial era. Over the past century, an average rate of about 2 \( \text{mm/year} \) is found, according to \cite{Douglas1991}. For the next century several predictions have been made, both empirical and computational, resulting in rates up to 5 \( \text{mm/year} \) \cite{WillisEtAl2010}, additionally an acceleration in the global sea level rise can be observed from sea level data of the 20th century \cite{ChurchAndWhite2006}. For Galveston Island, the Center for Operational Oceanographic Products and Services has predicted a sea level trend even bigger: 6.5 \( \text{mm/year} \) \cite{NOAA2014}. In Galveston Bay the changes of the water surface by a global sea level rise are less significant. Although for these areas a great relative sea level rise can be observed: 6-10 \( \text{feet} \) (2-3 \( \text{m} \)) during the 20th century, as determined by the Houston-Galveston Subsidence District \cite{ZilkoskiEtAl2014, KasmarekAndHouston2014}. This is mainly the result of subsidence due to ground water extraction, covered in Section 2.4.3.

Bathymetry

Just offshore of the Texas coast the Louisiana Texas (LATEX) Shelf can be found, resulting in a wide shallow foreshore. As stated in Section 1.2 this shallow foreshore is very sensitive to hurricane winds, which dramatically increase the water volume on top of the shelf and thus without any outside protection measures the water volume in Galveston Bay. This effect may result in the forerunner phenomenon: A first surge peak arriving at the Houston Ship Channel a few hours before the major peak. As observed during Hurricane Ike, this phenomenon greatly increases the peak surge and the size of the area that gets flooded. The surge characteristics will be further described in Section 2.5.2.

The water depth in the Galveston Bay is, except for the ship channel, everywhere between 2 and 4 meters, which makes it a pretty shallow Bay. Wind set-up is very important due to this small depth, making the entire length of the Galveston Bay important for the fetch. The distance from the basin entrance at Galveston to the beginning of the Houston Ship Channel at Morgan’s Point is about 40 \( \text{km} \), which will be the design value for the fetch.

After deepening and widening the Houston Ship Channel in 2005, it has a project depth of 45 \( \text{feet} \) (14 \( \text{m} \)) and a width of 530 feet which is about 160 \( \text{m} \) \cite{HOGANSAC2006}. Due to the small dimensions relative to the dimensions of the Bay, an average water depth of 2.5 \( \text{m} \) will be used for the calculations of bathymetry-related surge levels.
2.5.2 Storm conditions

Design storm event

The origin of Hurricanes and storms relevant for the coast of Texas is the Gulf of Mexico. While increasing in strength, these storms approach the coast from the sea, so from the southeast. As presented in Figure 1.2, Section 1.2, all major hurricanes relevant for the Galveston region come from the south or southeast, but after making landfall the tracks can shift to all northern directions when things will be less predictable. It may even be possible that the direction changes completely while moving over land. A storm might even return to the coast, as seen for Tropical Storm Allison in 2001, see Section 1.2 and Figure 1.3.

The hurricane storm surge behavior in Galveston Bay has been characterized by Sebastian et al. (2014), using the SWAN + ADCIRC model. Results show that water surface elevations present in Galveston Bay are dominated by the counterclockwise hurricane winds. The article shows that a small increase in wind speeds results in a magnified increase in surge elevations; 23 percent elevation increase for a 15 percent increase in wind speed, as found for modelling the conditions of Hurricane Ike.

Resulting from the study of historic storm events and different simulations, it has been found that an ‘Increased-Ike’ is a valid hurricane as an input for the design storm conditions, for the design of a barrier with a sufficient design life time. The character of Hurricane Ike will be used, but with increased wind speeds and a worst case landfall location (Sebastian et al. (2014), Christian et al. (2014)).

Besides the character, Sebastian et al. (2014) studied the effects of shifting the track of the storm westwards, which resulted in higher levels of surge. A maximum surge level was found in case of Hurricane Ike making landfall at the southwestern end of Galveston Island, as presented in Figure 2.16. As explained above, this worst case scenario will be used as input for the design conditions.
Figure 2.17: Computed stage hydrograph immediately upstream of proposed surge gate for (a) Hurricane Ike surge; and (b) synthetic surge variations of Hurricane Ike for 15 and 30 percent increases in wind speed (Christian et al., 2014). The rising water level for a closed gate can be related to river runoff.

Christian et al. (2014) developed a hydraulic model in order to investigate the effectiveness of a storm surge barrier at the Houston Ship Channel, which is able to simulate combined rainfall and surge inundation scenarios. Hurricane Ike (2008) has been used as input for the model. A comparison between observed discharges at the upstream rivers during hurricane Ike and modeled discharges shows the reliability of the model, although a typical error of about 10-20 percent can be seen as well.

The model has been used to determine the behavior of the water surface upstream of a storm surge barrier at the HSC, in case of closed gates and hurricane Ike conditions. The results are presented in Figure 2.17, note that the mentioned ‘proposed gate location’ is at the Fred Hartman bridge (see Sections 2.3.2 and 3.3), where according to Christian et al. (2014) the observed surge level was 3.80 m corresponding to the left graph of Figure 2.17 as well, while the maximum observed surge level at the HSC was 5.0 m so the effect of a gate may be different at that location.

As introduced in section 1.2, extensive rainfall during tropical storms can cause huge problems in the Houston-Galveston region. The 100-year rainfall discharges for each bayou in Harris County is given in Figure 2.5 presented in Section 2.2. For the region the 100 year rainfall event is over 30 cm in 24 hours. The trend however, is that weaker storms produce more rain and less surge so a 100 year rainfall event is not expected to occur in combination with a major hurricane. Recent research by Torres et al. (2015) showed that hurricanes which made landfall near the Bolivar Roads Inlet and San Luis Pass Inlet (near Galveston Island) led to higher rainfalls captured within the San Jacinto River Basin. More specifically, for these same landfall locations much of the rainfall fell west of Houston, but downstream of Barker Reservoir, producing peak rainfall-runoffs that were within 24 hours of timing separation with peak surge for that given hurricane scenario. This is an important finding since it highlights the importance of Operation Reliability of the Houston Ship Channel Barrier. If opening of the gate fails after surge mitigation, upstream flooding will be the result within 24 hours.
Surge

Hurricane events in the past 100 years, have revealed the vulnerability of the system around the Houston Ship Channel. As a result of Hurricane Ike (2008) the volume of water in the Galveston Bay nearly doubled (Sebastian et al., 2014). According to Dawson et al. (2013), the physical processes in storm surge are:

- Driving forces: wind, pressure, Coriolis, tides
- Complex coastlines, rough domains, complex boundary conditions, highly varying bathymetry and topography
- Inundation and recession
- Interaction of water with structures and vegetation
- Bottom friction

The bathymetry has already been mentioned in the previous section, the other relevant processes from remaining items in the list will be discussed in the following section.

Design value

While Hurricane Ike could be characterized as a Category 2 hurricane, the occurrence of hurricanes of Category 4 can be expected in this area as well, resulting in wind speeds of about 30 percent higher then during Ike. According to Sebastian et al. (2014) the surge level at the ship channel entrance would be increased to a maximum if Ike would have made landfall at the southwestern end of Galveston Island as presented in Section 2.5.2. A worst-case scenario would occur in case of both the shifted track and the category 4 hurricane, which will result in a 25 feet (8 m) surge at the Houston Ship Channel entrance. For the purpose of this thesis, a shifted Ike +30% will be associated with a 1/1,000 years event.

Table 2.3: Different model predictions for storm surge levels at the Houston Ship Channel entrance, near Morgan’s Point. (Stoeten, 2013; Whalin et al., 2014; FEMA, 2014).

<table>
<thead>
<tr>
<th>Model Return period:</th>
<th>1/100</th>
<th>1/100</th>
<th>1/100</th>
<th>1/1,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surge level:</td>
<td>4.3 m</td>
<td>4.2 m</td>
<td>6.1 m</td>
<td>8.0 m</td>
</tr>
</tbody>
</table>

Using a 1 in 100 year return period, the design surge level may be somewhat lower than 8 m. Besides, since multiple research and design studies are being executed, it may be possible to have additional protections in- or outside of the Galveston bay which will reduce the maximum surge level at the Houston Ship Channel entrance near Morgan’s Point (Whalin et al., 2014). According to the Federal Emergency Management Agency (FEMA, 2014) the 1 in 100 year surge level in the Houston Ship Channel is 20 feet (6.1 m) in the original state, where for this return period according to Stoeten (2013) the surge level will be 4.3 m and a model by Jackson State University resulted in 4.2 m water level elevation. Table 2.3 shows the surge level predictions for the different models, where the event called ‘Shifted Ike +30%’ will be considered to be an 1/1000 years event. These values will be further discussed at the end of this Section.
Wind conditions

The Saffir/Simpson Hurricane Winds Scale, presented in Table 2.4, enables one to categorize the different hurricanes. It makes a distinction between 5 different categories, based on the hurricane’s sustained wind speed. A tropical storm can be called a hurricane when reaching wind speeds of about 120 km/h. In that case a category 1 hurricane has arisen, which can grow further to a stronger hurricane with higher wind velocities, see Table 2.4.

It has been made clear that hurricane surge is a function of maximum storm wind speeds. However, wind speed alone cannot reliably describe surge. Hurricanes Ike (2008) and Katrina (2005) showed that the size of the hurricane plays an important role in surge generation as well. According to Irish et al. (2008) the storm size is especially important for storms making landfall at coasts with a very mild slope, like the Texas coast as described in Section 2.5.1 phrase Bathymetry. This is due to the fact that the wind field may be so large that it causes a significant buildup of water at the coast. There are also studies showing that the hurricane’s forward speed is a significant parameter in determining the storm surge (Rego and Li, 2009), where the peak surge heights increase for an increased forward speed of the hurricane. Rego and Li (2009) concluded that varying the forward motion may account for variations in flooded volumes equivalent to a hurricane of one category higher or lower on the Saffir/Simpson wind scale.

<table>
<thead>
<tr>
<th>Scale number</th>
<th>Winds, max for 1 minute</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category</td>
<td>[mph]</td>
</tr>
<tr>
<td>1</td>
<td>74-95</td>
</tr>
<tr>
<td>2</td>
<td>96-110</td>
</tr>
<tr>
<td>3</td>
<td>111-130</td>
</tr>
<tr>
<td>4</td>
<td>131-155</td>
</tr>
<tr>
<td>5</td>
<td>&gt;155</td>
</tr>
</tbody>
</table>

Table 2.4: The Saffir/Simpson Hurricane Winds Scale [Irish et al., 2008; Blake and Gibney, 2011].

Major hurricanes are from categories 3, 4 and 5, where wind speeds of more than 50 m/s or 180 km/h occur and devastating damages can be expected. Hurricane Ike (2008) was a category 2 hurricane, with wind speeds up to 100 mph. Ike did not grow any further then category 2, but its size grew while approaching the coast of Texas to a very large hurricane. The diameter of its tropical
storm winds covered a total of 425 miles (680 km) taken from northeast to southwest. When making landfall near Galveston, maximum sustained winds of 110 mph were observed. (National Weather Service 2014)

(Blake and Gibney 2011) gives return periods for the whole coast of the United States. The estimated return period in years for a hurricane (Category 1 or more) passing within 50 miles of Houston is 9 years, the estimated return period of a major hurricane (Category 3 or more) for this region is 25 years.

Design wind

Hurricane Ike caused huge problems and damages in the Houston Ship Channel area, even though it was just a category 2 storm. An important observation which can explain this is that for less intense hurricanes the amount of rainfall is usually larger (CBS/AP 2012) in combination with the bathymetry (the forerunner phenomenon), the size of the hurricane and its propagation speed, as described in the previous sections.

According to Sebastian et al. (2014) and Christian et al. (2014) the worst-case scenario will take place for conditions of Hurricane Ike with a shifted track as presented in Section 2.5.2. This would increase the maximum wind speeds by 30 percent to approximately 229 km/h which can be categorized as a category 4 storm. Elaborating on their findings, this value will be used for the design wind force.

Air pressure

Besides extreme wind velocities and intense rainfall (Sections 2.5.2 and 2.2.4 respectively), a significant drop of the air pressure is strongly related to hurricanes as well. The atmospheric pressure under normal conditions at sea level fluctuates between 1010 and 1015 millibars (NOAA 2014), in case of a hurricane this pressure can drop to less then 990 millibars (Li et al. 2012). During the Galveston Hurricane (1900) the pressure fell to 28.44" (inches of mercury) (Roth, 2009), which is about 963 millibars. Pugh (1996) has given a relation between local atmospheric pressure variations \( \Delta P_A \) and local sea level rise \( \Delta \zeta \), as presented in the equations below.

\[
\Delta \zeta = \frac{-\Delta P_A}{\rho g}
\]

(2.4)

Taking values of sea-water density \( \rho = 1026 \text{ kg/m}^3 \) and \( g = 9.81 \text{ m/s}^2 \) this will lead to the following linear relation, where \( \zeta \) is in centimeters and \( \Delta P_A \) is in millibars.

\[
\Delta \zeta = -0.994 \Delta P_A
\]

(2.5)

For a local pressure drop of about 30 millibars the resulting local elevation of the water surface is about 0.30 m or 1 foot, which is significant. However, surge due to wind set-up is more or less zero in the eye of a hurricane, where maxima for surge due to an air pressure drop can be found near the water surface. Surge due to wind set-up is much larger and therefore no additional water surface elevation will be added to the maximum surge levels found for winds. A visualisation of the pressure variations for hurricane Ike (2008) is presented in Figure 2.19.
Waves

Based on 30 years of hourly measurements of significant wave heights in the Gulf of Mexico (Panchang and Li, 2006) created a Gumbel distribution for some extreme wave values, resulting in significant wave heights just offshore Galveston Bay of 8.66 m, 9.29 m and 10.74 m for return periods of 1/50, 1/100 and 1/500 respectively. For 97 percent of the time the significant wave height was in the range 0-3 m, the maximum value within these 30 years was 5.80 m, observed on March the 12th, 1993.

With the use of the Brettschneider theory (Holthuijsen, 2009) the significant wave height $H_s$ and peak period $T_p$ within Galveston Bay can be computed, giving the set of Equations (2.6). Input variables are the local Fetch $F = 40$ km and Depth $d$, which is 22 to 30 m at the Houston Ship Channel. With gravity $g = 9.81$ m/s$^2$ and Category 4 Hurricane wind velocities of 65 m/s, this results in a maximum significant wave height $H_s = 10.2$ m and peak period $T_p = 11.7$ s.

$$H_s = \frac{\tilde{H} u_{wind}}{g} \tilde{H} = 0.283 \cdot \tanh(0.53d^{0.75}) \cdot \tanh\left(\frac{0.0125\tilde{F}^{0.42}}{\tanh(0.53d^{0.75})}\right) \tilde{d} = \frac{gd}{u_{wind}}$$ \hspace{1cm} (2.6)

$$T_p = \frac{\tilde{P} u_{wind}}{g} \tilde{T} = 7.54 \cdot \tanh(0.833d^{0.375}) \cdot \tanh\left(\frac{0.077\tilde{F}^{0.25}}{\tanh(0.833d^{0.375})}\right) \tilde{F} = \frac{gF}{u_{wind}}$$ \hspace{1cm} (2.7)
Waves within Galveston bay however, are fetch and depth limited (Jin et al., 2010), with the maximum deep water wave height in the region is about 7.0 m. Since the bay is normally pretty shallow, local water depths at the channel entrance can double and even triple due to hurricane wind set-up. Since the waves are depth limited, Jin et al. (2010) found a more or less linear relation between storm surge and maximum wave heights, as presented in Figure 2.20. The design wave is related to a Category 4 storm event with 6.0 m surge: $H_{\text{max}} = 5.0$ m and $T_p \approx 8$ s. For a 1/100 years surge event of 8.0 m, the approximate design wave height and peak period are 6.0 m and 9.0 s respectively.

![Figure 2.20: Wave characteristics as function of storm surge per hurricane category at San Luis Pass Bridge, from Jin et al. (2010): a) Maximum wave height $H_{\text{max}}$; b) Maximum wave period $T_{\text{max}}$.](image)

Discussion

As mentioned before there is a variety in data, predictions and models resulting in different values for a 1/100 year storm surge event. To validate these results a simple hand calculation will be carried out using a wind induced set-up calculation given by Holthuijsen (2009). The wind induced set-up can be determined by the balance between wind and bottom stress on the water and a hydrostatic force, expressed by the following formula: (Holthuijsen, 2009)

$$\Delta \eta = \frac{\tau_w F}{\rho g h_0} = \frac{\rho_a C_w W^2 F}{\rho g h_0}$$ (2.8)

In which:
- $\eta$ Water level $m$
- $\tau_w$ Wind stress $kg/s^{-2}$
- $\rho$ Water density $1026 kg/m^{-3}$
- $\rho_a$ Air density $1.2 kg/m^{-3}$
- $C_w$ Wind coefficient $0.001 [-]$
- $W$ Wind velocity $65 m/s$
- $F$ Fetch $40 km$
- $h_0$ Averaged bay depth $3.0 m$
- $g$ Gravitational acceleration $9.81 m/s^{-2}$

The above defined variables result in a wind set-up of about 6.7 m at Morgan’s Point, the Ship Channel entrance. This result is somewhat bigger than FEMA (2014) has presented for a 1/100 year storm event, apparently a larger bottom stress should be used. Despite this, the result confirms the order of magnitude of the storm surge levels given by Sebastian et al. (2014) and the Federal Emergency Management Agency (FEMA) in FEMA (2014), see Table 2.5. As described in Section 2.6, the Houston Ship Channel Barrier and the accompanying protection system will be designed to resist an 1/1,000 years storm event. For this event, a surge level of $MSL + 8.0 m$ will be used.
Table 2.5: Comparison of surge level at Morgan’s Point, for different studies and return periods.

<table>
<thead>
<tr>
<th>Return period [1/years]</th>
<th>Various studies</th>
<th>Other computations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/100</td>
<td>4.3 m</td>
<td>4.2 m</td>
</tr>
<tr>
<td>1/500</td>
<td>5.0 m</td>
<td></td>
</tr>
<tr>
<td>1/1,000</td>
<td>5.5 m</td>
<td></td>
</tr>
<tr>
<td>1/10,000</td>
<td>6.4 m</td>
<td></td>
</tr>
</tbody>
</table>

In the discussion of the presence of flood protection structures at Galveston Island, the Bolivar Peninsula and Bolivar Roads (The Coastal Spine concept (Merrell, 2012)) Stoeten (2013) states that the wind set-up contributes for about 50 percent to the surge and that the other 50 percent is due to the increase of water volume in the Bay. Following the model results of Stoeten (2013), the surge reduction at Morgan’s point is about 1 to 2 m in case of a Coastal Spine. This issue will be further elaborated in Chapter 3, Section 3.2.

2.6 Flood risk

It has been made clear that the Houston-Galveston region is a risky area, with the occurrence of hurricanes in a dense population with a lot of industrial activities and big environmental value. The definition of flood risk will be introduced in this section in combination with a risk-based design approach.

The probability of hurricanes in the area has been described in Sections 1.2 and 2.5.2. The consequences of floods will be quantified in this section in terms of potential damage, for society, business and environment. As introduced in the next subsection, the multiplication of probability of occurrence and consequences result in the potential risk. This risk will be described as amount of costs per year at the end of this section. Additionally the risk reduction by the construction of a barrier will be determined and evaluated by the use of a brief cost-benefit analysis and the computation of the Net Present Value (NPV).

2.6.1 Basics of Risk-Based Flood Protection

Several definitions of risk exist, but in this document risk is considered as the product of flood frequency (probability of flooding per year) and the impact of flooding: the consequences. In Section 2.6.3 several types of impacts have been introduced, namely impact on people, environment and businesses.

\[
Risk = \text{Probability} \times \text{Consequence}
\]  

(2.9)

In order to evaluate the costs of risk reduction, or an increase in safety level, risk should be expressed in $/year or as a net present value in USD, which can be established by expressing the consequences of flooding in USD. This enables the comparison between costs and benefits of a certain risk reduction. Additionally an optimal safety level can be found for which the total expected costs are minimal, as shown by the right image in Figure 2.21 (Jonkman et al. 2014).
The in Figure 2.21 presented Probability of Failure is the area for which the design load is larger than the strength of the structure of system. Where both strength related variables as load related variables have a mean value and a standard deviation. Considering strength related variables this can be related to (small) fluctuations during the production or construction process. An example of a random distribution of a load related variable is the wave impact, where the waves impacting on a structure during (for example) a 100 year period are probabilistic distributed and will therefore cause different impacts. Both distributions can be presented in a graph, showing a statistical interference where the load bigger is then the strength and thus failure may occur: left image in Figure 2.21. When this interference area is big the strength has to be increased, which can be done by for example using stronger materials or improving the production process resulting in a smaller standard deviation so smaller strength fluctuations, this will always increase the costs. If a structure is properly designed, the interference area is rather small, resulting in an acceptable probability of failure for a certain time period (design life time) and an economical optimum: right image in Figure 2.21 (Danzig, 1956).

2.6.2 Probability of Occurrence

According to Roth (2009) the average frequency of hurricanes in along any 80 km segment of the Texas coast is about 1 in 6 years. Assuming that significant consequences can only be found for hurricanes with a storm surge of more than 4 m, a probability of 1/26 years that damage occurs will be used for an unprotected Houston Ship Channel, as presented in Figure 2.22 (Keim et al., 2006). This probability of occurrence can be decreased significantly by a flood protection system.

A proposed protection system consisting of dunes, dikes, breakwaters and barriers, has to be designed for a certain reference period in which it is expected not to have bigger loads (storm surge, waves, etc.) than the design load. This design load follows from a design storm, which is related to a return period. A common used return period in the Netherlands is 1,000 years to 10,000 years for the most vulnerable areas, so the design loads are related to the conditions associated to a 1 in 10,000 years storm. For the design of the Houston Ship Channel Barrier, a 1/1,000 years storm event will be used as design storm, of which the related design conditions have been presented in Section 2.5.2. Using a design lifetime of 100 years, the probability that the design storm is exceeded will result from the Poisson equation: (Verhagen and Schiereck, 2012)

\[
P = 1 - \exp(-fT) \tag{2.10}
\]

In which:
- \(P\) Probability that a certain event occurs during the complete lifetime of the structure.
- \(f\) Frequency of the event, the number of events per year.
- \(T\) Relevant time period, the lifetime of the structure.
The damage reduction due to the construction of a storm surge barrier system can now be determined with the new probability of occurrence. During the lifetime of 100 years the probability that the design values are exceeded is \( P_f = 0.095 \), which is associated to system failure. The resulting damage reduction will be determined at the end of this section. First the associated consequences in terms of damage (USD) have to be determined.

The effectiveness of a barrier in terms of damage reduction has been modeled by Christian et al. (2014) for several situations to investigate the feasibility of a storm surge barrier, simulating combined rainfall and surge inundation scenarios. Figure 2.24 shows the reduction of the areas flooded in case of a barrier compared to the situation without a barrier for 8.0 m surge. Figure 2.23 shows the exact values of the associated reduction. The proposed HSC protection system offers a uniform protective benefit to all industries behind the HSC gate in the event of hurricane landfall into Galveston Bay. Obviously, the construction of a barrier will reduce the impacts dramatically. When comparing Figures 2.23 and 2.24 it can be seen that the amount of flooded area is not reduced that much by the gate, but the amount of inundated oil tanks is greatly reduced, resulting in a huge reduction of the impact. (Bedient, 2014)

### 2.6.3 Consequences of Flooding

Damage costs continue to increase since Houston is one of most flood prone cities in U.S. accommodating critical infrastructure concerns with huge industry along the HSC, Texas Med Center, water-related facilities, and power systems. In this section a distinction between the impact on people, environment and business will be made.

The effectiveness of a barrier in terms of damage reduction has been modeled by Christian et al. (2014) for several situations to investigate the feasibility of a storm surge barrier, simulating combined rainfall and surge inundation scenarios. Figure 2.24 shows the reduction of the areas flooded in case of a barrier compared to the situation without a barrier for 8.0 m surge. Figure 2.23 shows the exact values of the associated reduction. The proposed HSC protection system offers a uniform protective benefit to all industries behind the HSC gate in the event of hurricane landfall into Galveston Bay. Obviously, the construction of a barrier will reduce the impacts dramatically. When comparing Figures 2.23 and 2.24 it can be seen that the amount of flooded area is not reduced that much by the gate, but the amount of inundated oil tanks is greatly reduced, resulting in a huge reduction of the impact. (Bedient, 2014)

### Table: Floodplain Area Reduction

<table>
<thead>
<tr>
<th>Scenarios</th>
<th>Floodplain area</th>
<th>Area reduction</th>
<th>WSELs</th>
<th>WSEL reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without gate (km²)</td>
<td>With gate (km²)</td>
<td>Reduction (km²)</td>
<td>Reduction (%)</td>
</tr>
<tr>
<td>Ike surge (original)</td>
<td>102</td>
<td>95</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>Ike surge + 15%</td>
<td>111</td>
<td>98</td>
<td>13</td>
<td>12</td>
</tr>
<tr>
<td>Ike surge + 30%</td>
<td>127</td>
<td>102</td>
<td>25</td>
<td>19</td>
</tr>
</tbody>
</table>

Figure 2.23: Floodplain area reduction and Water Surface Elevation Level (WSEL) reduction for three different scenarios. (Christian et al., 2014)
Social consequences

A population growth of 28 percent can be seen over the last decade, when the population of the Houston-Galveston region changed from 4.8 million residents in the year 2000 to 6.1 in 2010. More important, people especially settle at the low lying, flood prone areas close to the Bay or the different rivers and bayous. And last but not least, especially social vulnerable groups are moving to the low intensity development areas, resulting in an increasing group of less protected people in more exposed areas (Brody, 2014). According to Read (2012) the estimated clearance time, which is the time required to evacuate the population in the flood prone evacuation zones, increased from 24 hours to 48 hours as a result of the population growth of the past 20 years. The population is expected to increase to 8.8 million in 2035, and the number of residents living in the evacuation zones to 1.6 million.

The social impact of a hurricane will follow from the loss of life, so the number of deaths. Then the controversial question of the value of a human life plays a role. There are different estimates for this value (Appelbaum, 2011; Viscusi, 2005) made by different US agencies and departments in a range from $6.1 million (Transportation Department, 2010) to $9.1 million (Environmental Agency, 2010) according to Appelbaum (2011). According to Viscusi (2005), the median value of statistical life from 30 U.S. studies was $7.0 million. To deal with possible uncertainties and the inflation for the next 100 years, the inflation rate for the USD fluctuated between 0.1% and 4.1% per year from 2004-2014 (USIC, 2015), a human life will be valued worth $10 million in the estimation of the consequences of a flood.

Environmental consequences

The waters of Galveston Bay provide nursery and spawning grounds for large amounts of marine life and additionally there is over 55 thousand hectares (550 km²) of coastal wetlands surrounding the lake, serving as habitat for the many harvest-able species found in the estuary. The bay is important for both commercial and recreational fisheries, oyster harvests, ecotourism, bird watching, hunting and boating. As presented in Table 2.1 in Section 2.2.2, is the total estimated annual economic value of Galveston Bay about $360 to $670 Million per year (Whittington et al., 1994).

The environmental damage of a hurricane is mainly secondary: Due to inundation of oil tanks with flood waters oil spills can be found at the Houston Ship Channel Industrial Complex, which severely damage natural areas around Galveston Bay. By closing off the Houston Ship Channel with a barrier and levee alignment, the size of the bay that gets effected by any oil spill gets greatly reduced. Even...
though the probability of oil spills will be greatly reduced due to a barrier, the occurring oil spill can be kept in the channel, preventing the oil from damaging the major part of Galveston Bay and allowing cleaning activities to concentrate at a much smaller area. It can be concluded that a storm surge barrier will reduce both the probability and the environmental consequences.

In order to quantify the environmental damages in terms of money, the example of Katrina is used, for which 330 Million USD was payed as compensation for oil damage, resulting in about 4 billion USD potential damage. This is in line with the results of the storm surge damage assessment of the Houston Ship Channel, presented by Rifai and Burleson (2013). See Appendix B Section B.3 for more information about tank inundation and recent oil spills. The direct costs of an oil spill of 0.3 million barrels are about 20 Million USD, using the Nasdaq Crude Oil Price (BMDS, 2015).

\section*{Industrial consequences}

The Port of Houston is one of the busiest ports of the world, responsible for over a million jobs throughout Texas, according to the Port of Houston Authority (PHA, 2010). The Port of Houston is the busiest export port in the nation generating $180 billion of total economic impact annually and more than $4.5 billion in state and local tax revenues. Its industrial complex contains the largest petrochemical complex in the United States and is of national strategic importance (PHA, 2010).

A report by the Perryman Group for the Independent Insurance Agents of Texas found that a Katrina-like storm that hit the Port of Houston area would result in the losses of $73 billion of gross product, over 800,000 permanent jobs and about $2.5 billion in state revenue. So in case of not functioning of the port of Houston, due to flooding, the state will loose $6.85 Million per day on average. This is, of course, in case of not functioning of the complete port, which will be only the case for the worst case scenario. Nevertheless, the costs per day will be significant for any flood.

\subsection*{2.6.4 Risk Reduction}

It is in the Federal interest and the State interest to protect both the economic vitality and the valuable ecosystem services provided by the Bay region its built infrastructure and natural resources. Hurricane Ike showed that both are vulnerable to storm surge. Where Hurricane Ike resulted in more then $30 billion in damages, it could have been much worse if Ike had made landfall 40 km more to the west. The storm damages for this worst case scenario would have been about $100 billion, with over a thousand deaths instead of the 84 deaths associated with Ike. As a result of the in this section described expected growth of consequences in combination with estimates presented by the existing literature, a number of 100 deaths will be used. Resulting in an estimated amount of consequences in USD for a 1/1,000 years storm event is 105 Billion USD.

\begin{align*}
\text{Social damages:} & \quad 1.0 \cdot 10^9 \\
\text{Environmental damages:} & \quad 4.0 \cdot 10^9 \\
\text{Industrial damages:} & \quad 100 \cdot 10^9 \quad \rightarrow \quad 105 \cdot 10^9
\end{align*}

A project is feasible as long as the cost-benefit analysis (CBA) turns out to be positive: The benefits are larger than the costs. Benefits are expressed in risk reduction, where costs follow from the investment costs and the yearly maintenance and operational costs. To have an infinite time horizon the interest rate is reduced, by which the Risk Reduction is divided, resulting in the Net Present Value (NPV) criterion:

\begin{equation}
I < \left( \frac{P_{I,0} - P_{I,N}}{r} \right) D \quad (2.11)
\end{equation}
In which:  
- \( P_{f,0} \) Failure probability without any protection.  
- \( P_{f,N} \) Failure probability after the construction of a flood protection system.  
- \( I \) Investment costs.  
- \( D \) Resulting damage.  
- \( r \) Interest rate, a value of 5 percent (0.05) will be used.

However, due to the sloping groundsurface at the Houston Ship Channel, more damage can be found for bigger storms. Jonkman and Schweckendiek (2015) gives the relationship between varying flood probabilities and economic damages. It can be determined by the use of the graphs presented in Figure 2.25. The annual risk or expected damage is the area below the curve of Figure 2.25-(b). The value of a barrier with protection level \( P_f \), expressed as risk reduction, is the area reduction resulting from structure. This risk reduction is expressed in USD per year, to obtain the NPV it only needs to be divided by the interest rate \( r \).

Figure 2.25: Relationship between return period and damage (a) and probability of exceedance and damage (b). The area below curve (b), left from a probability of exceedance value, represents the expected annual damage for that specific protection level.

The complete procedure is described in Appendix B, the resulting values are presented in Table 2.6, where the protection level of 1/26 year is the current situation. It can be seen that for all presented measures the benefit is much more than the assumed investment costs, the conclusion can be made that the project is feasible. Table 2.6 shows for the 1/10,000 year protection level the biggest Net Present Value. However, to adopt on local design trends\(^1\) the design of the Houston Ship Channel Barrier will be continued using a the protection level of 1/1,000 year.

<table>
<thead>
<tr>
<th>Protection level (Return period ( T ))</th>
<th>Surge ([m])</th>
<th>Frequency ( P_f ) [(y^{-1})]</th>
<th>Damage (Billion USD)</th>
<th>Investment (Billion USD)</th>
<th>NPV of risk reduction (Billion USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26 year:</td>
<td>4.0</td>
<td>0.038</td>
<td>$30</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>100 year:</td>
<td>6.0</td>
<td>0.010</td>
<td>$90</td>
<td>$6.0</td>
<td>$33.9</td>
</tr>
<tr>
<td>1,000 year:</td>
<td>8.0</td>
<td>0.001</td>
<td>$105</td>
<td>$7.0</td>
<td>$51.5</td>
</tr>
<tr>
<td>10,000 year:</td>
<td>9.0</td>
<td>0.0001</td>
<td>$110</td>
<td>$7.5</td>
<td>$53.4</td>
</tr>
</tbody>
</table>

\(^1\)In the US most structures are designed for smaller return periods. Frequencies of 1/100-1/500 year are commonly used (FEMA 2014).
2.7 Summary

In this chapter’s final section a summary will be made from the described characteristics. Before the design conditions can be determined in Section 2.7.2 a few assumptions have to be made. Most of these assumptions are already made along the different sections of this chapter, so this will be just a recap.

2.7.1 Assumptions and Boundary Conditions

- Design life time: 100 years.
  A design life time of 100 years is chosen for the barrier. This is a common lifetime for a civil structure. During its lifetime it should retain all occurring loads, the level of accepted damages has to be determined, obviously total failure is not accepted but some repairable damage might be accepted resulting in a more economical solution. During the 100 year life time the structure should withstand the design storm which has a certain return period, in this case 1000 years.

- Return period of design storm: 1/1,000 years
  A return period of 1000 years will be used, which is much smaller than used in the Netherlands, where 1/10,000 years is used for areas with a high level of floodrisk. However, typical design levels in the United States are 1/100 or 1/500, where 1/500 years is used for so-called “critical facilities” like hospitals. A 1/1,000 years design event gives an increase in safety level, which is in terms of yearly costs due to flood risk a more economical solution, as presented Table 2.6 of Section 2.6. For a lifetime of 100 years the probability of occurrence of the 1/1,000 years design storm is 0.095 (9.5%) according to the Poisson equation (Verhagen and Schiereck, 2012). This is proportional to the probability that during the structure’s design lifetime the design load gets exceeded.

- Other structures: Ike Dike and/or Bolivar Roads barrier (Coastal-Spine).
  A lot of different work has been done already, as well is the proposal for a dike system and a storm surge barrier at the bay inlet ‘Bolivar Roads’. This barrier will reduce the inlet of water from the Gulf of Mexico due to storm conditions resulting in a reduced water volume in the bay. Obviously, the design surge level will be reduced significantly by this assumption but still there will be a significant residual surge due to local wind set-up at the HSC entrance. For the design of the HSC Barrier only the presence of the Ike Dike is assumed, without a barrier at Bolivar Roads. This issue will be discussed in Section 3.2.

- Assumed relative sea level rise for the total life time: 1.0 m.
  This is the sum of the predicted global sea level rise and the local land subsidence, as a total for 100 years.

- Assumed design vessel: Suezmax, with a 2-lane traffic.
  The minimum channel dimensions are related to the navigational requirements of the Suezmax oil tankers. The resulting required channel dimensions are presented in Section 3.6 which are big enough to even accommodate a 1-lane traffic of larger tankers (VLCC) in the future. It is therefore expected that the barrier dimensions will not limit navigation during the complete design life time of 100 years.

---

2 A 1/1,000 year storm event is taken as design storm for the load combination. Overtopping and wave run-up of the levee system however, are quantified for a 1/100 year storm event as well. This explains why for the surge level and wave height the values are given for both return periods in the Table presented at the next page.
2.7.2 Design Conditions

Table 2.7 presents a summary of design conditions which relevant for the HSC Barrier. Right of the design values the section is presented in which the condition is described.

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Infrastructure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Existing main-channel dimensions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Channel depth</td>
<td>( h_{channel} )</td>
<td>15.0</td>
<td>m</td>
<td>2.3.2</td>
</tr>
<tr>
<td>Channel width</td>
<td>( W_{channel} )</td>
<td>200</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Bank slopes</td>
<td>( i_{channel} )</td>
<td>1:1</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>1.2 Navigational height limitation</td>
<td>( h_{max} )</td>
<td>54.0</td>
<td>m</td>
<td>2.3.2</td>
</tr>
<tr>
<td>1.3 Navigational width limitation</td>
<td>( W_{max} )</td>
<td>381</td>
<td>m</td>
<td>2.3.2</td>
</tr>
<tr>
<td>1.4 Design vessels:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Suezmax (2-lane traffic)</td>
<td></td>
<td></td>
<td></td>
<td>2.3.3</td>
</tr>
<tr>
<td>Length</td>
<td>( L_{suezmax} )</td>
<td>285</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Width</td>
<td>( W_{suezmax} )</td>
<td>50.0</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>Draught</td>
<td>( D_{suezmax} )</td>
<td>20.0</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>2. Geology</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Depth of supporting layer</td>
<td>( d )</td>
<td>50-60</td>
<td>m</td>
<td>2.4.2</td>
</tr>
<tr>
<td>2.2 Shear at shaft</td>
<td>( \tau_{s,gem} )</td>
<td>24</td>
<td>kN/m²</td>
<td>2.4.2</td>
</tr>
<tr>
<td>2.3 Point load capacity</td>
<td>( \sigma_{point} )</td>
<td>5,600</td>
<td>kN/m²</td>
<td>2.4.2</td>
</tr>
<tr>
<td>2.4 Subsidence (Order)</td>
<td>( \Delta h_{subs} )</td>
<td>~</td>
<td>cm/year</td>
<td>2.4.3</td>
</tr>
<tr>
<td>3. Coastal processes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.1 Global sea level rise</td>
<td>( GSLR )</td>
<td>2.0</td>
<td>mm/year</td>
<td>2.5.1</td>
</tr>
<tr>
<td>3.2 Tide (+MSL)</td>
<td>( \Delta h_{tide} )</td>
<td>0.1</td>
<td>m</td>
<td>2.5.1</td>
</tr>
<tr>
<td>3.3 Average water level</td>
<td>( MSL )</td>
<td>-0.2</td>
<td>m</td>
<td>2.5.1</td>
</tr>
<tr>
<td>3.4 Average bay depth</td>
<td>( h_0 )</td>
<td>3.0</td>
<td>m</td>
<td>2.5.1</td>
</tr>
<tr>
<td>3.5 Fetch</td>
<td>( F )</td>
<td>40</td>
<td>km</td>
<td>2.5.1</td>
</tr>
<tr>
<td>3.6 Surge</td>
<td>( S )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/100 year</td>
<td>( S_{1/100} )</td>
<td>6.0</td>
<td>m</td>
<td>2.5.2</td>
</tr>
<tr>
<td>1/1,000 year</td>
<td>( S_{1/1000} )</td>
<td>8.0</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>3.7 Wind</td>
<td>( u_{wind} )</td>
<td>65</td>
<td>m/s</td>
<td>2.5.2</td>
</tr>
<tr>
<td>3.8 Atmospheric pressure drop</td>
<td>( P_A )</td>
<td>-30</td>
<td>milibar</td>
<td>2.5.2</td>
</tr>
<tr>
<td>3.9 Rainfall</td>
<td>( P )</td>
<td>0.727</td>
<td>mm/s</td>
<td>2.5.2</td>
</tr>
<tr>
<td>Runoff situation 1: (8.0 m surge)</td>
<td>( Q_1 )</td>
<td>2,500</td>
<td>m³/s</td>
<td>2.5.2</td>
</tr>
<tr>
<td>Runoff situation 2: (6.0 m surge)</td>
<td>( Q_2 )</td>
<td>5,000</td>
<td>m³/s</td>
<td>2.5.2</td>
</tr>
<tr>
<td>Runoff situation 3: (0.0 m surge)</td>
<td>( Q_{1/100y} )</td>
<td>25,000</td>
<td>m³/s</td>
<td></td>
</tr>
<tr>
<td>3.10 Wind waves</td>
<td>( H_s )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/100 year</td>
<td>( T_p = 8 \text{ s} )</td>
<td>5.0</td>
<td>m</td>
<td>2.5.2</td>
</tr>
<tr>
<td>1/1,000 year</td>
<td>( T_p = 9 \text{ s} )</td>
<td>6.0</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>4. Flood Risk</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.1 Residents in watershed</td>
<td>( N_{people} )</td>
<td>1,000,000</td>
<td></td>
<td>2.6.3</td>
</tr>
<tr>
<td>4.2 Daily turnover</td>
<td>( E_{day} )</td>
<td>$6,850,000</td>
<td>tax revenues</td>
<td>2.6.3</td>
</tr>
<tr>
<td>4.3 Damage costs</td>
<td>( D_{1/26} )</td>
<td>$30</td>
<td>Billion USD</td>
<td>2.6.3</td>
</tr>
<tr>
<td>4.4 Damage costs</td>
<td>( D_{1/1000} )</td>
<td>$105</td>
<td>Billion USD</td>
<td>2.6.3</td>
</tr>
<tr>
<td>4.5 Net Present Value of risk reduction</td>
<td>( NPV )</td>
<td>$51.5</td>
<td>Billion USD</td>
<td>2.6.3</td>
</tr>
<tr>
<td>4.6 Floodplain reduction</td>
<td>( \Delta A )</td>
<td>25</td>
<td>km²</td>
<td>2.6.3</td>
</tr>
</tbody>
</table>
2.7.3 Requirements and Design Criteria

In order to achieve a barrier design that serves people, business and environment by retaining storm surge, some additional functional requirements are listed in this section. Key issues in the design are, obviously, hydraulics and structural aspects: the transfer of hydraulic loads to the foundation; the foundation, including bed and bank protection; the choice of gate types; and the mechanism used to move the gates. Next to these key issues, some relevant aspects with respect to durability, environmental impact, maintainability, reliability and constructability are added.

These 8 main requirements will serve as the main criteria on which the different possible barrier types will be evaluated. The evaluation will be done by a Multi Criteria Analysis (MCA), where the importance of each criteria will differ by the use of weight factors. The listed sub-requirements will serve as the sub-criteria. See Chapter 4 for the different gate designs and the selection of the most suitable type.

Flood Risk Mitigation

- Reliability of closure and opening
  Risk of flooding due to failure of the system. Requires reliable mechanisms, frequent testing and careful operation. This considers the complexity of the movement system and experience with existing barriers.

- Strength and stability
  Risk of flooding due to failure of the structure, which might be due to insufficient strength and stability of the gates, the foundations or the bed protections. Requires a suitable gate type, use of reliable materials and frequent inspection or maintenance.

Hydraulics

- Amount of storm water passing by leakage or overtopping should be limited
  Requires sufficient height of the barrier, the application of seals or the construction of additional structures like breakwaters.

- Vibrations and oscillations due to flow or waves have to be prevented
  Gate requires adequate dynamic properties.

- Closure induced translatory waves and differentia head should be acceptable
  This requires a suitable closure time and opening time.

- Discharge of river flow
  Requires an adequate flow opening.

Structure

- Foundation
  Should be strong enough but also not too heavy in order to limit settlements. So the required size of the foundation should be limited.

- Bed and bank protections
  Should be strong enough, but the same restrictions for settlements. So the structure should not require a massive amount of protection.

- Operation system
  A very complex system is more expensive and requires very specific knowledge while it might be more risky as well. A simple operation system/mechanism is favorable.
Navigation
- The barrier should allow the passage of ships
  The width and depth of the navigation opening have to be sufficient for the dimensions of the relevant design vessels (2-lane Suezmax traffic), for now but also in the future.
- Flow velocities
  In order to enable navigation the flow velocities during normal conditions should be limited. As well should turbulent structures be avoided.

Environment
- Passage of fish
  Requires a wet cross-section, limited flow velocities and the salinity gradient are of importance.
- Allow tidal flow
  Sill level, wet cross-section, effect of density differences.

Constructability
- Experience
  It is desired to have reference projects, which can function as an example. This requires the selection of an existing barrier type.
- Equipment
  In order to reduce costs, a barrier type which does not require special (custom-made) equipment is desired.
- Material
  For the reduction of both costs and environmental impact, the amount of materials required to construct the protection system is tend to be minimized. This has influence on the protection system alignment and the dimensions of the different elements.

Maintenance
- Accessibility
  Requires possibilities for a (temporary) dry dock for the gate(s).
- Material choice
  Protection against corrosion and rot is required.

Aesthetics
- Landscape integration
  Structures that integrate in the landscape are favorable, like dunes and grass dikes instead of concrete sea walls.
- Spatial quality
  The attraction of the gate type, aspects like slenderness and shape.
- Recreational quality
  The added value by attracting tourists or adding natural areas.
Chapter 3

Global Protection System Design

3.1 Introduction

The previous chapters have sequentially described the main problem, the characteristics of the location and the boundary and design conditions. The global design of the complete protection system will be made in this chapter, following the developed list of requirements and defined protection level. As presented in Figure 3.1 the global design will contain a number of sections, each with specific functions. These sections will designed subsequently in this Chapter.

First the possible presence of other structures is discussed in Section 3.2 setting the final boundary conditions. This is followed by the definition of the final barrier location and levee alignment in Section 3.3. The three different levee sections, presented in Figure 3.1, will be further designed in Section 3.4. In Section 3.5 a global design of the structure covered by the Environmental Section is presented. See Figure 3.1 for a presentation of the different elements the protection system consists of.

![Figure 3.1: The different sections the Houston Ship Channel Protection System, consisting of 3 Levee sections, an Environmental section and a Navigational section.](image)

The part of the protection system where the main focus will lie on is the Navigational Section, containing the Houston Ship Channel Barrier. This movable barrier will be designed in detail in Chapter 5. In Section 3.6 a start is made on the design of this barrier by defining the required dimensions of the navigational cross section. Subsequently the loads on the channel banks will be
determined, as a result of waves generated by shipping, which is followed by the design of a channel bank protection. This section concludes with the definition of the hydraulic load acting on a possible barrier, when the channel is closed off. This load will be used in Chapter 5.

### 3.2 The Presence of Other Structures

Due to recent Hurricanes and an increase in knowledge of the risks for the area, the described issue has received a lot of attention already. A great number of possible interventions has already been introduced in Section 1.1 and presented in Figure 1.1. The different structures can be categorized, resulting in the four protection strategies listed below and presented in Figure 3.2.

- **Do nothing**
  The above described reactive approach.

- **Protection on the outside**
  This is the coastal spine concept, a combination of dikes and barrier(s) at the seaside of the bay.

- **Protection on the inside**
  Construct dikes and barrier(s) on the inside of the bay, protecting just the vulnerable or valuable areas. This concept includes the Houston Ship Channel Barrier.

- **Multiple lines of defense**
  A combination of a coastal spine on the outside of the bay and a barrier at the HSC entrance.

![Figure 3.2: Sketch of strategic alternatives and interventions for flood risk reduction at Galveston Bay: Protection by a Coastal Spine (left) and by the Ship Channel Barrier with additional protection works (right).](image-url)

Where the SSPEED Center, an initiative of Rice University, is closely involved in the Houston Ship Channel Barrier-concept (protection on the inside) another concept is the Coastal Spine, developed by Texas A&M University at Galveston. The idea of the Coastal Spine-concept is to move the protection structures to the outside of the bay, containing sea dikes at Galveston Island and the Bolivar Peninsula.
and a movable barrier at Bolivar Roads, the bay entrance, resulting in a 'Coastal Spine' protecting both Galveston Bay and the Houston Ship Channel.

However recent studies show that due to residual surge the surge reduction at the HSC is not sufficient, if only the Coastal Spine concept is realized, the presence of a coastal spine may greatly influence the design conditions for a movable barrier at the Houston Ship Channel. A combination of the two concepts may be the best solution. See Figure 3.2 for the 2 different concepts.

The design conditions for the situation without any additional structures will be like the current situation as presented in Chapter 2 Analysis. So the 1/1,000 storm surge event will result at Morgan’s Point in a water level elevation due to surge of 8.0 m. However, the surge level will be reduced in case of a coastal spine, thanks to the protection against: (1) The huge increase in volume of the bay due to inflow; and (2) the forerunner-effect observed during Ike (2008), introduced in Section 1.2. Still there will be a significant surge due to wind induced set-up at the north end of the Bay, called residual surge, but as explained this will be somewhat smaller.

Stoeten (2013) developed a simplified probabilistic hurricane surge model for Galveston Bay which enables one to simulate the behavior of Galveston Bay. The model simulations confirm that both inflow and wind set-up are responsible for the surge within the Bay. Stoeten (2013) has found a contribution of the local wind set-up of about 50 percent of the total water level elevation, depending on the storm track and intensity. The surge level reduction due to the coastal spine can be 2 to 3 m at the Ship Channel Entrance (Jonkman et al., 2014). Brody and Atoba (2014) investigated the economic impact of the Coastal Spine for a number of different storms, they concluded that the damage reductions for Harris County will be about 80 to 60 percent.

Recent studies have showed that a Bolivar Roads Barrier may not result in the most effective solution. It will be expected in the near future that only an Ike Dike will be realized, but no complete Coastal Spine, protecting the residents at Galveston Island and the Bolivar Peninsula but allowing some water volume to enter Galveston Bay. The barrier design process however, will be continued without the presence of any new additional structures in order to minimize flood risks and maximize the protection level. The time domain in which an Ike Dike is constructed is unknown, the HSC barrier should therefore provide sufficient protection without the presence of other structures at the outer ends of Galveston Bay (Sebastian et al., 2014; Read, 2012; Bedient, 2014; Rifai and Burleson, 2013; Padgett and Kameshwar, 2013; Colbert and Shanley, 2013; SSPEED, 2013; Penland and Cibor, 2013). The loads for a 1/1,000 years event will therefore not be changed, having 8.0 m surge.

### 3.3 Barrier Alignment

Two different barrier locations at the Ship Channel Entrance have been proposed by Bedient (2014) with accompanying levee alignments, as presented with A and B in Figure 3.3. The most important difference between the two locations is that option B protects a larger area: the Barbour’s Cut Container Terminal in the southwest and some less developed areas in the northeast. Obviously the total length of the alignment is much larger for option B and will require larger investment costs. Appendix C gives a summary of the properties of the two different locations.

#### 3.3.1 Comparison of Alignments

Advanced surge models (ADCIRC-SWAN) are used to compare the response of the bay for the options with and without the gate, by SSPEED-Center at Rice University (Bedient, 2014). In these computer models a variety of storm tracks, winds, and surge levels have been used in order to enable a comparison between the different designs. Option A has been shown to have minimal effects on any increases nearby in the bay due to the large area over which the impact can be dissipated. Option B is currently being run and early results indicate a slightly larger effect than Option A, with some small impact near and in front of the levee and gate. These impacts are manageable compared to a direct hit.
3.3.2 Selection

Recently the SSPEED Center has stated its preference for option B (Penland, 2014) due to the increased amount of protection and the local height profile of the ground surface: Alignment B is almost everywhere about 6 m above mean sea level, and thus requires relatively small adjustments for the land dikes. Subsequently, Penland (2014) made some adjustment to the in Figure 3.3 presented alignment of option B, the environmental section at the Cedar Bayou Channel is moved more southwards and the barrier or dike at this section is replaced by an environmental dune combination as presented in Figures 3.1 and 3.7. With the adjusted alignment the Cedar Crossing Industrial Park is protected as well, as indicated in Figure 3.4(a), while the interference with private properties is minimized since the alignment is crossing mainly undeveloped areas. The environmental section including the row of dunes can be constructed of dredged spoil and may allow some erosion following the natural morphodynamics. This alignment will be used for the design of the protection system, including a movable barrier at Morgan’s Point, an environmental dune section at Cedar Bayou Channel and accompanying land dikes as presented in Figures 3.1 and 3.4.

3.4 Design of the levee sections

As mentioned in Section 3.3.2 and presented in Figures 3.1 and 3.4, the protection system contains dikes surrounding a movable barrier and an environmental dune section. Figure 3.4 shows the presence of three different levee sections accompanied with cross-sections showing the height profiles of the existing ground surface. From west to east one can find: Section 1-Morgan’s Point; Section 2-Atkinson Island; and Section 3-Beach City, presented by Images 3.4(b) to (d) respectively.
Figure 3.4: Alignment of the protection system with the three different levee sections and associated height profile: (a) Overview; (b) Levee section 1, Morgan’s Point; (c) Levee section 2, Atkinson Island; and (d) Levee section 3, Beach City. Obtained from [Penland, 2014; goo, 2014].

3.4.1 Crest height

For simplifying reasons the height of the dike will be designed deterministically and is assumed to be constant over each section, although its input parameters may have different distributions. According to Verhagen and Schiereck (2012) the dike height is the sum of the design water level and a certain freeboard. Which are defined as follows:

- Design water level:
  Astronomical tide + Storm surge (wind set-up) + Margin for seiches and gust bumps + Surcharge (for both sea level rise and/or increase of tidal amplitude) + Settlement of the subsoil.

- Freeboard:
  Follows from wave run-up; or height determined by critical overtopping; and is at least 0.50 m.

Design Water level

For both a 1/100 and a 1/1000 years storm event some computations will be made. The required characteristics for the storm events are introduced in Chapter 2, a quick overview is given in Table 3.1 resulting in the design water level (Design WL).

<table>
<thead>
<tr>
<th>$T_{\text{return}}$</th>
<th>Wind speed</th>
<th>Wave height</th>
<th>Surge</th>
<th>SLR</th>
<th>Tide</th>
<th>Base WL</th>
<th>Design WL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/100 years</td>
<td>50 m/s</td>
<td>5.0 m</td>
<td>6.0 m</td>
<td>0.2 m</td>
<td>0.1 m</td>
<td>-0.2 m MSL</td>
<td>MSL + 6.0 m</td>
</tr>
<tr>
<td>1/1000 years</td>
<td>65 m/s</td>
<td>6.0 m</td>
<td>8.0 m</td>
<td>0.2 m</td>
<td>0.1 m</td>
<td>-0.2 m MSL</td>
<td>MSL + 8.0 m</td>
</tr>
</tbody>
</table>
The design water level is the sum of the water elevation due to storm surge, the sea level rise (SLR) for the complete design lifetime of 100 years and the tide, which will all be added to the base water level. For a 1/100 years storm event this may result in a design water level of 6.1 m +MSL, but it will be assumed that due to the extensive width of the floodplains the amount of bed friction will be significant resulting in a slightly smaller wind set-up and a design water level of 6.0 m.

In Chapter 2 the subsidence issue has been introduced, in combination with settlements due to the weak subsoils during construction this results in a prediction of 1 m of bed degradation during the lifetime of 100 years. This issue will be tackled by constructing the crest height of the dike 1 m higher than currently required. For the run-up and overtopping computations however, the resulting crest height will be used.

As presented in Section 2.5.2 the 1/1,000 year storm surge is 8.0 m, comparable to a ‘Shifted-Ike+30%’. The dike will be designed to be at least this height in order not to have a complete ‘failure’ of the dike, when the storm surge level is higher than the dike crest resulting in overflow. A significant amount of overtopping however, is accepted for a 1/1,000 years storm, allowing some damage resulting in a more economical design (Verhagen and Schiereck, 2012).

Freeboard

The freeboard should be at least 0.50 m, and meeting the requirements with respect to wave run-up or overtopping volumes defined by Verhagen and Schiereck (2012). In order to be able to execute the run-up and overtopping computations, first an estimation will be made based on the ultimate design surge level which is 8 m, associated to the 1/1,000 years storm. The estimated crest height is than 8 m plus the minimum freeboard of 0.5 m which results in a crest height of 8.5 m +MSL. This preliminary levee will have 1:3 slopes which are grass covered without the presence of a berm.

Crest width

All three dike sections will be constructed with a road at the dike crest. These roads however, have very different traffic intensities: At levee section 1 the largest traffic intensity can be found; levee section 3 will accommodate a slightly smaller traffic intensity; and the road constructed at levee section 3 will only enable transport related to the maintenance and operation of the barrier, resulting in different crest width requirements. The preliminary designs will have crest widths of 40 m, 20 m and 6 m for levee sections 1, 3 and 2 respectively.

3.4.2 Dike Optimization by wave Run-up and Overtopping criteria

For sections 1 and 3 the surface height just in front of the dike more or less equals the storm surge level and therefore these floodplains can be seen as a very wide berm at the levee slope. The dike crest for these two sections is elevated only 2.5 m above the local ground surface, but due to the relatively very wide berm (floodplain) of 750 and 1,000 m respectively, it is very unlikely that the waves will cause severe run-up and overtopping since they will break at the shoreline and loose most of their wave energy. Additionally, the distance between the point of breaking and the toe of the levees is expected to be large enough for wave dissipation. As presented in Table 3.2 the hand calculation confirms that the wave run-up associated to a 1/100 years storm at the levee slopes is negligible, and so is the amount of overtopping. The wave run-up at levee section 2 is very large, so this levee section requires a different design dike design which will result from the overtopping criterion as determined in the next phrases.
Table 3.2: Run-up and Run-down for a 1/100 years storm surge for the three levee sections. See Appendix C for a detailed description of all required parameters and computations. (Verhagen and Schiereck, 2012)

<table>
<thead>
<tr>
<th>Sections</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iribarren number</td>
<td>$\xi$ [-]</td>
<td>0.019</td>
<td>1.73</td>
</tr>
<tr>
<td>Run-up (Hunts’ formula)</td>
<td>$R_u$ [m]</td>
<td>0.09</td>
<td>6.93</td>
</tr>
<tr>
<td>Berm width (floodplain $x_{fp}$)</td>
<td>$B_b$ [m]</td>
<td>750</td>
<td>2300</td>
</tr>
<tr>
<td>Average distance SSL-Berm</td>
<td>$h_b$ [m]</td>
<td>0.5</td>
<td>5.8</td>
</tr>
<tr>
<td></td>
<td>$L_b$ [m]</td>
<td>815</td>
<td>4815</td>
</tr>
<tr>
<td></td>
<td>$x$ [m]</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>Berm correction factor</td>
<td>$\gamma_b$ [-]</td>
<td>0.54</td>
<td>0.92</td>
</tr>
<tr>
<td>Grass protection factor</td>
<td>$\gamma_r$ [-]</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>Run-up:</td>
<td>$R_u$ [m]</td>
<td>0.05</td>
<td>6.03</td>
</tr>
<tr>
<td>Run-down:</td>
<td>$R_d$ [m]</td>
<td>0.04</td>
<td>1.85</td>
</tr>
</tbody>
</table>

Table 3.2 introduces the Iribarren number $\xi$, which describes the ratio between the slope of the coast ($\tan \alpha$) and the steepness of the incoming waves. It can be used for the determination of the wave run-up using the formula described by Hunt. The determination of these parameters is presented by Equation 3.1, in which $H$ and $L_0$ are the height and length of the incoming waves respectively.

$$\xi = \frac{\tan \alpha}{\sqrt{H/L_0}} \rightarrow \xi = \frac{R_u}{H}$$

The amounts of overtopping resulting from a 1/1,000 years storm are presented in Table 3.3, computed by the use of methods provided by Pullen et al. (2007) and Verhagen and Schiereck (2012). It can be seen that for a 1/1,000 years storm the amount of overtopping is negligible for levee sections 1 and 3 as well, from which can be concluded that the preliminary dike design is sufficient for these two sections. A sketch of the situation of levee sections 1 and 3 with the design of the dike to be constructed can be seen in Figure 3.5. As expected however, the amount of overtopping (q [l/s]) at levee section 2 is not negligible at all: 4.88 m$^3$/s or 4,875 l/s. Although the protected area behind levee 2 is not developed, this amount of overtopping is way too much with respect to the inner slope stability, which is described by the overtopping criterion for high quality slopes: $q < 10$ l/s.

Table 3.3: Overtopping amounts for a 1/1,000 years storm for all three levee sections. See Appendix C for a detailed description of all required parameters and computations. (Pullen et al., 2007)

<table>
<thead>
<tr>
<th>Sections</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1,000 years storm characteristics</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storm surge level (+MSL)</td>
<td>$SSL$ [m]</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Freeboard</td>
<td>$h_c$ [m]</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Significant wave height (depth-limited)</td>
<td>$H_s$ [m]</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Design wave height (after wave breaking)</td>
<td>$H_d$ [m]</td>
<td>2.0</td>
<td>5.6</td>
</tr>
<tr>
<td>Average water depth</td>
<td>$h_b$ [m]</td>
<td>2.5</td>
<td>7.0</td>
</tr>
<tr>
<td>Correction factor</td>
<td>$\gamma$ [-]</td>
<td>0.85</td>
<td>0.94</td>
</tr>
<tr>
<td>Dimensionless overtopping</td>
<td>$Q$ [-]</td>
<td>$5.76 \cdot 10^{-34}$</td>
<td>$5.29 \cdot 10^{-29}$</td>
</tr>
<tr>
<td>Dimensionless freeboard</td>
<td>$R$ [-]</td>
<td>13.27</td>
<td>0.05</td>
</tr>
<tr>
<td>Average overtopping per meter dike</td>
<td>$q$ [m$^3$/s]</td>
<td>$1.90 \cdot 10^{-29}$</td>
<td>4.88</td>
</tr>
</tbody>
</table>

Obviously, it can be concluded that the preliminary dike design is not sufficient for levee section
2. There are a few design characteristics which can be adjusted in order to decrease the amount of overtopping: the crest height, the berm width or the slopes of the dike. Table C.4 in Appendix C shows the effects of the adjustments and the most economical design in terms of the required amount of construction material. This optimization results in a design for levee section 2 containing a crest height of 12.5 m, a berm width of 12 m and 1:5 slopes. The average overtopping per meter dike at section 2 becomes 9.68 l/s, and thus meets the requirement. A sketch of the dike to be constructed at levee section 2 can be seen in Figure 3.6.

Appendix C contains all computations required to obtain the designs of the three levee sections, as well with sketches and background information.

Figure 3.5: Situation at levee sections 1 and 3, with the design of the dike to be constructed.

3.5 Environmental section

East of Atkinson Island an environmental barrier section has to be constructed, which connects dike sections 2 and 3 with a span of about 1,500 m. The local depths are limited by 2.0 m. In Section 3.3.2 it has already been mentioned that the choice will be made to construct a dynamic combination of dunes out of dredged spoils, which allows movement of the coastline by erosion and deposition of sediments. The dunes will be backed by a road which enables road transport to Atkinson Island. This road will be partly constructed on land and partly on a light caisson structure, which enables tidal movement and the passage of fish and other animals. These water openings can be closed off in times of hurricanes, protecting the Cedar Bayou. See Figure 3.7 for the integration of the environmental barrier into the levee alignment and the existing fairways.

Since the chosen alignment does not conflict with navigational fairways, it will not be necessary to make any adjustments to the existing Cedar Bayou Channel, which connects the Cedar Bayou with the Houston Ship Channel. Because of this, the Environmental Barrier will not be required to allow ship passage. It is designed just to enhance the natural character of the area.
3.6 Navigational section

Following the design rules delivered by PIANC [1997], the channel dimensions will follow from the navigational requirements. Appendix D gives the full elaboration of the calculations for the required channel dimensions as a function of a number of different vessel dimensions. As presented in Sections 2.3 and 2.7, the navigational barrier may not be a limiting factor for shipping in the next 100 years, and will therefore slightly overdimensioned. The resulting channel depth $D_c$ is much larger than the current existing depth at the Houston Ship Channel. This depth, however, will be used to respond on possible channel deepenings in the next 100 years. The chosen design vessel is the Suezmax tanker for which the resulting channel dimensions are given in Table 3.4.

Figure 3.8: Navigational cross sections: (a) at the barrier; (b) at the channel.

The navigational cross section will have a rectangular shape at the location of the barrier. At the remaining parts of the channel where there are no quay walls present, the shape will be trapezoidal.

---

1 It is assumed that the Houston Ship Channel in the future is prepared to accommodate larger vessels. However, terminals able to accommodate these larger vessels may be constructed at other locations at Galveston Bay. This relocation results in smaller channel requirements, which will greatly reduce the required amount of deepenings of the Houston Ship Channel. Subsequently it will reduce the dimensions of a possible barrier. To conclude, the final dimensions of the barrier may be downgraded, following the long-term vision of the HSC Authorities.
with sloping banks of 1:3 (18°) which will be protected with rip-rap slope protection. As presented in Section 5.6.3, the resulting total channel width \( W_c + 2 \cdot 3D_c = 470 \) m, determined with the use of rules provided by F&O (2014). See Figure 3.5 for a sketch of the navigational cross sections.

Table 3.4: Required channel dimensions, for a 2-lane Suezmax traffic. The resulting design can be seen in Figure 3.8 (PIANC, 1997; F&O, 2014)

<table>
<thead>
<tr>
<th>Design vessel</th>
<th>Length: L</th>
<th>Width: B</th>
<th>Draught: D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suezmax</td>
<td>285 m</td>
<td>50 m</td>
<td>20 m</td>
</tr>
</tbody>
</table>

\[
\text{Channel width: } W_c = 2w_{lw} + w_{vc} + 2\sum w_i = 335 \text{ m} \\
\text{Channel depth: } D_c = D - d_{tide} + \sum d_i = 22.2 \text{ m}
\]

3.7 Additional protection methods

In addition to the in this Chapter described structural measures, the protection level can be increased by non-structural measures as well. In this Section some non-structural measures are introduced.

3.7.1 Early warning systems

Early warning systems may be very helpful enabling residents and companies to protect their properties with additional temporary measures like sandbags and the movement of vulnerable or valuable goods to safe (higher) places like a second floor. Early warning systems may improve evacuation procedures as well, thanks to an increased available time for evacuation the process may proceed more organized.

3.7.2 Education

Local residents need to know what the risks of flooding are and when they have to evacuate. A good awareness can be achieved by education at schools, by advertising brochures or messages on television or internet. An important issue is the fact that it is more difficult to reach (poorer) people while they live in the more vulnerable areas. A proper campaign is required.

3.7.3 Evacuation

When a dangerous hurricane is predicted to make landfall in the Houston-Galveston area, one can decide to evacuate the residents living in the area. By evacuation the people may be moved to the higher areas in the hinterland where there is no danger of flooding. This seems like a good measure, but it may be a dangerous process. A good example of a very badly performed evacuation is the evacuation for Hurricane Rita in 2005, see Section 1.2. Hurricane Rita was expected to make landfall in the Houston-Galveston region, what made the government to decide to evacuate the residents. Finally hurricane Rita made landfall much more to the east, causing no danger for the evacuated regions, but still a lot of casualties are seen during the evacuation. A situation like in 2005 should be prevented at all time (Roth, 2009).
3.7.4 Landuse planning

Although the occurrence of hurricanes is well known for the area and severe damages have occurred in the past decades, local residents continue settling at the flood prone areas close to the shores of Galveston Bay. Land use management programs can help protecting future urban development from flood damage and avoiding urban encroachment. It is recommended to combine this with education. Land use planning can improve the evacuation procedure and reduce the consequence of flooding.

3.8 Conclusions

A lot of work has been done considering flood protection of Galveston Bay, resulting in a range of approaches and possible protection structures either inside or on the outside of the bay. Recent studies by the SSPEED Center and Stoeten (2013) have showed that with a closure of Galveston Bay by a seadikes and a barrier on the bay entrance, called the Coastal-Spine concept, the reduction of surge elevation at the Houston Ship Channel entrance is limited due to local wind set-up. Because of this, a barrier at the entrance is needed that will protect HSC facilities from flooding by retaining extreme high water levels in the bay, as a result of water surface elevation in the bay and local wind set-up at the entrance.

Global Design of the Total Protection System

In order to have an optimal balance between the size of the protected area, the total length of the protection system and the integration with the existing infrastructure, the barrier will be located near Morgan’s Point, stretching to Atkinson Island. This location is in line with existing work by Penland (2014) and Bedient (2014), also referred to as ‘Location B’. The chosen alignment of the system provides protection to the Port of Houston and Barbour’s Cut in the West, the Houston Ship Channel in the center and the Cedar Crossing Industrial Park in the East.

The Houston Ship Channel Barrier is the navigational part of a large protection system, which also includes levee sections and an environmental barrier. Three different levee sections have been distinguished and for each section an appropriate design has been made. The function of the environmental barrier is to enhance the natural character of the area by allowing tidal movement and the passage of (water) animals. It can be closed off during a storm event to protect the areas behind it from flooding. Additionally the levees and environmental barrier provide an improvement of the accessibility of Atkinson Island.

Definition of the Navigational Section

Following the navigational requirements the dimensions of the navigational cross section have been determined, providing the global dimensions of the HSC Barrier. The possible gate types for the navigational barrier will be described in the next Chapter. The different types will be evaluated after which the best suitable type is selected to be designed in detail in Chapter 5.

Additional Non-Structural Protection Methods

This chapter has concluded with the introduction of non-structural flood protections. A proper landuse management is recommended, including landuse planning and regulation. To include surge prediction for Galveston Bay, current flood warning systems need to be expanded and the public has to be educated as new expanding population moves into surge zones.
Chapter 4

Navigational Gate Selection

4.1 Introduction

In the previous chapter the alignment of the protection system is defined. A design has been made for the different levee sections and the environmental section. The previous chapter ends with the definition of the location and resulting dimensions of the navigational section. This navigational section will contain a movable storm surge barrier, for which several gate alternatives are possible. In this chapter the different gate alternatives will be evaluated after which a best suitable solution will be selected to design in detail in Chapter 5.

First a selection of different gate alternatives is presented and for each alternative some relevant characteristics are described. For most of the alternative a reference project can be found, where the described alternative is used in practice. These reference projects will be described in combination with their costs, reliability, technical issues and dimensions. At the end of Section 4.2 a summary will be made of the different gate alternatives.

In Section 4.3.1 the different criteria are defined by which each gate alternative will be evaluated. These criteria follow from the barrier requirements listed at the end of Section 2.7. Since the different criteria do not have the same importance, weight factors will be used in the evaluation. These weight factors are introduced in Section 4.3.2 giving each criterion a weight to take the importance differences into account.

Finally, a Multi Criteria Analysis (MCA) will be executed giving a total score for each gate alternative. This score is the sum of the different scores the alternative received for each criterion multiplied by the weight factor related to that particular criterion. The gate type rated with the highest score is the best suitable alternative for this situation. The conclusion about which gate type to be designed in detail in the next chapter is made at the end of this chapter, Section 4.4.

Figure 4.1: Gates types: (a) Sliding/rolling door; (b) Mitre gate; (c) Sector gate; (d) Barge gate; (e) Flap gate; (f) Lifting gate; (g) Radial gate; and (h) Inflatable rubber dam. (Dircke et al., 2012)
4.2 Gate alternatives

A variety of storm surge barriers has been developed in the past century. There is not a single perfect gate type, each type has favorable aspects which can be combined in order to find the best solution for the situation one is dealing with. Dircke et al. (2012) has provided an overview and comparison of navigable storm surge barriers. Some of the gate types Dircke et al. (2012) mentions are presented in Figure 4.1. A selection of different gate types is briefly described in this section, concluding with an overview of either favorable or unfavorable aspects of each gate alternative. In Section 4.3.3 a selection is made by the use of a Multi Criteria Analysis.

4.2.1 Open channel

In addition to the construction of a gate, there is also the possibility to have the navigational channel totally opened. This is called the open channel alternative, in fact the channel is left open as it is at the moment. Consequences of ‘doing-nothing’ are pretty much presented in the previous chapter, Section 3.2, but this is different from doing nothing since the surge related increase of water volume at the Houston Ship Channel might be reduced by the construction of other structures. Either levees around the channel or outside the bay, the Ike Dike, are examples of these measures.

Obviously, the protection level of this alternative is much lower than for any other alternative and is therefore probably not an optimal solution for the flood risk problem in the Houston Ship Channel Complex. As mentioned before, the residual surge at the HSC entrance near Morgan’s Point is still significant when the Bay is fully closed by the combination of an Ike Dike and a barrier at Bolivar Roads resulting in the Coastal Spine. Although the conclusion has been made that a surge retaining structure at the HSC entrance is required anyway, the open channel situation gives the zero-state conditions with which the effects of other alternatives can be compared and evaluated.

<table>
<thead>
<tr>
<th>Table 4.1: Horizontally moving gates, not floating versus floating.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Not floating</strong></td>
</tr>
<tr>
<td>Distribution towards the sides</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td>Floating</td>
</tr>
<tr>
<td>Distribution towards the sides</td>
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</tbody>
</table>
4.2.2 Horizontally moving gates

Horizontally moving gates can be distinguished in two types, based on the mechanism by which the gates move into place, namely by sliding or floating. When moved in place these gates may be sunk until the bottom of the gate rests on an at the bottom constructed sill and the river cross section is fully blocked. These two types are compared in Table 4.1. The gate type that moves by horizontal sliding is the Horizontally Sliding Door, presented in Figure 4.1-(a). Examples of floating gate alternatives are Sector gates and Barge gates, presented by Figure 4.1-(c) and -(d) respectively. The characteristics of these three gate types are presented in the following subsections.

Horizontally sliding door

According to Dircke et al. (2012), the horizontally sliding or rolling gate is a good alternative for relatively wide spans. This gate type has been considered as a preliminary designs for the Maeslant Barrier (Rigo, 2005) and as well in Hamburg (Germany) a barrier containing a horizontally sliding door is proposed to protect the Port of Hamburg from flooding (Sass, 1986). At the Oranjesluizen, navigational locks near the Dutch capital Amsterdam, this type of gates is used as shown by Figure 4.2. The characteristics of horizontally sliding gates are presented in Table 4.2.

<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural aspects, layout and operation</td>
<td></td>
</tr>
<tr>
<td>Large span possible</td>
<td>Large space and deep excavation required for chambers</td>
</tr>
<tr>
<td>No clearance height limitation</td>
<td>Flat and smooth slide way required</td>
</tr>
<tr>
<td>Not subjected to wind</td>
<td>Sector gates: load transfer to hinges,</td>
</tr>
<tr>
<td>Suitable for deep waters</td>
<td>maintenance, corrosion, growth</td>
</tr>
<tr>
<td>Relatively simple movement mechanism</td>
<td>Straight gates: sluice gates may be required;</td>
</tr>
<tr>
<td>can be used</td>
<td>time for dock filling</td>
</tr>
<tr>
<td>Stable structure; no load concentration</td>
<td></td>
</tr>
<tr>
<td>Dry docks: maintenance, no collision</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic and hydrodynamic aspects</td>
<td></td>
</tr>
<tr>
<td>Limited differential head and horizontal flow contraction in last stage of closure</td>
<td>Sector gates: ship collision; siltation in open chambers</td>
</tr>
<tr>
<td>Excess water: through sluice opening</td>
<td></td>
</tr>
<tr>
<td>Suitable for reverse head and flow</td>
<td></td>
</tr>
<tr>
<td>Not sensitive to flow vibrations</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.2: Horizontally sliding gate at Oranjesluizen, navigational locks in Amsterdam, the Netherlands. (Rijkswaterstaat 2014)
Mitre gates

Mitre gates are a common gate type that can be seen for a lot at navigational locks of inland waterways in Europe. They consist of 2 doors which rotate around a hinge at the channel sides. In closed position they have a pointed shape. However, for the defined dimensions of the navigational section this type of gates is not possible to use, since the maximum span related to Mitre gates is limited by 35 m. When the required span exceeds this value, usually sliding doors are used (Loman et al., 2012). The concept is presented in Figure 4.1-(b).

The SSPEED Center in combination with the University of Houston developed a combination of the mitre gates and the floating barge gate (Colbert, 2015). Having two floating barges that rotate at sides, after which they sink to a sill at the channel bottom by filling with water, to have a closure in a pointed shape like the Mitre Gates. Colbert (2015) uses for this concept the name 'Valve Gate'. A visualization of this concept is given in Figure 4.3.

![Visualization of the Valve Gate concept: 3 phases in the closing process. Obtained from Colbert (2015).](image)

Sector gates

Examples of storm surge barriers consisting of sector gates are the Maeslant Barrier (Netherlands), the New Bedford Hurricane Barrier (US), the Harvey Canal Flood Protection Barrier (US) and the IHNC New Orleans Hurricane Protection Barrier (US).

![Table 4.3: Favorable and unfavorable aspects of horizontally floating gates and sector gates.](image)

<p>| Table 4.3: Favorable and unfavorable aspects of horizontally floating gates and sector gates (Dircke et al., 2012) |
|---------------------------------|---------------------------------|</p>
<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural aspects, layout and operation</td>
<td></td>
</tr>
<tr>
<td>Large gate span possible</td>
<td>Large space required</td>
</tr>
<tr>
<td>No clearance height limitation</td>
<td>Complicated operation (water inflow control)</td>
</tr>
<tr>
<td>Shallow dry dock: easy inspection and maintenance and collision protection</td>
<td>Load concentration; forces on hinges</td>
</tr>
<tr>
<td>Can be immersed if sill is covered with silt</td>
<td>Problems for negative differential head</td>
</tr>
<tr>
<td>No flatness of sill required</td>
<td>Mobilization time: filling of dry docks</td>
</tr>
<tr>
<td></td>
<td>Objects on sill can cause damage</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic and hydrodynamic aspects</td>
<td></td>
</tr>
<tr>
<td>Vertical closure of flow opening (no strong horizontal flow contraction)</td>
<td>Sensitivity to flow-induced oscillations</td>
</tr>
<tr>
<td>Separate sluice openings may be applied to reduce differential head and discharge excess water</td>
<td>Sensitive to dynamic wave forces</td>
</tr>
<tr>
<td></td>
<td>Limited resistance to negative differential head</td>
</tr>
</tbody>
</table>

The Maeslantkering, presented in Figure 4.4, is located in the 'New Waterway' near the Port of Rotterdam. It has been completed in 1997 and is part of the Dutch surge protection system called The
Delta Plan. It gets a lot of attention and visitors in a range of tourists, students, business relations and other groups for both educational and recreational purposes. It really is one of the icons of the Dutch Deltaworks. However, this structure is not perfect at all: It gives only protection against a high water level at the sea. In case of a reflective flood wave in seaward direction the gates may even encounter severe damages if they are in a (semi) closed position. In addition to that, a significant probability of failure is detected, related to operational failure by either closure or opening of the gates. Another negative aspect of the structure is the significant amount of maintenance required, giving an extensive amount of yearly costs.

The construction costs of the Maeslant Barrier are about 450 million euros. The navigational depth and width are $17 \, m$ and $360 \, m$ respectively, offering navigational space for a CEMT-class VIb design vessel. The total number of ship passages is about 70,000 per year. The Maeslant barrier is constructed to give protection against a 1:700 years storm with surge elevations until $MSL + 5 \, m$, where closure takes place if water levels exceed $MSL + 3.0 \, m$.\cite{Janssen2014, Rijkswaterstaat2014, SDO2004}.

Barge gates

A barge gate is a floating gate that rotates around a hinge like a mitre gate or sector gate. It consists of one floating barge that moves in place by the flood wave current until it blocks the port entrance. The difference is that the barge gate stays floating all the time in contradiction to the above described gates which, when in place, will be submerged until the bottom of the gate rests on a sill. A barge gate distributes its forces to the both sides where the other gates distribute forces to the sill.

Thanks to the movement system, which simply follows the hydraulic head difference, a complex mechanism is unnecessary, resulting in a very reliable opening and closing system. An example of a barge gate is presented in Figure 4.5 as a proposed design for the Bolivar Roads Barrier.
4.2.3 Vertically moving gates

Flap gate

Examples of storm surge barriers consisting of flap gates are the Stenford Hurricane Barrier (US) and the Venice Storm Surge Barrier (Italy), also known as the MOSE project, presented in Figure 4.6.

Table 4.4: Favorable and unfavorable aspects of flap gates. (Dircke et al., 2012)

<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural aspects, layout and operation</td>
<td></td>
</tr>
<tr>
<td>No limitation of the span</td>
<td>Natural frequencies low; small stiffness, great mass</td>
</tr>
<tr>
<td>Separate flaps; reduced failure risk</td>
<td>Maintenance difficult</td>
</tr>
<tr>
<td>No clearance height and little space required</td>
<td>Pneumatic: not fully controlled</td>
</tr>
<tr>
<td>Invisible and not subjected to wind</td>
<td>Hydraulic: concentration cylinders</td>
</tr>
<tr>
<td>Suitable for deep waters</td>
<td>Underwater: corrosion, growth</td>
</tr>
<tr>
<td>Controlled operation flow and wave</td>
<td>Hinges may wear out in sand</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic and hydrodynamic aspects</td>
<td></td>
</tr>
<tr>
<td>No strong confinement of horizontal flow</td>
<td>Sensitivity to vibrations</td>
</tr>
<tr>
<td>Vertical closure, single flap</td>
<td>Small stiffness during operation</td>
</tr>
<tr>
<td>Enabled water inlet control</td>
<td>Subject to down-pull flow forces and wave loads</td>
</tr>
</tbody>
</table>

The MOSE project is the name of series structures at the 3 inlets protecting the Venice Lagoon from high water levels. It is still under construction, but an impression of the construction and operation is given in Figure 4.6. Different from the horizontally moving gates, the MOSE barrier is completely submerged. Operation goes by the injection of air in the doors to initiate floating. Where the other reference projects attract tourists thanks to their visibility, the Italian government aims for an undisturbed natural horizon around Venice, resulting in this ‘invisible’ barrier.

As explained, the invisibility can be seen as an advantage although maintenance and inspection are complicated, depending on the requirements of the stakeholders. A second advantage of the MOSE project is the relative simple operation system; the gates are lifted by upward air pressure and will block elevated water levels up to $M.S.L + 3.0$ m by ‘floating’ on it. This mechanism is relatively simple and therefore does not result in a large probability of mechanical failure. A disadvantage of the structure are the huge amount of construction costs of approximately €6 Billion (Westendorp, 2014).
Lifting gate

Table 4.5: Favorable and unfavorable aspects of lifting gates. (Dircke et al., 2012)

<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural aspects, layout and operation</td>
<td></td>
</tr>
<tr>
<td>Large gate span (up to 300 feet)</td>
<td>Limited clearance height for shipping</td>
</tr>
<tr>
<td>Little space required</td>
<td>Raised gate subject to wind load</td>
</tr>
<tr>
<td>Controlled operation under flow and wave</td>
<td>Water depth versus gate height</td>
</tr>
<tr>
<td>Raised gate accessible for maintenance</td>
<td>Wheel gates weak spot, wearing</td>
</tr>
<tr>
<td>Proven concept</td>
<td>Smooth slide required versus growth underwater</td>
</tr>
<tr>
<td>Hydraulic and hydrodynamic aspects</td>
<td></td>
</tr>
<tr>
<td>Vertical closure, underside is free of sill</td>
<td>Sensitivity to vibrations</td>
</tr>
<tr>
<td>Discharge of excess water, accepts reverse flow</td>
<td>Small stiffness during operation</td>
</tr>
<tr>
<td>Limited vertical flow forces and wave loads</td>
<td>Subject to down-pull flow forces and wave forces</td>
</tr>
</tbody>
</table>

Examples of storm surge barriers consisting of lifting gates are the Hollandse IJssel Barrier, the Hartel Barrier and the Eastern Schelde Barriers, all three were constructed in the Netherlands as parts of the Dutch Delta Plan. This is the largest Dutch system of flood defenses, designed by the Dutch Ministry of Infrastructure and Environment as a response to the flood disaster of 1953, which took 1,836 lives and revealed the vulnerability for the Dutch southwestern regions (Rijkswaterstaat, 2014; SDO, 2004). The Eastern Schelde barrier - the largest tidal surge barrier in the world - is a typical environmental barrier (Balkema, 1994). The Hollandse IJssel Barrier and the Hartel Barrier however, are typical navigational barriers which could be good examples for the HSC Barrier.

Just like the Maeslant Barrier, the Hartel Barrier serves the Port Of Rotterdam where a lot of navigational traffic is present. The barrier is presented in Figure 4.7. It is constructed to close the Hartelkanaal in case of a storm surge. The barrier consists of two vertical lifting gates which have a length of 49 m and 98 m. When the barrier is opened for navigation, the gates are lifted to 14 m above the water level limiting the size of the vessels that can pass. When the gates are closed, which has occurred only once since the year of operation 1997, the barrier provides protection against water levels up to MSL + 3.0 m. The gates are constructed out of steel and slide vertically at concrete pillars which were at the moment of construction the biggest in the world (Rijkswaterstaat, 2014). The construction costs are approximately €250 Million (Dircke et al., 2012).

Figure 4.7: Hartel Barrier, obtained from www.binnenvaartinbeeld.com (2008).
Radial gate

Table 4.6: Favorable and unfavorable aspects of radial gates. (Dircke et al., 2012)

<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structural aspects, layout and operation</strong></td>
<td></td>
</tr>
<tr>
<td>Large gate span feasible</td>
<td>Load transfer and concentration</td>
</tr>
<tr>
<td>Immediately ready for operation</td>
<td>Segment gate: high sill tolerance demands;</td>
</tr>
<tr>
<td>Controlled operation flow and wave</td>
<td>vulnerable to silting, objects and corrosion</td>
</tr>
<tr>
<td>Little space required</td>
<td>Segment gate: access and maintenance</td>
</tr>
<tr>
<td>Not subjected to wind</td>
<td>Radial gate: limited clearance height</td>
</tr>
<tr>
<td>Segment gate: no clearance limitation</td>
<td></td>
</tr>
<tr>
<td>Inspection and maintenance</td>
<td></td>
</tr>
<tr>
<td><strong>Hydraulic and hydrodynamic aspects</strong></td>
<td></td>
</tr>
<tr>
<td>Limited horizontal flow contraction</td>
<td>Segment gates: sensitive to oscillation in case of overflow</td>
</tr>
<tr>
<td>Excess water through gate</td>
<td>Open gates subject to down-pull forces and wave loads</td>
</tr>
<tr>
<td>Suitable for reverse head and flow</td>
<td></td>
</tr>
<tr>
<td>Radial gate kept free of sill</td>
<td></td>
</tr>
</tbody>
</table>

Examples of barriers consisting of radial gates are the Fox Point Hurricane Barrier (US) and the Thames River Barrier (Figure 4.8), which will be described in the following lines.

The Thames river barrier was built in 1982 on the eastern side (downstream) of London, the capital of the United Kingdom. It is designed to protect 125 square km of central London from flooding caused by tidal storm surges up to MSL + 7.0 m, which can be related to a 1:1,000 years event. The barrier consists of 10 steel gates which have different spans. There are four main gates, located in the central part of the barrier, which have a width of 61.5 m and are mainly used for navigation. The remaining spans are much smaller and have more environmental purposes. The total span of the whole barrier is 520 m. When closed, which is done when water levels in central London might exceed 4.8 m, the barrier gates have a height of 15.0 m from the bottom. Total construction costs were approximately 630 million euros (Env, 2014).

Figure 4.8: Thames River Barrier, obtained from Astrid (2012).
Inflatable rubber dam

A rather new and innovative gate type is the inflatable rubber barrier. At Rampsol in the Netherlands a barrier of this type has already been constructed, as presented in Figure 4.9. It makes use of three inflatable nylon-reinforced rubber dams with a span of 75 m each. The dam is inflatable with air while water is allowed to flow in, retaining high water levels. Lowering of the barrier is done by pumping out the water and releasing the air.

Previous studies from Valk (2014) and Breukelen (2013) have showed that although the barrier at Rampsol is worldwide the biggest of its type, it is possible to apply such a gate type at locations with a much bigger depth and width, as in Houston. It is a relatively innovative alternative.

Table 4.7: Favorable and unfavorable aspects of an inflatable rubber dam. [Dircke et al., 2012]

<table>
<thead>
<tr>
<th>Favorable</th>
<th>Unfavorable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural aspects, layout and operation</td>
<td></td>
</tr>
<tr>
<td>No limitation of span</td>
<td>Flexible structure, low frequencies, small stiffness</td>
</tr>
<tr>
<td>No clearance height limitation</td>
<td>Internal pressure determines stability</td>
</tr>
<tr>
<td>Not subjected to wind</td>
<td>Control of storage and immersion of rubber sheet</td>
</tr>
<tr>
<td>Little space required and invisibility</td>
<td>Not suitable for deep water</td>
</tr>
<tr>
<td>Direct transfer of hydraulic load</td>
<td>Difficult inspection, maintenance and replacement</td>
</tr>
<tr>
<td>No need for hinges and driving system</td>
<td>Vulnerable to vandalism</td>
</tr>
<tr>
<td>Hydraulic and hydrodynamic aspects</td>
<td></td>
</tr>
<tr>
<td>Vertical closure of the flow opening</td>
<td>Ships or objects collision</td>
</tr>
<tr>
<td>Not sensitive to silting of sill</td>
<td>Strong flow contraction in last stage</td>
</tr>
<tr>
<td></td>
<td>Considerable response to wave loads</td>
</tr>
<tr>
<td></td>
<td>No spill of excess water; overflow vibrations</td>
</tr>
</tbody>
</table>

Parachute barrier

In A Movable Storm Surge Barrier for the 21st Century, Ir. Floris van der Ziel investigates a storm surge barrier constructed out of synthetic material like a big parachute that retains the high water levels (Ziel, 2010). This type of barrier uses cloth or canvas instead of a steel, wood or concrete door and it makes use of ropes instead of steel cables for movement of the 'gate'. This makes it a more environmental friendly, dynamic and economical alternative. This gate type has not been developed yet and further research is required.
4.2.4 Costs of Existing Storm Surge Barriers

Dircke et al. (2012) provides a global overview of the construction time and the approximate costs of some existing storm surge barriers, as presented in Figure 4.10. From this overview one is not yet possible to draw any conclusions, since the overview does not give any dimensions of the barriers or the area protected by the barrier. However, it enables a nice comparison between some reference projects.

<table>
<thead>
<tr>
<th>Barrier</th>
<th>Type</th>
<th>Country</th>
<th>Design and construction time (years)</th>
<th>Year of operation</th>
<th>Approximate value in 2010 (million Euros/dollars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollandse Ijssel Storm Surge Barrier</td>
<td>Lifting gate</td>
<td>The Netherlands</td>
<td>4</td>
<td>1958</td>
<td>98/127</td>
</tr>
<tr>
<td>Oosterschelde Storm Surge Barrier</td>
<td>Lifting gate</td>
<td>The Netherlands</td>
<td>10</td>
<td>1986</td>
<td>3850/5005</td>
</tr>
<tr>
<td>Maeslant Storm Surge Barrier</td>
<td>Floating-sector gate</td>
<td>The Netherlands</td>
<td>8</td>
<td>1997</td>
<td>545/709</td>
</tr>
<tr>
<td>Europoort Barrier and Hartel Barrier</td>
<td>Lifting gate</td>
<td>The Netherlands</td>
<td>6</td>
<td>1997</td>
<td>253/329</td>
</tr>
<tr>
<td>Rammol Storm Surge Barrier</td>
<td>Rubber dam</td>
<td>The Netherlands</td>
<td>5</td>
<td>2002</td>
<td>68/88</td>
</tr>
<tr>
<td>Venice Storm Surge Barrier</td>
<td>Flap gate</td>
<td>Italy</td>
<td>ongoing</td>
<td></td>
<td>42/70/5550</td>
</tr>
<tr>
<td>New Bedford Hurricane Barrier</td>
<td>Rolling-sector gate</td>
<td>United States</td>
<td>4</td>
<td>(Bowman et al. 2008)</td>
<td>1966 (Bowman et al. 2008)</td>
</tr>
<tr>
<td>Stamford Hurricane Barrier</td>
<td>Flap gate</td>
<td>United States</td>
<td>4</td>
<td>(Bowman et al. 2008)</td>
<td>1968 (Bowman et al. 2008)</td>
</tr>
<tr>
<td>Harvey Canal Flood Protection Barrier, GIWW WCC</td>
<td>Sector gate</td>
<td>United States</td>
<td>3</td>
<td>2008</td>
<td>730/1014</td>
</tr>
<tr>
<td>IECC New Orleans Hurricane Protection Barrier</td>
<td>Sector gate</td>
<td>United States</td>
<td>5</td>
<td>2011</td>
<td>1014/1315</td>
</tr>
<tr>
<td>Thames Barrier</td>
<td>Segment gate</td>
<td>United Kingdom</td>
<td>Appr. 10</td>
<td>1982</td>
<td>2100/2730</td>
</tr>
<tr>
<td>Fox Point Hurricane Barrier</td>
<td>Radial gate</td>
<td>United States</td>
<td>6</td>
<td>1966</td>
<td>58/75</td>
</tr>
<tr>
<td>St Peterburg Barrier</td>
<td>Lifting gate and floating-sector gate</td>
<td>Russia</td>
<td>Start in the 90s, resumed in 2002</td>
<td>2010</td>
<td>5100/6600</td>
</tr>
<tr>
<td>Ems Storm Surge Barrier</td>
<td>Lifting gate and radial gate, plus segment gate</td>
<td>Germany</td>
<td>5</td>
<td>2002 (PIANC, 2005)</td>
<td>220/286 (Meinhold, 2005)</td>
</tr>
</tbody>
</table>

Figure 4.10: Development costs and construction time of storm surge barriers. (Dircke et al., 2012)
4.2.5 Summary

Table 4.8 gives a summary of the described barriers or gate types. By the use of a Multi Criteria Analysis, which is performed in the next section, one type will be selected based on its characteristics. This selected gate type is technically designed in detail in Chapter 5.

Table 4.8: Summary of the characteristics of the described gate types. (Dircke et al., 2012)

<table>
<thead>
<tr>
<th></th>
<th>Horizontally moving gates</th>
<th>Vertically moving gates</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Roller door</td>
<td>'Valve' gates</td>
</tr>
<tr>
<td>Span limits</td>
<td>unlimited</td>
<td>not sufficient</td>
</tr>
<tr>
<td>Depth limits</td>
<td>unlimited</td>
<td>unlimited</td>
</tr>
<tr>
<td>Clearance height</td>
<td>unlimited</td>
<td>unlimited</td>
</tr>
<tr>
<td>Space required</td>
<td>large</td>
<td>small</td>
</tr>
<tr>
<td>Load transfer</td>
<td>to sill or side ends</td>
<td>to sill or hinge</td>
</tr>
<tr>
<td>Foundation</td>
<td>heavy</td>
<td>heavy</td>
</tr>
<tr>
<td>Sill requirements</td>
<td>very large</td>
<td>large</td>
</tr>
<tr>
<td>Siltation sensitivity</td>
<td>very large</td>
<td>large</td>
</tr>
<tr>
<td>Wave loads</td>
<td>sensitive</td>
<td>very sensitive</td>
</tr>
<tr>
<td>Wind loads</td>
<td>very small</td>
<td>small</td>
</tr>
<tr>
<td>Leakage</td>
<td>low</td>
<td>low</td>
</tr>
<tr>
<td>Vibration</td>
<td>sensitive</td>
<td>very sensitive</td>
</tr>
<tr>
<td>Negative head difference</td>
<td>some issues</td>
<td>no issue</td>
</tr>
<tr>
<td>System Reliability</td>
<td>good</td>
<td>medium</td>
</tr>
<tr>
<td>Approximate closure time</td>
<td>slow</td>
<td>fast</td>
</tr>
<tr>
<td>Durability</td>
<td>very good</td>
<td>medium</td>
</tr>
<tr>
<td>Accessibility</td>
<td>very good</td>
<td>medium</td>
</tr>
<tr>
<td>Visibility</td>
<td>low</td>
<td>low</td>
</tr>
<tr>
<td>Slenderness</td>
<td>robust</td>
<td>slender</td>
</tr>
</tbody>
</table>

CHAPTER 4. NAVIGATIONAL GATE SELECTION 85
4.3 Multi Criteria Analysis

4.3.1 Criteria

The evaluation criteria are already introduced in Section [2.7.3] The design requirements of the barrier. They will be briefly mentioned here.

1. Flood risk (Reliability of the gate)
   - [1.1] Structure failure - The chance that the structure fails: stability and strength.
   - [1.2] System failure - The chance that the system fails due to complex mechanisms.

2. Hydraulics
   - [2.1] Leakage - Amount of water passing when gates are closed.
   - [2.2] Dynamics - Sensitivity to vibrations due to currents, wind or waves.
   - [2.3] Closure/opening time - Duration.
   - [2.4] Flow opening - In opened position, bigger openings are favorable in order not to have accelerations.

3. Structure
   - [3.1] Foundation - Need of big abutments or very large piles due to big weight or sill requirements.
   - [3.2] Required bed and bank protection - Flow velocities near the bed need to be as small as possible, for both opened and closed situations. The expected amount of turbulence is covered in this criterion as well.
   - [3.3] Operation system - Big hinge or multiple wheels, complexity.

4. Navigation
   - [4.1] Width limitations - Navigation channel should be as wide as possible.
   - [4.2] Depth limitations - Span of gates, or do they have to be lifted very high to open.
   - [4.3] Current velocities - Turbulence and flow accelerations near the gates may harm navigational possibilities.

5. Environment
   - [5.1] Fish passage - A safe passage of fish is required, smaller flow velocities are favorable.
   - [5.2] Tidal flow allowance - The opening needs to be as big as possible to enable tidal flow.

6. Constructability
   - [6.1] Experience - The availability of reference projects.
   - [6.2] Equipment - The requirement of very specific equipment.

7. Maintenance
   - [7.1] Accessibility - The possibility to lift or move the gates and the mechanisms to a dry location.
   - [7.2] Material vulnerability - When opened, are the gates dry and safe or submerged and vulnerable to material degradation like corrosion.
8. Aesthetics

[8.1] Landscape integration

[8.2] Spatial quality - Slenderness or robustness of the structure.

[8.3] Recreational quality - Expectations for tourism and the possibility to construct a road on top of the barrier.

[8.4] Space required - Storage space, when gate is in opened position.

4.3.2 Weight factors

Table 4.9: Weight factor for each sub criterion. Rates are from 1 (small importance) to 5 (priority, major importance).

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Factor</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Flood risk</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Structure failure</td>
<td>5</td>
<td>Protection is the main function of the gate.</td>
</tr>
<tr>
<td>1.2 System failure</td>
<td>5</td>
<td>Protection is the main function of the gate.</td>
</tr>
<tr>
<td>2 Hydraulics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Leakage</td>
<td>4</td>
<td>Should be as small as possible in order to minimize water level elevation at land side.</td>
</tr>
<tr>
<td>2.2 Dynamics</td>
<td>1</td>
<td>Not a big issue by itself.</td>
</tr>
<tr>
<td>2.3 Closure/opening time</td>
<td>1</td>
<td>Not very important in case of proper forecasting.</td>
</tr>
<tr>
<td>2.4 Flow opening</td>
<td>4</td>
<td>If not big enough, accelerations will cause erosion.</td>
</tr>
<tr>
<td>3 Structure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.1 Foundation</td>
<td>5</td>
<td>Weak soils so foundation is an important issue.</td>
</tr>
<tr>
<td>3.2 Required bed/bank protection</td>
<td>4</td>
<td>Similar to foundation, but little less important.</td>
</tr>
<tr>
<td>3.3 Operation system</td>
<td>4</td>
<td>Complexity contributes negatively to protection.</td>
</tr>
<tr>
<td>4 Navigation</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.1 Width limitations</td>
<td>3</td>
<td>Navigation is one of the main functions as well, but is inferior to flood protection.</td>
</tr>
<tr>
<td>4.2 Depth limitations</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>4.3 Current velocities</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>5 Environment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.1 Fish passage</td>
<td>4</td>
<td>Minimizing the environmental impact is very important.</td>
</tr>
<tr>
<td>5.2 Tidal flow allowance</td>
<td>4</td>
<td>&quot;</td>
</tr>
<tr>
<td>6 Constructability</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.1 Experience</td>
<td>2</td>
<td>Could contribute to quality of the structure.</td>
</tr>
<tr>
<td>6.2 Equipment</td>
<td>2</td>
<td>Will be closely connected to construction costs.</td>
</tr>
<tr>
<td>6.3 Materials</td>
<td>2</td>
<td>Will contribute to the costs and environmental impact.</td>
</tr>
<tr>
<td>7 Maintenance</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.1 Accessibility</td>
<td>3</td>
<td>Is important for inspection and maintenance.</td>
</tr>
<tr>
<td>7.2 Material vulnerability</td>
<td>2</td>
<td>Connected to maintenance costs.</td>
</tr>
<tr>
<td>7.3 Sedimentation sensitivity</td>
<td>3</td>
<td>Could cause system failure.</td>
</tr>
<tr>
<td>8 Aesthetics</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.1 Landscape integration</td>
<td>2</td>
<td>Is desirable, but not priority.</td>
</tr>
<tr>
<td>8.2 Spatial quality</td>
<td>2</td>
<td>&quot;</td>
</tr>
<tr>
<td>8.3 Recreational quality</td>
<td>1</td>
<td>&quot;</td>
</tr>
<tr>
<td>8.4 Space required</td>
<td>3</td>
<td>For storing the gates.</td>
</tr>
</tbody>
</table>
4.3.3 Results

The tables in Figures 4.11, 4.12 and 4.13 give the results of the Multi Criteria Analysis. Figures 4.12 and 4.13 present the scores for each subcriterion. These scores are rated from 1 (very poor performance) to 5 (excellent performance). A summary with the total scores for each main criterion and the resulting total gate score is given in Figure 4.11.

![Table](image)

Figure 4.11: Total scores for the different gate types.

![Table](image)

Figure 4.12: Scores of the horizontal moving gate types, for each subcriterion.
4.4 Conclusions

A number of barriers is present all over the world, which can be used as references for the design of the Houston Ship Channel Barrier. A variety of gate designs is used for the different barriers that exist, having each its pros and cons. Typical navigation barriers are for example the Maeslant Barrier and the Thames River Barrier, located in Rotterdam (NL) and London (UK) respectively. Both barriers serve a major port with a lot of industrial activities, a dense population and major economical impact. In this chapter a summary of different gate types has been given. Subsequently one gate type is chosen based on the results of a Multi Criteria Analysis (MCA), to be designed in detail.

As presented in Figure 4.11 the horizontally moving Roller (or sliding) Door has the best rating, resulting from the MCA. This gate type provides the best rating for navigational and environmental aspects, since it enables a wide span without the need of any structures that limit navigation or natural dynamics. Furthermore it is resistant to a negative head and minimizes leakage when it is in operation. In the next chapter a technical design will be made for the barrier with this type of gate. Unfortunately, the MCA shows that the Roller Door does not provides the best score of all gates for all of the main criteria. The Barge gate, for example, scores better on Structure, Constructability, Maintenance and Aesthetics, so the horizontal moving gate is far from perfect yet. In Chapter 5 the horizontally sliding door is technically designed, Chapter 6 sequentially presents an alternative design of the horizontally sliding gate for which the mentioned aspects are improved where possible.
Chapter 5

Technical Design: Horizontally Sliding Gate Barrier

5.1 Introduction

In this chapter the design of a horizontally sliding gate is presented. The concept of this gate type has been made clear in Section 4.2. A first sketch of a barrier of this type at the chosen location near Morgan’s Point is given in Figure 5.1. There is a (dry) dock where the gate is stored when the channel is opened for navigation. In the dry dock the maintenance and inspection can take place. Additionally, all installations required for movement of the gate are located at the dry dock. Opposite of the dry dock, on the other side of the channel, there is a much shorter ‘dock’ where the gate its front-end locks when the gate is in closed position. A technical drawing of the overview of the gate is presented in Figure 5.2.

Figure 5.1: Sketch of a roller door barrier at the final alignment, in opened (left image) and closed (right image) position.
There is the option to split the door in two parts which may result in smaller reaction forces due to a reduced span length. In that case the two parts are separately stored, each side of the channel accommodates one part of 200 m. Although that may be a better option in terms of load distribution and movable system requirements for some situations, the decision is made to have only one full length gate at this Houston Ship Channel case, for the following reasons: (1) To minimize the interference and impact at the Atkinson Island Wildlife Management Area; (2) The accessibility of the Morgan’s Point side is much better than the side of Atkinson Island; (3) The major part of the required infrastructure is already present at Morgan’s Point; and (4) It is probably more cost-efficient to have all required installations on only one side instead of double.

Figure 5.2: Overview of the barrier and the two abutments. Three cross-sections of the channel.

A conceptual design for a barrier with a horizontally sliding door has already been developed by Penland and Cibor (2013), this design will be optimized and designed in a higher level of detail in this chapter. Starting with an explanation of the concept and an introduction of the different elements to be designed, in Section 5.2. In this section some aspects which are important as initial conditions for the following design steps are given as well, considering movements possibilities and the possible failure mechanisms.

In Section 5.3 the actual load components resulting from the 1/1,000 years storm event will be described after which the design load can be quantified. Subsequently the expected load transmission will be defined. The barrier design will be checked for stability and strength in Section 5.4, encountering the defined design load. When the final design of the sliding gate door is defined, the mechanism that moves the gate will be designed in Section 5.2.1. This is followed by the design of the foundation and the abutments, of which the results are presented in Sections 5.6 and 5.7 respectively. This chapter concludes with an evaluation of the proposed design, resulting in possible points of improvement which will be used in the next Chapter.
5.2 Concept

In this section the base concept of a horizontally sliding gate is introduced, based on the existing design of Penland and Cibor (2013) as presented in Figure 5.3. Movement of the gate can be done using different driving mechanisms, auxiliary systems and guidance structures. The different options are briefly described in Section 5.2.1, where finally the choice is made to have a sliding gate which will be guided by a rail which is integrated in the foundation slab. The possible failure mechanisms, related to a horizontally sliding gate are described in Section 5.2.2. One of the mechanisms is tackled by the use of the guidance structure, where the other two failure mechanisms will be addressed by the designs of the different elements in the remaining sections.

Figure 5.3: Concept of a roller door type of barrier. Side view of the barrier at the dock, with a possible foundation. Obtained from Penland and Cibor (2013).

5.2.1 Movement mechanisms

There are different options to move the gate from the dry dock to the position in which it closes the channel and protects the hinterland from flooding. The possible mechanisms associated with these two options will be briefly discussed in this section. Finally, the choice is made to have a non-floating gate where friction is reduced by the installation of hydrofenders, as designed in Section 5.5.

Floating vs Non-Floating

Floating will greatly reduce the amount of power required for movement by reducing the resistance relative to friction. The gate might float by itself, without the need of any floating elements, by constructing a gate structure that has a smaller self weight than the amount of water with the same volume, comparable to a Barge Gate. This makes the gate design greatly dependent to the choice of material of the gate and the design of its cross section.

In case of a storm surge, the gate has to float to its position, accurately guided by cables or thug boats. In position it may be sunk to the bottom by adding weight, for example by filling hollow spaces with water. This will allow a complete closure of the navigation channel and it improves the stability by increasing its gravity.
Floaters

If the gate structure will not float by itself, due to a relatively heavy self weight, it is possible to increase its floating capacity by the use of floating elements like inflatable airbags or blocks made of light material like styrofoam. Inflatable bags may be permanently connected to the barrier, during movement these bags may be filled with air until the whole structure floats. In case of styrofoam blocks the floaters have to be attached to the gate before movement, in position the blocks have to be slowly detached for a controlled sinking procedure.

Hydro feet

An ingenious way to move a gate is by the installation of so-called hydro feet or hydrofenders, which are permanently installed pumps in the cross section of the gate which lift the gate by pushing water downwards, creating a thin layer of water between the gate bottom and a sill or rail structure underneath it. It can be compared with a hovercraft, but where a hovercraft uses air the hydrofeet use water. An example of this mechanism can be found at the Oranjesluizen (Orange locks) of Amsterdam in the Netherlands, as presented in Figure 5.4.

Figure 5.4: Hydrofeet: (a) Concept of the mechanism; (b) Cross section of one hydrofoot at the Oranjesluizen; (c) Sideview of the hydrofoot, cross section A-A’ from (b); (d) Picture of the gate bottom with one hydrofoot. (Rijkswaterstaat 2014)

Ory (2003) investigated the feasibility of the use of hydrofeet for the Kallo locks in Antwerp, Belgium. The dimensions of the locks in this study are much smaller than determined for the Houston Ship Channel Barrier, although the characteristics will be similar. The study concluded that the main advantages of hydrofeet are:

- The system is easy to maintain.
- The loads are distributed over a big surface.
- The supporting structure (sill or rail) can be simplified.
- By simplifying the underwater motion system the sustainability of the system improves.
Not floating

In case the choice will be made not to have a floating gate, the gate will have to roll or slide over a guidance rail to arrive in the position to close off the channel. This requires a precisely constructed and installed sill structure. Additionally, due to the weak soil layers, a big and heavy foundation structure may be required.

Push vs Pull

It may be possible to move the door by either pushing or pulling. In case of pushing a mechanism has to be installed at the dock where the gate is stored when not in operation. In case of pulling a mechanism has to be installed on the other side of the channel, connected to the gate by cables. These cables may have negative effects for navigation, but it may result in a smoother movement since it keeps the gate better aligned with the sill by restraining the gate from moving sideways.

5.2.2 Failure mechanisms

Some issues may occur during movement of the gate, related to a horizontally sliding gate type. Three important issues which may cause a complete failure are presented in Figure 5.5. These failure mechanisms are closely related to the movement mechanisms introduced before: A floating gate is very sensitive to the side current but not sensitive to siltation or uneveness’s of the supporting rail; while a gate that moves by a pulling mechanism is probably less sensitive to the three presented failure mechanisms than a gate moved by a pushing mechanism. The final designs for the movement mechanism and guidance rail or sill are presented in Sections 5.5 and 5.6 respectively.

![Figure 5.5: Three possible failure mechanisms during closure of the gate: (a) due to side current; (b) due to siltation of a rail; and (c) due to variations (bumps) in the height profile of the rail or sill.](image)

Current

The river flow might be a problem during closure of the gate, causing a load on the gate side. This lateral force causes a bending moment in the gate which is maximum at the edges of the dry dock. Depending on the stiffness of the structure, a displacement might be the result of this bending moment. A maximum of the bending moment can be found just before total closure of the gate, when the channel is almost fully blocked and the gate is supported only on one side. The strength of the gate itself and its supports has to be great enough to withstand this load, otherwise it may lead to structural failure.

This type of failure mechanism is eliminated by the integration of a guidance rail into the sill. This guidance structure limits the horizontal displacement of the gate by guiding the gate through the channel. A rail however, may be associated with some other failure mechanisms which may be not an issue in case of a floating gate. These two failure mechanisms are briefly described below, examples to solve these problems are given as well.
Sediment deposition

If the choice will be made to have the gate moving over a rail, siltation of the rail has to be avoided at all times. This may be established by maintenance or an additional supporting structure at the gate front end that 'cleans' the rail just before the gate arrives. Examples of gate cleaning systems are jets, brushes or scrapers to be installed at the gate front.

Bumps

With a total rail or sill length of about 800 \( m \) (2 times the span plus some overlap), it is almost impossible not to have any height variations. A very small variation in the order of \( \text{cm's} \) may cause huge moments in the door or even a jamming of the system. To avoid big load differences it is very important to have a perfectly smooth rail without any bumps or a segmented gate that allows some height variations, when the choice will be made to use a not floating gate. Another solution for this problem is to construct the door or gate out of segments which have flexible connections to allow some height variations. This last solution can be compared to a train consisting of multiple separately moving sections.

5.3 Load definition

The loads on the barrier follow from the design storm event and the design conditions described in Chapter 2. In combination with the in Chapter 3 defined barrier dimensions these loads can now be quantified. In Section 5.3.1 the load definition will be presented after which the load distribution through the structure is described in Section 5.3.2.

5.3.1 Loads on the barrier

It is expected that the main load on the barrier will be the load due to the surge elevation on the bayside of the gates, causing a water level difference which is called the hydraulic head. This hydraulic head will cause a distributed load directed from the bayside to the channel. Some additional loads related to extensive hurricane winds, waves and flow currents are expected as well. The size of these loads on a barrier gate can be computed by the use of the Manual Hydraulic Structures (Vrijling et al., 2015), which gives the following expressions for these load types:

\[
q_{\text{wind}} = 0.5C_D \rho_a u_{\text{wind}}^2 \cdot h_c
\]

\[
q_{\text{wave}} = \rho_w g H_i \left( \frac{\exp(kd) - \exp(-kd)}{2k \cdot \cosh(kd)} + \frac{H_i}{2} \right)
\]

\[
q_{\text{current}} = \rho_w h u_{\text{current}}^2
\]

In which:

- \( u_{\text{wind}} \) Design wind velocity \( 65 \ [m/s] \)
- \( \rho_a \) Air density \( 1.25 \ [kg/m^3] \)
- \( h_c \) Gate freeboard, "dry" area above Water Level \( 4.76 \ [m] \)
- \( C_D \) Wind coefficient \( 0.7 \ [-] \)
- \( \rho_w \) Water density \( 1,025 \ [kg/m^3] \)
- \( H_i \) Incoming wave height (non-breaking) \( 4.0 \ [m] \)
- \( k \) Wave number: \( 2\pi/L \) \( 0.3 \ [m^{-1}] \)
- \( L \) Wave length \( 20 \ [m] \)
- \( d \) Water depth (SSL) \( 3.0 \ [m] \)
- \( u_{\text{current}} \) Design flow velocity of the river runoff \( -0.2 \ [m/s] \)
Note that in closed position of the gate the flow velocity from the river directed to the bay very near the gate surface is zero. Due to the channel blockage of the gate, the river run-off results in an increasing water level at the river side. In other words: The horizontal flow velocity of the river run-off (y-direction) transforms to an upward directed velocity (z-direction) by which the level of the water surface at the riverside of the barrier rises. The speed of this water-level rise has been determined in Section 2.2.4 for the case that no drainage or discharge pumps are installed, as discussed in Section 3.5. Anyway, when the gate is in closed position the horizontal flow velocity at the gate surface is zero which results in a neglectible horizontal load due to the current and an increasing water level at the river side. The distributions of the different load components on the gate are presented in Figure 5.6.

Figure 5.6: Loads on the structure. Note that the support reactions ($R_{v1}$ and $R_{v2}$) are only present when the gate rests on the sill. During gate movement, these two loads are not present. Horizontal force equilibrium will be established by horizontal support reaction $R_H$ by the contact between the gate and the side of the sill at the Bay side and the impaction at the abutment head. When in closed position, the six hollow (light grey) rectangular spaces can be filled with water separately, depending on the water level surrounding the gate, improving the gate stability by increasing the vertically directed dead weight.
Design load

The loads on a closed barrier are highly dynamic and will constantly vary, for simplicity reasons the design load will be determined in a deterministic way using a load combination which is likely to give the relevant design load for the barrier. According to Eurocode 0:

$$E_d = \sum_{j \geq 1}^{n} \gamma_{G,j} \cdot G_{k,j} + \gamma_{P} \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i>1}^{n} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$$

(5.4)

In which:
- $E_d$ Design value of the load effect
- $G_{k,j}$ Characteristic value of permanent load $j$
- $\gamma_{G,j}$ Partial factor for permanent load $j$
- $P$ Representative value for the pre-stressing load
- $\gamma_{P}$ Partial factor for the pre-stressing load
- $Q_{k,1}$ Characteristic value of the main variable load
- $\gamma_{Q,1}$ Partial factor for the main variable load
- $Q_{k,i}$ Characteristic value of variable load $i$
- $\gamma_{Q,i}$ Partial factor for variable load $i$
- $\psi_{Q,i}$ Combination reduction factor for variable load $i$

In case of the barrier design, the horizontal loads are most important, inducing huge moments and shear loads in the gates when in closed position or during closing. These horizontal loads are all variable loads, derived from water level differences between both sides (hydraulic head), wind, waves and currents, where the hydraulic head is the main variable load. Although the design storm is a 1/100 years storm event, and the probability of occurrence of a 1/1000 years storm is 10 percent during the design lifetime of 100 years, a total destruction of the barrier is not accepted for this 1/1000 years storm event. Therefore the design load for the strength computations will follow from the 1/1,000 year storm surge of 8.0 m. Assuming the absence of pre-stressing, the design value of the distributed horizontal load for the full height of the gate will than be:

$$q_d = \gamma_{Q,1} \cdot Q_{\Delta H} + \psi_Q (\gamma_{Q,wind} \cdot Q_{wind} + \gamma_{Q,wave} \cdot Q_{wave})$$

(5.5)

In which:
- $q_d$ Design value of distributed horizontal load $[kN/m]$
- $Q_{\Delta H}$ Maximum hydraulic head (8.0 m) $2.091 [kN/m]$
- $\gamma_{Q,\Delta H}$ Partial factor for the main variable load $1.5 [-]$
- $Q_{wind}$ Characteristic value of wind load $11 [kN/m]$
- $\gamma_{Q,wind}$ Partial factor for wind load $1.5 [-]$
- $Q_{wave}$ Characteristic value of wave load $500 [kN/m]$
- $\gamma_{Q,wave}$ Partial factor for wave load $1.5 [-]$
- $\psi_Q$ Combination reduction factor for variable loads $0.5 [-]$

With the use of the introduced design values of the above expressed parameters, the design value for the distributed horizontal load will than be: $q_d = 3,520 \text{ kN/m}$, for the complete height of the gate in closed position. The further load distribution is described in the following Section.

5.3.2 Load distribution

With the use of the 'Strip Method' [Fennis, 2012], the load distribution by the vertical door can be determined. The door will be modelled as a concrete slab which is clamped at the sill and on both
side-ends, as presented in Figure 5.7. The maximum horizontal displacement due to the horizontal design load can be expected at location A. This displacement can be determined in two ways, δA,1 and δA,2, where by definition: δA,1 = δA,2. This results in a load factor α = 0.994, so 99.4 percent of the horizontal loads is transmitted to the sill, and 0.6 percent is transmitted to the sides.

![Figure 5.7: Determination of load transmission: Strip Method. (Fennis, 2012)](image)

These moment resistant connections are only valid in ultimate design conditions, when deformation or displacement of the gate are present due to rotation of the complete gate structure. As presented in Figure 5.8-(b), rotation of the structure induces a counter-acting torque. At the abutment heads this takes place at a gate rotation θ₁ = 0.77° relative to the vertical axis. Horizontal deformation at the abutment head is limited by 0.5 m, resulting in a rotation less than one degree. These are very small rotations, which justify the assumption of moment resistant connections.

![Figure 5.8: Rotation of the gate, resulting in counteracting reaction forces: (a) Top View, maximum rotation relative to x-axis θ₀ = 1.8°; (b) Cross-section at abutment head, maximum rotation θ₁ = 0.77°; and (c) Cross-section at the channel, maximum rotation θ₂ = 6.4°.](image)

For the connection between the guidance rail and the gate, the rotation is important as well. Where especially the maximum possible rotation is important, since this has to be limited to prevent derailing of the gate. As presented in Figure 5.8-(c), the rotation for which derailing occurs θ₂ = 6.4°. For this rotation a horizontal displacement of 0.25 m is required at the bottom of the gate, which is
not available for a design with 0.20 \textit{m} free space (10 \textit{cm} on each side). The guidance rail will stop the rotation of the gate, where a counter-acting torque is the result as well. As presented in Figure 5.8-(b) this clamped property starts acting at a rotation induced horizontal displacement $\delta_y = 4.16$ \textit{m} at the top of the gate.

In reality however, the gate is expected not to be rotating this much. Thanks to its vertical directed dead weight, the gate remains stable and upright. Some horizontal displacement is expected, which results in a counteracting horizontal force at the bay side of the guidance rail, as presented in Figure 5.6. Although this is situation is not in line with the above described modelling, it is expected that the guidance sill structure will transmit the major part of the horizontal loads for this situation too. The determined load distribution will not be changed.

### 5.4 Design of the cross section

The cross section for this gate is inspired by \cite{PenlandCibor2013}: A concrete wall with a number of hollow spaces. The assumed material used for this gate type is reinforced concrete, C70/85. Significant crack widths ($w_c > 0.2 \text{ mm}$) in the concrete are not allowed to prevent corrosion of the reinforcement steel due to contact with water, which is an important boundary condition for the strength checks. The concrete wall contains hollow spaces in order to reduce the required amount of material and the amount of friction during movement by decreasing the self weight of the wall. The required door width will be the result of a stability analysis. The required door height follows from the surge level and the wave height. These checks will be briefly described in this section, see Appendix E for a detailed description of all the required steps and computations.

In order to end with an economically optimized design of the cross section\footnote{Following the original design of \cite{PenlandCibor2013} the shape of the gate has not been changed. Optimization has only been executed with respect to the outer gate dimensions and the inner configuration of the hollow spaces. For future design studies, it is recommended to investigate other shapes of a horizontally sliding concrete gate.}, which meets all strength and stability requirements, a computational model has been produced. The designer’s input for the model is the thickness of the walls and slabs in the cross section, for which the model checks the stability, the strength and the resulting amount of concrete required to produce the gate. All elements are checked for compression and tensile stresses, as presented in Appendix E. The configuration with 6 holes turned out to be the economical optimum in terms of required amount of concrete. This configuration is presented in Figure 5.6 and the final dimensions are given in Table 5.2. The checks to arrive at this design are briefly described in the following sections.

![Figure 5.9: Eight different configurations of the hollow spaces in the door cross-section. Note that for all configurations transversal load bearing measures need to be present, like buttress walls or steel bracings. Although these measures are not drawn, it is assumed to be present each 50 \textit{m} of gate length.](image)
5.4.1 Stability

The horizontal design load for the barrier has been determined in Section 5.3.1, resulting in an over the gate length (x-axis) distributed load of \( q_{d,x} = 3,520 \text{ kN/m} \) for the total barrier height (z-axis). When distributed over both directions a design load of \( q_{d,x,z} = 100 \text{ kN/m} \) can be found. The Strip Method (Fennis, 2012) has showed that 99.4 percent of all horizontal loads is transmitted to the sill, as described in Section 5.3.2. It is assumed that in the middle of the channel all horizontal loads are transmitted to the sill, as presented in Figure 5.6.

For the stability check, the loads presented in Figure 5.6 are used, for which the actual values are expressed in Table 5.1. There will be stability as long as the moment around point \( S_1 \) in Figure 5.6, resulting from the horizontal loads and self weight of the barrier, will be anti-clockwise and thus can be adopted by an upward directed vertical reaction force in the sill or the movement mechanism. The resulting self weight is determined in Appendix E for all different configurations. For configuration 6, door height \( h_{gate} = 35 \text{ m} \) and width \( w_{gate} = 18 \text{ m} \), the submerged self weight is \( G_0 \approx 4,000 \text{ kN/m} \), which is the minimum dead load when all holes are empty.

The gate stability is checked for a number of situations: 1. At the moment of closing, when the dead load \( G_0 \) and the hydraulic head \( (q_{H,bay} - q_{H,river}) \) are minimal; 2. Closed position, just before the gate is supported by the sill while slowly submerging; and 3. Closed position, with a maximum hydraulic head can be found. The different checks determine whether or not an additional downward directed load is required, which can be provided by filling the empty spaces in the cross-section with water.

Strength checks, presented in the next Section, show that the horizontal inner slabs of the barrier are strong enough for this procedure. The maximum obtainable dead load follows: \( G_{filled} \approx 7,500 \text{ kN/m} \). Results of the stability checks are given in Table 5.1 for all three situations.

### Table 5.1: Stability check for the concrete door (configuration 6). The moment around \( S_1 \) (Figure 5.6) is required to be positive (anti-clockwise) for stability. The points of action are the heights on which the in Figure 5.6 introduced resultant forces act, measured from point \( S_1 \) at MSL -22 m.

<table>
<thead>
<tr>
<th>Loads</th>
<th>Point of action</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_{H,river} )</td>
<td>2,433 kN/m</td>
</tr>
<tr>
<td>( q_{H,bay,d} )</td>
<td>4,525 kN/m</td>
</tr>
<tr>
<td>( q_{waves,d} )</td>
<td>500 kN/m</td>
</tr>
<tr>
<td>( q_{wind,d} )</td>
<td>11 kN/m</td>
</tr>
<tr>
<td>( G_0 )</td>
<td>4,000 kN/m</td>
</tr>
<tr>
<td>( G_{filled} )</td>
<td>7,500 kN/m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Situation</th>
<th>Stability check</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. During closure</td>
<td>Loads: ( G_0 ) and ( q_{H,bay} - q_{H,river} ) ( M_{/S_1} : ) 20,000 ( &gt; 0 ) → Stable</td>
</tr>
<tr>
<td>2. Just before closure</td>
<td>Loads: ( G_0 ) and ( q_{H,bay,d} ) ( M_{/S_1} : ) -7,500 ( &lt; 0 ) → Unstable ( \text{Note: } ) Real situation is less unfavorable.</td>
</tr>
<tr>
<td>3. Closed position</td>
<td>Loads: ( G_{filled} ) and ( q_{H,bay,d} ) ( M_{/S_1} : ) 25,000 ( &gt; 0 ) → Stable</td>
</tr>
</tbody>
</table>

Table 5.1 shows that only for situation 2 some issues may occur considering stability. However, the real-life situation is much less unfavorable than used for the check. For the check an instantaneous surge elevation of 8 m is used leading to \( q_{H,bay,d} = 4,525 \text{ kN/m} \). In reality the surge will elevate slowly after closure, which can take several hours. Additionally, when closed the gate is clamped by both sides, which greatly increases the stability. According to this it can be concluded that the door stability will not be an issue, as long as the duration of filling of the gate with water is shorter than the time between gate-closure and maximum surge levels.

CHAPTER 5. TECHNICAL DESIGN: HORIZONTALLY SLIDING GATE BARRIER 101
5.4.2 Strength

The eight possible configurations presented in Figure 5.9 are checked for strength by varying the thickness of the inner and outer walls, resulting in an optimal design in terms of the required amount of concrete. The results of these checks are presented in detail in Tables E.4 and E.5 of Appendix E. Table 5.2 shows a summary of the required dimensions resulting from this strength checks, which shows that configuration 6 is the economical optimal design. Figure 5.10 shows a qualitative representation of the expected load transmission through the transverse walls. A computational check of the internal load transmission using software Matrixframe confirms that the internal strength is sufficient, as presented in Appendix E.

Table 5.2: Resulting dimensions for the different configurations. Note that configuration 6 gives the economical optimum in terms of required volume of concrete.

<table>
<thead>
<tr>
<th>Configuration:</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2*4</td>
<td>1*2</td>
<td>1*3</td>
<td>1*4</td>
<td>1*5</td>
<td>1*6</td>
<td>1*7</td>
<td>1*8</td>
</tr>
<tr>
<td>Total gate width $W_{gate}$ [m]</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Total gate height $h_{gate}$ [m]</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>Concrete $V_{concrete}$ [m$^3$/m]</td>
<td>280</td>
<td>418</td>
<td>383</td>
<td>323</td>
<td>299</td>
<td>269</td>
<td>272</td>
<td>277</td>
</tr>
<tr>
<td>Height openings $h_{opening}$ [m]</td>
<td>8.05</td>
<td>15.50</td>
<td>10.27</td>
<td>7.15</td>
<td>5.40</td>
<td>4.53</td>
<td>3.70</td>
<td>3.08</td>
</tr>
<tr>
<td>Submerged dead weight $q_{dw, gate}$ [kN/m]</td>
<td>4046</td>
<td>6048</td>
<td>5537</td>
<td>4671</td>
<td>4324</td>
<td>3891</td>
<td>3940</td>
<td>4013</td>
</tr>
<tr>
<td>Thickness slabs $t_{c, hor}$ [m]</td>
<td>0.7</td>
<td>2.0</td>
<td>1.4</td>
<td>1.6</td>
<td>1.6</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Thickness walls $t_{c, ver}$ [m]</td>
<td>2.0</td>
<td>5.0</td>
<td>4.4</td>
<td>3.0</td>
<td>2.2</td>
<td>1.7</td>
<td>1.4</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Figure 5.10: Qualitative representation of the load distribution to the transverse walls and transmission to the bottom, for a 100 m cut of the concrete rectangular door.
5.4.3 Dynamics

The above described stability and strength checks are executed in a deterministic way, using maximum values for the occurring loads. In reality these loads are fluctuating in time, causing vibrations on the structure. Instability of the structure may occur due to vibrations as well. These vibrations, or excitation, may be induced by waves and unstable flow patterns. The resulting load follows from basic dynamics, as described by the equation of motion:

\[
(m + m_w) \frac{d^2 y}{dt^2} + (c + c_w) \frac{dy}{dt} + (k + k_w)y = F
\]  

(5.6)

In which

- \(m\): Mass \([kg]\)
- \(y\): Displacement \([m]\)
- \(t\): Time \([s]\)
- \(c\): Damping \([Ns/m]\)
- \(k\): Stiffness \([N/m]\)
- \(F\): Impressed force \([N]\)
- \(m_w, c_w, k_w\): Hydrodynamic components of \(m, c\) and \(k\)

For now, the wind load will be neglected, since it is much smaller than the other two loads due to the hydraulic head and the wind waves. These two loads can both be described as a harmonic wave, with a wave period, a wave length and a propagation speed. Which results in the following sinusoidal approximation for the displacement of the water surface: \(\text{Vrijling et al., 2015}\)

\[
\eta(r,t) = a \cdot \sin(\omega t - kr + \alpha)
\]  

(5.7)

In which

- \(a\): Wave amplitude \([m]\)
- \(\omega\): Angular frequency \([rad/s]\)
- \(k\): Wave number \([rad/m]\)
- \(t\): Point in time \([s]\)
- \(r\): Point in space \([m]\)
- \(\alpha\): Phase \([rad]\)

Acting on a surface with width \(B\), this results in the sinusoidal load as presented in Equation 5.8, describing the load components \(F_i\) for either the surge wave or the wind waves. The values for the different components are given in Table E.1, Appendix E. The load envelope associated to the water surface difference between the bay and the channel is given in Figure 5.11.

\[
F_i = B \cdot 0.5 \rho_w g \eta^2 = B \cdot 0.5 \rho_w g \left(a \cdot \sin(\omega t - kr + \alpha)\right)^2 \quad [kN]
\]  

(5.8)

The strength checks of the previous section have already presented that the strength of the different gate elements is sufficient to bear the maximum loads. However, vibrations which are close to the eigenfrequency \(p\) of the structure may be a new problem, called resonance. To address this issue, the characteristics of the structure will be determined in the following phrases. The frequency which may be expected to be an issue in terms of resonance is the one associated to the wind waves \(\omega_\text{waves} = 0.698\) \(rad/s\), where the structure will encounter resonance when equation 5.9 is valid.

\[
\frac{\omega}{p} = 1
\]  

(5.9)
Figure 5.11: Graph for the resulting load envelope, for the complete 48 hours duration of a hurricane. Loads are given as the total load difference between the two sides of the barrier, for the full height of the gate and a gate segment width of 50 m. Note that the period of the significant wave height ($H_{s,1/1000} = 8.0$ m) is not in the correct time scale, it is just to give an impression of the varying water level difference, in reality the associated peak period $T_p \approx 9$ s. The tidal wave ($H_{T/2} \approx 0.1$ m) is neglected.

The response of the gate, in terms of displacement $y$, speed $\dot{y}$ and acceleration $\ddot{y}$, depends on the stiffness of the gate. Where the motion parameters for $y$ describe the eigenfrequency $p$. It is expected that the gate is very stiff thanks to the transverse walls. However, if the expected load transmission to the transverse walls as presented in Figure 5.10 is not correct, the longitudinal walls have to distribute the horizontal loads to the bottom. In that case, the gate can be modelled as a frame structure as presented in Figure 5.12. The gate width is divided into segments of 50 m each, covering the distance between two bearing sections surrounding the hydrofenders. For reinforced concrete, [ECS, 2004] gives a damping of 10 percent for reinforced concrete. The stiffness can be determined for each wall using equation 5.10. With wall height $L = 5.83$ m, moment of inertia $I_y = \frac{1}{12} B t^3 = 20$ m$^4$ (using width $B = 50$ m and wall thickness $t = 1.7$ m) and Youngs Modulus $E_c = 38,500$ N/mm$^2$ this results in a stiffness $k_{\text{wall}} = 47.7 \cdot 10^6$ kN/m for each wall.

$$k = \frac{12 E_c I_y}{L^3} \quad (5.10)$$

Figure 5.12: Modelling of a frame structure with 6 floors.

---

2 Modelling the gate as a frame structure is just to check a simplified 'worst-case' scenario for which none of the loads are distributed to the transverse (buttress) walls which are present each 50 m. This might be the case if the connections between the buttress walls and the other walls are very weak. However, it is expected that the majority of the horizontal loads is transmitted to the foundation by these transverse walls, presented with the two diagonals in the left image of the Figure.
A structural analysis has been executed in which first the Mass Matrix $[M]$ and the Stiffness Matrix $[K]$ have been determined, which are both 6x6 matrices. The next step is the determination of the Flexibility Matrix $[F] = [K]^{-1}$, followed by the Dynamic Matrix $[D] = [F][M]$. The Dynamic Matrix can be seen in Equation \(5.12\) all other structural matrices can be seen in Section 5.1.3 Appendix E Concrete barrier design. The eigenvalues and corresponding eigenvectors can now be determined using the Method of Stodola (Clough and Penzien 2003), expressed by Equation \(5.11\)

\[
[D]{\phi} = \lambda{\phi} = \frac{1}{p^2}{\phi} \tag{5.11}
\]

The first mode $\lambda_1$ describes the first natural frequency $p_1$ by $\frac{1}{p_1}$. By the execution of a matrix iteration, which is fully described in Appendix E, this first mode has been determined using the criterion $\{\phi\}_1 = [D]\{\phi\}_0$, where eigenvector $\{\phi\}$ has the shape of a 6x1 vector matrix. This iteration process has been repeated after which the following mode shape and eigenvector were found:

\[
[D]\{\phi\}_0 = 10^{-5}
\begin{bmatrix}
2.79 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
3.40 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
4.38 & 4.38 & 4.38 & 6.13 & 10.2 & 30.6 \\
10.2 & 10.2 & 10.2 & 10.2 & 10.2 & 30.6 \\
30.6 & 30.6 & 30.6 & 30.6 & 30.6 & 30.6
\end{bmatrix}
\begin{bmatrix}
0.430 \\
0.433 \\
0.441 \\
0.464 \\
0.535 \\
1.000
\end{bmatrix} = 10.14 \cdot 10^{-4} \{\phi\}_1 
\tag{5.12}
\]

Now the first mode shape $\{\phi\}_1$, combined with Equation \(5.11\) gives the first natural frequency as presented by Equation \(5.13\). This first natural frequency is much larger than the frequency of the load by the waves $\omega_{waves} = 0.698$ rad/s, therefore the resonance criterion presented in Equation \(5.9\) has not been met. Since the higher order natural frequencies are by definition larger than the first mode, the resonance criterion will never be met. It can be concluded that the loads due to vibrations will not be an issue.

\[
\lambda_1 = 10.14 \cdot 10^{-4} \rightarrow p_1 = \sqrt{\frac{1}{10.14 \cdot 10^{-4}}} = 31.4 \text{rad/s} \tag{5.13}
\]

From the first natural frequency $p_1$ one can conclude that the concrete gate is a very stiff structure, when constructed as a frame structure. As mentioned before however, in the final design the stiffness has even been increased by the construction of transverse walls which will distribute the majority of the horizontal loads to the bottom. As presented in Figures 5.10 and 5.13 a pair of walls is present around the hydrofenders each 50 m of door, resulting in a very stiff and robust gate that remains stable and provides sufficient strength.

### 5.5 Design of the Movement mechanism

The cross section computations have showed that the gate will not be floating by itself. Therefore it will be supported by a sill to restrain the gate from large horizontal displacements by guiding the gate through the channel. To eliminate the two described failure mechanisms associated to a rail-supported mechanism it has been decided to install hydro feet, as described in Section 5.2.1 and Figure 5.4. This moving mechanism has huge influence on the sill requirements. The design of the sill is presented in Figure, which will be further explained in Section 5.6. To minimize the amount of head-resistance, the front-end of the gate will pointed, like the shape of an arrow.
Hydrofenders

Hydrofenders will create a thin layer of water between the bottom of the gate and the sill, which enables movement without a very big friction force. Additionally, the hydrofenders will act like jets which flush away the sediments from the sill structure. The sill will have some lower laying sides, where the flushed sediment can be collected without that the contact area between the sill and gate gets affected. When in position, the hydrofeet will slowly reduce their power resulting in a gentle positioning of the gate. Figure 5.13 presents a part of the concrete gate with the installed hydrofenders.

![Diagram of hydrofenders installation](image)

Figure 5.13: Installation of the hydrofenders in the door. The left image presents a sideview of the door at one of the locations that contains two hydrofenders, which will be installed every 50 m. The right image presents the cross section at the same location. The hydrofenders are surrounded by two 1.0 m thick buttress walls, which distribute the horizontal loads to the bottom.

The hydrofenders do not solve the issue of uneveness’s or bumps in the sill when the gate is in action, supported by the sill. In order to have a load transmission between the gate and the sill which is equally distributed, the sill needs to be perfectly horizontal which is almost impossible for a structure with the dimensions of the HSC Barrier. To deal with this issue the top surface of the sill, which is in contact with the gate bottom, will be covered with the relatively flexible material rubber. According to [Eng 2015], the characteristic friction coefficient \( \mu \), required to determine the horizontal bearing capacity by friction, is for friction between rubber and wet concrete about 0.45-0.75 times the vertical load by dead weight. Additionally, local subsidence which cause unequal settlements of the sill have to be prevented at all costs, for a proper execution of the function of the barrier. This will require a very strong foundation structure, as already expected during the Multi Criteria Analysis of Chapter 4. The foundation is designed in Section 5.6.
Driving Mechanism

The barrier will be pulled to its place by a double pulley system. At both sides of the dock a pulley is installed which forces the gate to move towards the other side by reeling in a cable that is connected to the back-end of the gate. At the same time a double reel-system at the back end of the dock looses two cables which are connected to the back-end of the gate as well. These cables are connected to the gate to pull the gate back to the dock when the storm has passed. (Figure 5.21)

5.6 Design of the Foundation

As mentioned before, the gate will be supported by a sill consisting of concrete slabs which are supported by a pile foundation. These slabs offer load transmission and guidance of the barrier keeping the gate in place. To be able to keep the gate in place, all loads have to be properly transmitted by the foundation to the bearing soil layer.

5.6.1 Sill

The sill will be designed as a concrete bridge structure on which the concrete door can move. The sill is supported by a pile foundation, consisting of inclined piles to establish a bearing capacity in both horizontal and vertical directions.

The preliminary design of the sill consists of a simple rectangular shape. The sill is supported by pile groups of 12 piles, with a horizontal distance (x-direction) of 20 m between each pile group. The total foundation length in x-direction is 900 m, resulting in a number of 45 spans of 20 m sill.

External Loads

The maximum vertical load on the sill occurs when the gate is in operation, filled with water and resting on the sill. As presented in Section 5.4, this maximum load is $G_{filled} = 7,443 \text{kN/m}$. The size of the additional loads due to the self weight of the sill and the water columns above the sill depend on the shape of the sill. With a basic rectangular shape which is 25 m wide and 5 m thick this results in the following load combination.

$$q_{d, sill} = G_{filled} \cdot \gamma_G + G_{sw} \cdot \gamma_Q + q_{water} \cdot \gamma_Q$$

(5.14)

In which:
- $q_{d, sill}$ Design value vertical load on sill [kN/m]
- $G_{filled}$ Weight of the gate, when filled with water [kN/m]
- $G_{sw}$ Self weight of the sill, submerged [kN/m]
- $q_{water}$ Weight of the water column on top of the sill [kN/m]
- $\gamma_G$ Load factor for self weight [-]
- $\gamma_Q$ Load factor for variable load [-]

The load factor for self weight has a value 1.0 (Vrijling et al., 2015). The water level, and thus the size of the water column, is variable but has some limitations: A water level for a 1/1000 years storm event is used. To deal with the variability, the load factor used for variable load has a value 1.10 [-]. With the above expressed dimensions, this result in a maximum stress in the concrete sill which is equal to $6.3 \text{N/mm}^2$ which is much larger than the concrete tensile strength ($f_{ctd} = 2.3 \text{N/mm}^2$). To increase the strength, the sill thickness can be increased or the span length can be decreased. For this design, however, prestressing will be the method used to solve the strength-load-issue.

The sill will be constructed out of concrete slabs of 20 m in length, 25 m wide and a thickness of 5 m. These slabs will be prestressed using post-tensioned steel with bond. Each tendon will consist...
of 19 strands ($A_{p,\text{strand}} = 140 \text{ mm}^2$), prestressing steel type Y1860S7, and is anchored by wedges. Prestressing and anchoring are executed on land, after which the prestressed concrete elements will be installed on top of the pile foundation, which make each slab act as a simple supported beam with maximum bending moment half way the span. Following from this load scheme, all prestressing tendons will get a curved profile of which the drape $f$ will have a maximum value in order to have an economical optimal design in terms of required amount of prestressing steel. The eccentricity at the ends of the slab will be zero, and the eccentricity half way the span length will be 2 m. For the design of this type of prestressing, [Walraven and Braam, 2012] gives the following four boundary conditions, in case of crack formation is not permitted:

- In the unloaded state, the stress in the pre-tensioned compression zone should not exceed a specified tensile stress.
- At the same time, the stress in the pre-compressed tension zone, should not exceed a specified compressive stress.
- At maximum load, the tensile stress in the pre-compressed tension zone should not exceed a certain value.
- At the same time, the compressive stress in the pre-tensioned compression zone should not exceed a certain value.

For this situation, only two of the above described boundary conditions will be decisive, resulting in the following two boundaries for the prestressing:

$$\sigma_{c,\text{top}} < 0.00 \frac{N}{mm^2} \rightarrow -\frac{P_{\infty,\text{max}}}{A_c} + \frac{P_{\infty,\text{max}} \cdot e_p}{W_t} - \frac{M_{\min}}{W_t} < 0$$

$$\sigma_{c,\text{bot}} < 2.30 \frac{N}{mm^2} \rightarrow -\frac{P_{\infty,\text{min}}}{A_c} + \frac{P_{\infty,\text{min}} \cdot e_p}{W_b} + \frac{M_{\max}}{W_b} < 0$$

Where 2.30 $N/mm^2$ equals the concrete tensile strength, so cracking does not occur. For the given sill dimensions and eccentricity $e_p$ these boundary conditions result in the following prestressing limits:

$$147,059kN < P_{\infty} < 310,714kN$$

For the used tendons ($A_{p,\text{tendon}} = 19 \cdot A_{p,\text{strand}}$) this result in the required amount of prestressing steel as presented in equation [5.19], where 20 percent loss is assumed ($P_{\infty} = 0.80P_0$) as a result of time-dependent processes like shrinkage, creep and relaxation. The resulting design of the prestressed sill slabs is presented in Figure [5.14]. Note that the tendon profile meets the minimum radius requirement, $R > 40\phi_t$ according to the Dutch code NEN6720 [Walraven and Braam, 2012], where the used tendon diameter $\phi_t = 80 \text{ mm}$.

$$A_p > 131.8 \cdot 10^3 \text{mm}^2 \rightarrow n_{\text{tendons}} = 50$$

The final design of the cross section of the sill is presented in Figure [5.15] for an end-supports of the slab. As presented, the final design is somewhat different from the preliminary rectangular designed slabs. Some adjustments have been made for the following reasons: The amount of required concrete is slightly reduced; the connection with inclined piles is improved; and the gate is clamped by the new shape.
Checking the stability of the sill, the situation presented in Figure 5.6 is used as for the stability of the door, but now the point of rotation will be moved to the center of the foundation: $S_F$. For situation 3 (Table 5.1) this results in an eccentricity $e = 6.44$ m (see Figure E.1), where the upward directed support reaction equals the downward directed dead weight.

Figure 5.14: Prestressing design of the sill elements.

Figure 5.15: Final design of the cross section of the sill. The tendon groups are presented for the location at the ends of the slab, where in line with the neutral axis (n.a.) the eccentricity and thus the resulting prestressing moments are zero. The curvature for the fictitious tendon remains the same, with drape $f_p = 2.0$ m.

**Internal load transmission**

The internal load transmission at the sill is presented in Figure 5.16, for which the internal tensile stresses require a properly designed reinforcement configuration. The sill reinforcement requires stirrups, which is an anti-splitting reinforcement for which the actual required amount of steel follows from the shear forces. The load transmission is modelled by the use of a Strut and Tie Model, presented in Figure 5.16(b), for which the blue lines represent the negative stresses (Struts) and the red lines the positive stresses (Ties). Since concrete tensile strength is much smaller than the compressive strength, the positive stresses are most important for the determination of the amount of shear reinforcement.
The design of the shear reinforcement is presented in Figure 5.16-(c) by the black lines. The red lines indicate two locations where vertical prestressing might be necessary.

A check on punching shear loads, as presented in Equation 5.20, shows that the sill does not require the application of punching shear reinforcement since $v_{Rd,c} > v_{Ed}$. Where the minimum tensile reinforcement is used, according to Fennis (2012): $A_{s,min} = 0.26 f_{ctm} bd / f_{yk} = 10.9 \times 10^5$ $mm^2/m$, for mean concrete tensile strength $f_{ctm} = 3.5$ $N/mm^2$ and characteristic value of the yield strength of the reinforcement steel $f_{yk} = 500$ $N/mm^2$. With a concrete cross-sectional area $A_c = 6.0 \times 10^6$ $mm^2/m$ this tensile reinforcement ratio $\rho_t = 0.182\%$. The outer perimeter $u_{out}$ where punching shear reinforcement is required has a length of 4150 $mm$, giving a distance $r = 0.66 m$. This is smaller than half the pile diameter $\phi_t$ which confirms that no punching shear reinforcement is required.

$$v_{Rd,c} = 0.12 k (\rho_t f_{ck})^{1/3}; \quad v_{Ed} = \frac{V_{Ed}}{u_1 d_{eff}}$$

(5.20)

In which:
- $v_{Rd,c}$: Punching shear resistance of the concrete slab: 0.35 $[N/mm^2]$
- $v_{Ed,c}$: Punching shear stress in the concrete slab: 0.017 $[N/mm^2]$
- $f_{ck}$: Design compression strength of concrete C70/85: 70 $[N/mm^2]$
- $V_{Ed}$: Maximum shear load: 7,500 $[kN/m]$
- $u_1$: Control perimeter: $= 2\pi(0.5\phi_t + 2d)$ 83 $\times 10^3$ $[mm]$
- $\phi_t$: Pile diameter, determined in next section: 2.5 $[m]$
- $d_{eff}$: Effective sill thickness: $\approx 0.9d$ 5.4 $\times 10^3$ $[mm]$
- $d$: Sill thickness: 6.0 $\times 10^3$ $[mm]$
- $k$: Factor: $= 1 + \sqrt{200 + d_{eff}}$ 1.19 [--]
- $\rho_t$: Tensile reinforcement ratio: $= A_{sl}/A_c$ 1.82 $\times 10^{-3}$ [--]

5.6.2 Pile foundation

The sill slabs are supported by pile groups consisting of 12 piles with a distance of 20 $m$ between each pile group. In order to enable transmission of both the vertical and the horizontal loads, all piles are installed with an angle of inclination of 1:3 (horizontal:vertical). Additionally, to be able to resist horizontal loads in x-direction due to movements of the barrier, four piles of each pile group have an inclination in the x-direction. The resulting pile configuration $^3$ is presented in Figure 5.17.

$^3$In order to have a relatively simple force equilibrium, the designed pile foundation is symmetrically. However, with respect to the maximum load situation the optimal design might be asymmetrical. For further optimization, it is recommended to investigate this alternative.
As presented in Section 2.4 Geology, the soil type at at the bottom of the channel is clay, at MSL − 22 m. This type of soil has not sufficient bearing capacity for a heavy structure like this. The bearing soil layer is dense sand, which is present at depths from MSL − 50 m and MSL − 60 m. The piles have to be installed with a penetration of 4 m into the bearing soil layer, resulting in required pile lengths of approximately 26 to 36 m.

The type of pile foundations that will be used is a foundation consisting of steel close-ended tubular piles because this foundation type is able to provide good strength and stability for these relatively long pile lengths, according to [Abede and Smith (1994)]. The piles will be installed by driving, which will be executed until the required depth is reached after which the remaining part of the pile can easily be cut off.

Figure 5.17: Configuration of the pile foundation. A pile group is located every 20 m, supporting the 20 m long sill elements. Note that half of the pile groups contain 4 piles that have a bearing capacity for the x-directed horizontal loads.

**Bearing Capacity**

The stability and strength of the pile foundation will now be checked by computing the bearing capacity of each pile with the use of formulas provided by [Vrijling et al. (2015)], [Geotechniek (2014)], [Abede and Smith (1994)]. For a detailed description of all required design steps and formulas see Appendix G. For steel close-ended tubular piles the maximum bearing capacity of each pile is the sum of the Tip Bearing Capacity, the Skin Friction and the (negative) Dead Weight of a pile, as described by Equation 5.21.

\[
F_{t,\text{max}} = F_{t,\text{tip}} + F_{t,\text{skin,}+} - F_{t,\text{skin,} -} - F_{t,\text{dw}} [kN]
\]  

According to [Sin (2003)] the negative skin friction \( F_{t,\text{skin,} -} \) can be neglected for steel close-ended tubular piles founded on a sand layer, which is the case for the foundation of the HSC Barrier. The remaining elements can be computed with the following equations: [Vrijling et al., 2015]

\[
F_{t,\text{tip}} = A_t \cdot P_{t,\text{tip}} = \frac{1}{4} \pi \phi^2 t \cdot P_{t,\text{tip}} [kN]
\]  
\[
F_{t,\text{skin,} +} = \pi \cdot \phi_t \cdot h_{x,j} \cdot P_{t,\text{skin}} [kN]
\]  
\[
F_{t,\text{dw}} = L_t \cdot (\gamma_s \cdot \frac{1}{4} \pi \phi^2 t + \gamma_c \cdot \frac{1}{4} \pi (\phi_t - 2t)^2) [kN]
\]
In which: \( A_t \) \([m^2]\) Cross sectional area of the pile tip 
\( \phi_t \) \([m^2]\) Diameter of the steel tubular pile 
\( t_t \) \([m]\) Wall thickness of the tube outer steel 
\( L_t \) \([m]\) Pile length 
\( P_{t,\text{tip}} \) \([kN/m^2]\) Maximum soil pressure under the pile tip 
\( P_{t,\text{skin}} \) \([kN/m^2]\) Skin friction pressure 
\( h_{s,i} \) \([m]\) Thickness of soil layer \( i \) 
\( \gamma_s \) \([kN/m^3]\) Selfweight of steel 
\( \gamma_c \) \([kN/m^3]\) Selfweight of concrete

For now the inclination, penetration depth and design loads will be taken as constant deterministic variables. The in Figure 5.17 presented configuration has been checked by increasing the diameter of the piles, which will be the same for all piles, resulting in the bearing capacities as presented in Table 5.3. As presented, the final outer diameter of the piles \( \phi_t = 2.5 \) \( m \), the thickness of the steel \( t_t = 0.025 \) \( m \), the inclination of the piles is 1:3 and the total number of piles per slab is 12 (Figure 5.17).

Table 5.3: Bearing capacities associated with the final design of the pile foundation \( (\phi_t = 2.5 \) \( m; \) \( t_t = 0.025 \) \( m; \) \( \alpha_t = 71^\circ) \), which will vary between these values for varying bearing depths. As presented, the bearing capacity will always be larger than the characteristic design loads.

<table>
<thead>
<tr>
<th>Pile bearing depth:</th>
<th>MSL − 64 m</th>
<th>MSL − 54 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical span ( L_z ) ([m])</td>
<td>35.5</td>
<td>35.5</td>
</tr>
<tr>
<td>Pile length ( L_t ) ([m])</td>
<td>37.4</td>
<td>37.4</td>
</tr>
<tr>
<td>Total bearing capacity ( F_{t,\text{max}} ) ([kN])</td>
<td>154,300</td>
<td>128,900</td>
</tr>
<tr>
<td>Pile tip bearing ( F_{t,\text{tip}} ) ([kN])</td>
<td>147,800</td>
<td>120,100</td>
</tr>
<tr>
<td>Positive skin friction ( F_{t,\text{skin}+} ) ([kN])</td>
<td>12,580</td>
<td>12,580</td>
</tr>
<tr>
<td>Negative skin friction ( F_{t,\text{skin}−} ) ([kN])</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pile dead weight ( F_{t,\text{dw}} ) ([kN])</td>
<td>6,080</td>
<td>3,748</td>
</tr>
<tr>
<td>Vertical design load per slab ( F_{z,d} ) ([kN])</td>
<td>340,600</td>
<td>374,900</td>
</tr>
<tr>
<td>Horizontal design load per slab ( F_{y,d} ) ([kN])</td>
<td>0</td>
<td>70,400</td>
</tr>
<tr>
<td>Vertical bearing capacity ( R_{z,d} ) ([kN])</td>
<td>701,100</td>
<td>569,500</td>
</tr>
<tr>
<td>Horizontal bearing capacity ( R_{y,d} ) ([kN])</td>
<td>254,300</td>
<td>217,800</td>
</tr>
</tbody>
</table>

5.6.3 Protection against erosion

Bed Protection

For the determination of the required bed protection the propeller wash related to the design vessels is assumed to be decisive. Verhagen and Schiereck (2012) gives for the flow velocity near the bed \( u_b \) the expression presented by Equation 5.25 for a Suezmax tanker with engine power \( P = 15,000 \) kW and propeller diameter \( d = 4 \) \( m \).

\[
 u_b = u_{b,\text{max}} - 0.5 V_s \tag{5.25}
\]

In which: \( u_b \) Design flow velocity near the bed, results in: 2.5 \([m/s]\) 
\( u_{b,50} \) Maximum flow velocity behind the vessel 7.2 \([m/s]\) 
\( V_s \) Design velocity of the vessel 9.6 \([m/s]\)
For determination of the bed stability, Verhagen and Schiereck (2012) gives the expression of Equation 5.26. For the above computed design bed velocity \( u_b = 2.5 \, \text{m/s} \), gravitational acceleration \( g = 9.81 \, \text{m/s}^2 \) and relative density \( \Delta = 1.6 \, [-] \), this results in a required nominal stone diameter \( d_{n50} = 0.5 \, \text{m} \). A rock grading of stone class HMA 300-1000 (300-1000 \( \text{kg} \)), which is associated with a nominal stone diameter \( d_{n50} = 0.59 \, \text{m} \), should be sufficient for maintaining stability of the channel bed. A minimum layer thickness of about 0.9 \( \text{m} \) might is required.

\[
\Delta d_{n50} = 2.5 \frac{u_b^2}{2g} \tag{5.26}
\]

According to Verhagen and Schiereck (2012), the best type of bed protection of a barrier of this size is a granular protection. The original bed material at the Houston Ship Channel is clay, which is a very fine sediment. For a granular filter a number of sand layers is therefore required, where diameters increase from bottom to top. A good example of different layers is presented in Figure 5.18, resulting in a closed granular filter layer. As presented, the top layer consists of the required stone class HMA 300-1000. If it is required to protect the complete channel bed, it might be more cost-efficient to use a geotextile. This greatly reduces the amount of filter layers and can directly be covered by the armor stones of HMA 300-1000. However, since the channel needs to be dredged periodically it is expected that some bed erosion is no issue as long as it is far enough away of the barrier and the stability of the structure is not endangered.

![Figure 5.18: Cross-section of the sill and the bed protection layers around the barrier, obtained from Verhagen and Schiereck (2012).](image)

Bank protection

Waves induced by propagating vessels will cause loads on the channel banks. In order not to have bank erosion the bank slopes will be protected with a rip-rap slope protection, as mentioned in the previous phrase. The required stone size of the rip-rap slope protection will follow from the primary waves associated with the limit speed of Suezmax tankers, and can be determined using equation 5.27 (Verhagen and Schiereck, 2012).

\[
\frac{v_s^2}{gh} = \frac{2z/h}{(1 - A_s/A_c - z/h)^{-2} - 1} \tag{5.27}
\]

In which:
- \( z \) Water level depression \([\text{m}]\)
- \( h \) Channel depth (Base WL) \([\text{m}]\)
- \( A_s/A_c \) Blockage, ratio vessel/channel cross-section areas \([\text{-}]\)
- \( v_s \) Vessel speed (70% of limit speed \( V_l \)) \([\text{m/s}]\)
The computation presented by Equation [5.27] results in a water level depression \( z = 0.78 \) m. According to Verhagen and Schiereck [2012] some corrections have to be made to the computed water level depression for design purposes. First there is the trapezoidal cross section, which results in \( z_{ecc} = 1.31 \) m using equation [5.28]. Secondly there is the two line traffic which gives a maximum height of the stern wave \( z_{max} = 1.96 \) m as presented in equation [5.29].

\[
    z_{ecc} = (1 + \frac{2y}{b})z
    \tag{5.28}
\]

\[
    z_{max} = 1.5z_{ecc}
    \tag{5.29}
\]

In which:
- \( y \) Distance channel axis to ship axis \( 160 \) [m]
- \( b \) Width of the channel including slopes \( 470 \) [m]
- 1.5 Constant, with standard deviation of 10 % (Van der Wal, 1990)

The required nominal stone diameter \( (d_{n,50}) \) of the rip-rap now follows from equation [5.30] where \( \Delta \) is the relative density and \( \alpha \) is the slope of the banks. With \( \Delta = 1.6 \) and \( \alpha = 0.33 \) this results in a required nominal stone diameter \( d_{n,50} = 0.57 \) m. A standard class of rock grading (EN 13383) like HMA 300-1000 or HMA 1000-3000 would be sufficient, where HM means heavy gradings. To deal with possible variations in loads or strengths, the choice will be made to use HMA 1000-3000, which is described with stones of 1-3 tons and a nominal stone diameter of 0.90 m. The minimum required layer thickness is 1.35 m \( (1.5d_{n,50}) \), but the decision is made to have a layer thickness of two times \( d_{n,50} \) (Verhagen and Schiereck, 2012).

\[
    \frac{z_{max}}{\Delta d_{n,50}} = 1.8\cot(\alpha)^{0.33}
    \tag{5.30}
\]

5.7 Design of the Abutments

When in operation, the gate closes the channel and is clamped by both ends: the abutments. As presented in Section [5.3.2] and Figure [5.7] only a very small part (0.6%) of the total load on the barrier will be distributed to the abutments. Besides the common function of the load transmission, the two abutments have each their own functions:

- **Dry Dock**
  In the biggest one of the two abutments, the dry dock, the gate is stored when the channel is opened for navigation during normal weather conditions. This abutment contains a lot of free space around the gate, in order to enable maintenance. Additionally, all installations and systems required for gate movement are located around this dock. The dry dock is closed of from the river by a mitre gate consisting of two doors.

- **Upper side**
  At the upper side of the channel only a small abutment can be found, without a dock. In addition to the load transmission function, this abutment has the function of closing off the channel. Big rubber seals are installed on the sides to minimize the amount of leakage.
5.7.1 Abutment head

The abutment head contains the most complex elements of the abutments. The abutment head will have the same shape for each side, consisting of two robust pillars at each abutment which have the function to bear the horizontal loads, keeping the gate at its place and minimizing the amount of leakage. The pillars will have the shape of an octagon consisting of eight sides of 4 m resulting in a diameter of about 10 m. The top of each pillar will be at $MSL + 14$ m, being 1 m higher than the gate and resulting in a total height of 36 m, measured from the channel bottom at $MSL - 22$ m.

At the side of the dry dock, the abutment part just behind the pillars contains some important elements. First there are the mitre gates, which have the function of keeping water out of the dock when the gate is stored during normal conditions. When operation is needed, these doors have the function of controlling water inlet. Each gate door will have multiple inlets at different heights, which can be opened separately allowing the water to flow into the dock with the desired discharge. The installations at the abutment head on the side of the dry dock are presented in Figure 5.19.
Design of the pillars

The load distribution from the gate to the abutments has already been introduced in Figure 5.8 Section 5.3. The load part that is distributed to each abutment, originating from the gate in closed position, is equal to:

\[
F_{\text{abutment}} = \frac{1}{2} \cdot \alpha \cdot q_{y,d} \cdot L_{\text{gate}} = \frac{1}{2} \cdot 0.006 \cdot 3520 \cdot 335 = 3.540 \text{kN} (5.31)
\]

Assuming that this load is transferred to only 1 square meter of pillar surface, a normal compression stress \( \sigma_c = 3.5 \text{ N/mm}^2 \) can be found in this part of concrete. This stress is much smaller than the design value of the concrete compressive strength \( f_{\text{ccd}} = 46.67 \text{ N/mm}^2 \), for concrete C70/85. When modeling this load transmission as a clamped beam of one meter width, thickness \( t_c \) and a span \( L = 8 \text{ m} \), the required wall thickness for the abutment to have no cracks in the concrete can be computed with Equation 5.32 resulting in \( t_c > 0.67 \text{ m} \).

\[
f_{\text{ccd}} > \frac{M_{\text{max}}}{W_c} = \frac{1}{8} \cdot \frac{F_{\text{abutment}}L}{1/6 \cdot t_c^2} \rightarrow t_c > \sqrt{\frac{6F_{\text{abutment}}L}{8f_{\text{ccd}}}} (5.32)
\]

In addition to load transmission of the loads on the barrier, the abutments have to be designed on other loads as well. When the channel is opened for navigation, a hydraulic load is acting on the pillars induced by head pressure differences in and outside the pillar and by wave impact. This 'base-load' is presented in Figure 5.20(a), where the transmission of the loads acting on the barrier is described as a point (Figure 5.20(b)). Figure 5.20(c) represents loading due to collision with a navigating vessel. Although measures are taken to prevent this type of loading, it is an important type which may not be forgotten (Vrijling et al., 2015). A detailed description and overview of different load types is given in Appendix E, concluding that the presented design provides sufficient strength to retain the three described load types.

![Image of loads acting on the pillars](image)

Figure 5.20: Loads acting on the pillars: (a) Hydraulic loads by pressure differences and wave impact; (b) Load transmission of barrier loads; and (c) Collision force by a vessel.

5.7.2 Dry Dock

The dry dock is in fact an open concrete box, consisting of a bottom slab and diaphragm walls on three sides, located just behind the abutment head. The dry dock contains all installations and systems required for operation, maintenance, inspection and construction, as shown by Figures 5.19 and 5.22.
Bottom Slab and Soil Retaining Walls

The shape of the bottom slab will be similar to the sill located at the channel bottom, but with sides extended by 12 m on each side. The total width of the dry dock then becomes 52 m. The diaphragm walls will bear at MSL – 56 m, resulting in walls of 62 m high. The diaphragm walls will be constructed first, after which the dock can be excavated. Due to this excavation the local ground water level needs to be lowered and the water has to be drained out of the building pit. When the desired depth has been reached, the pile foundation can be installed by driving. Finally the bottom slab is constructed on top of the pile foundation, on three sides connected to the diaphragm walls.

A detailed description of the design of the bottom slab and the diaphragm walls can be seen in Appendix [E]. The diaphragm walls will experience great lateral loads from the soil due to the big dimensions. The maximum rotational moment in the wall can be found at the connection with the bottom slab. For this design moment the required amount of reinforcement steel in the wall has been determined, resulting in sufficient capacity to bear retain this load. This capacity requirement is presented in Equation (5.33). The final wall thickness is 3 m, with an applied reinforcement ratio $\rho_s = 0.030$ or 3.0%.

$$A_s f_y d \cdot 0.4t_{wall} + \alpha b x_u f_{cd} \cdot (0.5t_{wall} - \beta x_u) > M_{Ed}$$

In which:
- $f_y d$: Design yield strength of steel B500B: 435 · $10^3$ [kN/m$^2$]
- $f_{cd}$: Design compression strength of concrete C70/85: 46.7 · $10^3$ [kN/m$^2$]
- $t_{wall}$: Thickness of the diaphragm wall: 3.0 [m]
- $\alpha_c$: Area factor of concrete, for C70/85: 0.62
- $b$: Width of the wall, will be considered per 1 m 1.0 [m]
- $x_u$: Length of concrete compression zone 1.27 [m]
- $\beta$: Depth ratio of concrete 0.35
- $M_{Ed}$: Bending moment acting on the connection 83,400 [kNm/m]

Figure 5.21: Soil stresses acting on the dry dock.

CHAPTER 5. TECHNICAL DESIGN: HORIZONTALLY SLIDING GATE BARRIER 117
Installations and Systems

In addition to all structures, the dry dock will need several installations for operation purposes. These installations can be associated with accessibility, movability, maintenance and support. Since the gate gets filled probably a few times a year, most of the installations have to be water resistant or removable. Some of the required will be briefly mentioned on the next page, an indication of their exact locations can be seen in Figure 5.19. An overview of the Dry Dock is presented in Figure 5.22.

- **Access structures**
  To make the dock accessible for both people, equipment and materials a lift and a staircase will be installed in both the inner corners of the dry dock.

- **Fixed equipment**
  It may be required to have equipment either in or around the dry dock. Equipment that enables this, and suits the situation of the HSC Barrier Dock is a 'Top-running-bridge-crane' or 'Overhead-crane' (End [2015]). This type of crane can move equipment in all horizontal directions and lift materials or other equipment in or out the dock. It requires a rail on each long side of the dock, resulting in a span of about 60 m. This fixed equipment causes significant additional loads to the soil and the soil retaining walls.
• Movability
  Installations considering the movability of the gate have been discussed in Section 5.5, containing a double pulley system. Two pulleys are installed just behind the pillars and the mitre gates, one on each side. Additionally a pulley construction has to be installed at the end of the gate, enabling pull-back of the gate when water levels, after a storm surge, have decreased again.

• Water drainage
  Although the dock is designed to be fully dry and water resistant, there is always the ability for water to enter by both rainfall or leakage. When water will stay for long time in the dock, it might damage the structure. To remove water out of the dock pumps need to be installed that are able to drain both small amounts of water due to leakage or rainfall, but also has to be able to drain the water out of the dock when the dock is completely filled with water after operation.

Construction method

The construction of the diaphragm walls will be the first step in the construction of the whole dry dock. When the construction of the diaphragm walls on each side of the dock is finished, the excavation of the dock can take place while the local ground water is being drained. When sufficient depth (± MSL − 28 m) has been reached, driving of the pile foundation can take place. Subsequently the prefabricated bottom slabs can be installed on top of the pile foundation and connected to the diaphragm walls, resulting in a water tide dry dock. Finally the gate and the required equipment and installations can be installed. The required steps associated with the construction of the diaphragm walls are presented in Figure 5.23.

![Figure 5.23: Steps for the construction of the diaphragm walls. Obtained from Vrijling et al. (2015).](image)
5.8 Conclusions

A horizontally sliding gate barrier, constructed out of concrete, is a suitable solution to protect the Houston Ship Channel from storm induced flooding. The installation of a guidance rail at the bottom of the channel enables gate movement over the relatively wide stretch, preventing failure due to the presence of transversal currents. It is able to transmit both horizontal and vertical loads to the foundation, keeping the gate structure in place. However, two other failure mechanisms are related to this type of guidance structure, induced by sedimentation and an uneven surface due to variations in settlements or construction errors. These issues are eliminated by the design of the foundation and the movement mechanism respectively.

Gate design

Following the original design proposed by Penland and Cibor (2013), the cross-section of the gate will have a rectangular shape. The total barrier height is 35 m measured from the top of the guidance rail at the channel bottom at MSL − 22 m. The width of the gate is 18 m and it will be constructed as one 400 m long beam. Stability is mainly provided by the guidance structure the channel bottom, the abutment heads at both ends have a minor contribution to the load transmission. The cross section of the gate will contain six hollow rectangular spaces of which the height and width are 4.5 m and 14.6 m respectively, resulting in the most economical optimum in terms of required amount of material while providing sufficient strength to bear the design load. When stored at the dock, the hollow spaces are accessible for maintenance and inspection. When in operation, the hollow spaces can be filled with water to improve the stability by increasing its dead weight.

Movement

Movement will be done by a double pulley system that pulls the gate into the channel, and pulls the gate back after operation. First the dry storage dock needs to be filled with water, which is done by gradual opening of the water retaining gates at the abutment head. To reduce the amount of friction and increase the smoothness of the sliding surface, a set of hydrofenders is installed each 50 m of gate. These hydrofenders create a thin layer of water between the bottom of the gate and the sill. This enables movement without a very big friction force and additionally they act like jets which flush away the sediments from the guidance structure, eliminating failure due to sedimentation.

Foundation

The bearing soil layer is present at depths of MSL − 50 to −60 m. A pile foundation consisting of inclined close-ended steel tubular piles is designed to distribute both the horizontal and vertical loads to the bearing soil layer. This type of foundation is the best alternative for this situation, due to the relatively long required pile lengths. Additionally the length of this type of piles can be easily adjusted by cutting of the remaining part when the appropriate depth has been reached, which is a big advantage since the pile lengths range from 26 to 38 m. For simplicity, the piles are all the same, having an outer diameter $\phi_t = 2.5$ m, a steel thickness $t_t = 0.025$ m and an inclination of 1:3, providing sufficient bearing capacity for all required depths. Prefabricated prestressed concrete slabs are installed on top of the pile foundation. By the use prefabrication, a very high concrete quality is possible with precise structural dimensions. This minimizes the risk of an uneven guidance rail, where uneven settlement are eliminated by the pile foundation. The guidance rail simply results from the shape of these concrete slabs, it will just be covered with some kind of bitumen or rubber in order to provide protection which is smooth, flexible and water resistant. The width of the foundation slabs is 30 m, which is sufficient to prevent problems related to seepage.

The required protections have been determined for the channel bed and banks. The bank protection is established by a 1.8 m thick layer of HMA 1000-3000 rip-rap protection, consisting of stones of 1-3
tons and a nominal stone diameter \( d_{n,50} = 0.90 \text{ m} \). The channel bed is protected from scour by a granular protection consisting of multiple layers. The required top grading around the navigational barrier is HMA 300-1000 to protect the bottom from propeller jet related erosion.

**Abutments and Dry Dock**

On each side of the channel an abutment is constructed, which have the functions of load transmission and leakage reduction. The gate is stored at one side of the channel at a dry dock, protected by the western abutment. This enables one to have all systems and installations required for operation on only one side, reducing the amount of requirements for the other side. The side of the dry dock has therefore a number of additional functions and installations related to maintenance, inspection, operation, visitors and accessibility to the gate. The dry dock has the shape of a rectangular concrete box, with an open top. The sides consist of diaphragm walls, which bear on the same soil layer as the pile foundation. Halfway the height of these walls is the concrete bottom slab of the dry dock constructed, which has the same shape as the concrete slabs at the channel, but with extended sides in order to provide space for construction of the gate, maintenance and inspection.

**Evaluation**

The horizontally sliding gate provides a suitable and robust solution for the flood risk reduction at the Houston Ship Channel. The gate and its support structures are a stable and strong piece of engineering, where the existing infrastructure at Morgan’s Point provides good accessibility for construction and operation.

However, there are some aspects that might need some improvement. The concrete rectangular gate has a great dead weight of 250 - 500 \( kN/m^2 \) when submerged, and 450 \( kN/m^2 \) when stored at the dry dock. A heavy foundation is required to transmit these loads. A second aspect is the rectangular shape: vertical straight walls are in terms of horizontal load transmission not the best solution. Wave reflection and vibrations can be expected while the horizontal loads have to 'travel' a long way to get to the foundation.

Besides these structural aspects some improvement can be achieved in aesthetics. Landscape integration can be improved, resulting in more spatial quality. Additionally recreational quality can be increased by a more attractive design, which can be achieved by a more slender structure or a recognizable landmark. These improvements will be further elaborated in the next chapter, which concludes with a summary of technical specifications of both barrier alternatives (Table 6.2).
Chapter 6

Alternative Design: Horizontally Sliding Steel Truss

6.1 Introduction

An design of the horizontally sliding door barrier has been presented in the previous chapter, where the gate cross section is inspired by the original concept of Penland and Cibor (2013). The resulting design is a robust construction that meets the strength and stability requirements, it deals with issues that may be related to gate movement and it enables maintenance and inspection. However, this gate type did not have the best rating for each criterion in the Multi Criteria Analysis of Chapter 4. Aspects which may easily improvable compared to the original concrete design are:

- **Hydraulics**
  The sub-criterion *Dynamics* could be improved by a sloping wall which reduces the vibrations due to the wave impact. Additionally, for the original design it is expected that filling of the dry dock will take multiple hours, a design for which this time has been reduced will greatly improve the rating for the *Closing/opening time*.

- **Structure**
  The sub-criterion *Foundation* could be improved by the construction of a more slender and light structure, which reduces the loads on the foundation and by that enables the construction of a more slender foundation.

- **Aesthetics**
  The sub-criteria *Spatial quality* and *Recreational quality* could be improved by the construction of a more tourist attractive structure. *Integration* with the *Landscape* could be improved as well.

In this chapter an alternative design will be presented, for which it has been attempted to improve the above described aspects. The barrier location will not be changed, neither are the channel dimensions nor the type of load transmission to the bearing soil layer. The concept of this Alternative Design will be briefly described in Section 6.2, presenting the shape of the new barrier and a brief introduction of the additional structures.

The remaining sections will contain the same design steps as for Chapter 5. The load distribution for the new design is described in 6.3 after which the stability, strength and vibration requirements will be checked in Section 6.4. This is followed by the selection of a movement mechanism that suits this new concept, presented in Section 6.5. Subsequently the Foundation and Abutments are designed in Sections 6.6 and 6.7 respectively. The final Section of this Chapter contains the Conclusions. For extensive descriptions of all design steps there will be referred to Appendix F.
6.2 Concept

The concept of this alternative design is based on an alternative design for the Maeslant Barrier in Rotterdam, introduced in Section 4.2.2, consisting of a triangular truss constructed out of steel tubes as presented in Figure 6.1. A structure like this is much lighter than the concrete door of the previous Chapter, improving requirements on the foundation and enabling a faster movement of the gate. Additionally, due to the sloping walls the load transmission to the foundation is much more favorable, while vibrations due to wave reflection are expected to reduce as well.

Figure 6.1: Alternative design of the Maeslant Barrier: (a) Lateral view of the sliding gate in operation position [Rigo, 2005] and; (b) Scaled dimensions for the Closed-Open-super dike ring "Rijnmond" [Arecco et al., 2013]

For application at the Houston Ship Channel the in Figure 6.1-(b) presented barrier has to be upscaled to a height which is larger than 30 m, so the barrier requires dimensions of approximately 3 times the dimensions expressed in Figure 6.1-(b). The shape can be more or less the same, having an inclined retaining wall of about 63° at the bay side and 45° inclined tubes at the channel side. To enable the required magnification of the gate for the HSC, the final cross section will consist of three layers of triangles instead of two, resulting in a shape as presented in Figure 6.2 and dimensions as presented in Figure 6.4.

Gate movement

Similar to the design presented in the previous chapter, movement will be enabled by sliding where a guidance rail limits the horizontal displacements of the gate due to the side current. This guidance rail is integrated into the shape of the sill, as presented in Section 6.6. Load transmission to the bearing soil layers is similar as well, where the guidance rail distributes the loads to the pile foundation which subsequently transmits the loads into the soil. It is expected that due to the reduced weight of the barrier a lighter foundation is required and that gate movement requires less power while a faster movement is enabled. For a smooth movement, allowing small variations in the height profile of the guidance rail, it may be required to construct the steel truss in segments which are interconnected by a flexible material like a rubber. This question will be further addressed in Section 6.4.

Landscape Integration

Just like the concrete barrier, this Alternative Design contains a dry dock at Morgan’s Point as described in Section 6.7. To improve aesthetics however, the alternative dry dock will be completely integrated into the design of the levees resulting in an ‘invisible’ dry dock. All required systems and installations to enable gate movement, maintenance and inspection are located in this closed dry dock. Additional structures are present inside and around the dock as well. These structures are introduced at the end of Section 6.7.
6.3 Load definition

The external loads are the ones resulting from the hydraulic head, waves and wind. These loads act on the water retaining slab, which distributes it to steel H-girders, on which the slabs are installed on. Subsequently, the H-girders distributes the external loads and the selfweight of the girders and the retaining slabs to the triangular cross sections presented in Figure 6.4. Finally these of steel tube consisting sections distribute the loads to the foundation. The load transmission through the steel tubes has been modelled using the software MatrixFrame. This software makes use of the fundamental load combination expressed by Equation 6.1 in which as the maximum head difference the sum of the water level difference ($H_{bay} - H_{river}$) plus the wave amplitude has been used, resulting in the loads presented by Figure 6.3. The design value of the load effect $E_d$ for each element is:

$$E_d = \sum_{j=1}^{n} \gamma_{G,j} \cdot G_{k,j} + \gamma_{P} \cdot P + \gamma_{Q,1} \cdot Q_{k,1}$$  \hspace{1cm} (6.1)

In which:
- $G_{k,j}$ Characteristic value of permanent load $j$ (Selfweight of the elements)
- $\gamma_{G,j}$ Partial factor for permanent load $j$: 1.10 [−]
- $P$ Representative value for the pre-stressing load (Only for the concrete elements)
- $\gamma_P$ Partial factor for the pre-stressing load: 1.00 [−]
- $Q_{k,1}$ Characteristic value of the main variable load (Maximum head difference)
- $\gamma_{Q,1}$ Partial factor for the main variable load: 1.43 [−]
The load acts as distributed force $E_d$ on the concrete slab that is constructed on the bay side of the gates. The concrete slab distributes this force to the H-girders which subsequently distribute the load to the four outer beams. This results in the load distribution as presented in this figure, with 4 point loads in both horizontal and vertical direction.

### 6.4 Design of the Truss Gate

The gate will be designed in a triangular shape consisting of multiple small triangles like a truss, as presented in Figures 6.3 and 6.4. The horizontal forces are transmitted to the foundation by the presented triangular supporting cross-sections, which are present every 50 m resulting in 8 ‘segments’.

Figure 6.4: Cross sections of the steel truss structure: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay.

\footnote{The back (river) side of the gate is modelled as a free support, bearing only vertical loads. As a result of this, the internal horizontal loads have to ‘travel’ a long distance, which is not optimal. The foundation is designed like this to minimize the probability of sedimentation of the guidance rail. It is recommended to investigate other alternatives.}
Due to vertical variations in the sill, as a result of construction errors or uneven soil settlements, it might happen that the barrier does not act like how it has been modelled. As introduced at Section 5.2.2, small variations may cause significant changes in the load distribution of the barrier. Figure 6.5 shows the occurring stresses in the truss tubes when one support has disappeared. Compared to Figure 6.9, the maximum stresses become 2 times larger for the situation of a missing support, substantially exceeding the yield strength of the steel.

However, due to the resulting deformations it is not possible to have a situation with less supports than planned. The displacement of the points where a support is missing is about 35 to 40 cm, resulting in support from the foundation slab as long as height profile variations are smaller than these values. A typical height variation of a few centimeters can be expected in the sill profile, which is small enough to eliminate this issue. At the locations where a support is correctly acting, the maximum displacement is about 4 cm. This initial deformation gives the required gap between the sill and the horizontal bottom tubes, which needs to be larger in order to have no movement issues. In the final design, this gap is 10 cm, offering enough space for deformations without impeding the sliding movement of the gate.

According to the above described characteristics, it will not be necessary to have a segmented gate. The gate seems to be flexible enough to respond on variations (< 15 cm) in the sill profile without exceeding the maximum allowed stresses in the gate, while the desired transmission of the loads to the foundation is maintained.

![Figure 6.5](image)

**Figure 6.5:** Occurring stresses in the steel tubes for a 1/1000 years storm surge level, wind speeds and wave impacts when one support has disappeared as a result of variations in the height profile of the foundation slab.

### 6.4.1 Stability

Thanks to the triangular shape this truss barrier is less sensitive to instability, there will always be stability as long as there is a positive hydraulic head: \( H_{\text{bay}} \geq H_{\text{river}} \). This stability can be defined as the rotation around the support at the river side (Support \( K_1 \) in Figure 6.1). In times of a negative head however, the barrier shape may contribute to instability. This can be defined as a negative rotational moment around support \( K_1 \), as expressed by the following equation:

\[
M_{/K_1} = q_{sw} \cdot \Delta y_{qsw} + 1/2 \gamma_w H_{bay}^2 \cdot \frac{H_{bay}}{3} - 1/2 \gamma_w H_{river}^2 \cdot \frac{H_{river}}{3}
\]  

(6.2)

In which:
- \( M_{/K_1} \) \([ \text{kNm/m} ]\) Rotational moment around support \( K_1 \), per running m in x-direction.
- \( q_{sw} \) \([ \text{kN/m} ]\) Vertical load due to the selfweight of the gate
- \( \Delta y_{qsw} \) \([ \text{m} ]\) Leverarm of \( q_{sw} \)
- \( \gamma_w \) \([ \text{kN/m}^3 ]\) Specific weight of water
With $\gamma_c = 10 \text{ kN/m}^3$, $\Delta y_{sw} = 16.5 \text{ m}$ and the gate selfweight $q_{sw} = 750 \text{ kN/m}$ this results in the following limit for the water levels:

$$M_{/K1} = 12375 + \frac{10H_{bay}^3}{6} - \frac{10H_{river}^3}{6} = 12375 - \frac{5}{3}(H_{river}^3 - H_{bay}^3) \geq 0 \quad (6.3)$$

Since $H_{river} = H_{bay} + \Delta H$, the rotational moment around $K1$ can be expressed as a function of the negative head $\Delta H$ for different water levels at the bay side $H_{bay}$, resulting in the graph presented in Figure 6.6. As presented, the maximum allowed head differences are greater for lower water levels in the bay side. During normal conditions the water level at the bay side is $22 \text{ m}$, giving a maximum negative head of about $4.2 \text{ m}$ and a maximum water level at the river side $H_{river} \leq 26 \text{ m}$. As long as the negative head is smaller than the expressed values in the Graph of Figure 6.6 the gate remains stable and thus can be closed. When the water levels at the river side are expected to exceed these limits, the gate has to be opened in a timely manner. From Figure 6.6 it can be concluded that the lower the bay water level, the larger the negative head is allowed.

Figure 6.6: Graph of the rotational moment around $K1$ as a function of the negative head $H_{bay} < H_{river}$, for eleven different water levels at the bay side.

For a 1/100 years surge event a water level of about $28 \text{ m}$ can be found at the bay side. Assuming that the gate is closed $12 \text{ hours}$ before the peak levels of the surge can be found, at a time that the water levels in the channel have raised two meters to a water depth $H_{bay} = 24 \text{ m}$. According to the graph of Figure 6.6 the gate can be closed until a negative head of about $3.6 \text{ m}$ can be found. As determined in Section 2.5.2, the water level elevation at the river side with a closed barrier is $2.5$ to $3.0 \text{ m}$ in $10 \text{ hours}$ due to the rainfall driven river discharge associated with such a storm event. At the peak of the storm, the water depth at the river side may have been elevated to $H_{river} = 24 + 3.0 = 27 \text{ m}$, having a water level of $\text{MSL} + 5.0 \text{ m}$. Which requires the gate to be opened since this is the water level that flooding occurs and tank inundation may be found.

In the above described case it is not the negative head that limits the allowed time of closure of the gate but the water levels. Probably the only aspect for which the negative head may be important is when the amount of time between closure of the gate and the moment that the actual storm surge arrives is too big, resulting in elevating water levels at the river side while the water level at the bay side remains relatively low. The determination of the moment of closure will be a very important activity, where closing should be as short as possible before the storm surge arrives. To prevent heavy loads due to high velocity currents, opening of the gate is best at the moment of equilibrium $H_{bay} = H_{river}$, when the peak surge has passed and the water level at the bay side is decreasing again.
### 6.4.2 Strength

Three elements are distinguished for the strength checks, distributing the loads subsequently towards each other. Each element will be designed in order to provide just enough strength to retain the maximum loads for a 1/1000 years storm event.

1. Concrete water retaining wall.
2. H-girders, bearing the concrete slab and distributing the loads to the triangular cross sections.
3. Triangular cross sections, constructed out of steel circular hollow tubes.

#### Water retaining wall

The water retaining slab will be constructed of High Strength Concrete C90/105. It distributes the external loads to the steel H-girders as presented in Figure 6.7, additionally its self weight is transmitted to the steel girders. The H-girders subsequently transmit all loads in x-direction over two times 25 m to the bearing cross-sections. The occurring stresses in the H-girders due to these loads are presented in Figure 6.8, where H-girders with height \( h_H = 1.0 \text{ m} \), width \( b_H = 2.0 \text{ m} \) and steel thickness \( t_H = 0.20 \text{ m} \) are used.

![Figure 6.7: Transmission of the external design loads \([kN]\) to the concrete slab and the steel H-girders in x-direction: (a) Dimensions of the concrete slab; (b) Loads acting on the concrete slab and resulting support reactions acting on the H-girders; (c) Resulting bending moments in the concrete slab; and (d) Stresses in the concrete slab, the maximum negative concrete stress \( \sigma_{c,max} = 38 \text{ N/mm}^2 \) due to this load combination can be found at the clamped connections with the H-girders.](image)

Figure 6.7 (c) shows the occurring bending moment in the Concrete slab as a result of the external loads and its self weight. The maximum load can be seen in the bottom of the slab, which gives a bending moment equal to 1,600 kNm, resulting in a maximum tension stress at the side of the concrete slab of presented in Equation 6.4. The design tensile strength associated with high strength concrete is \( f_{c,t,d} = f_{c,tm,fl}/\gamma_c \), in which the characteristic flexural tensile strength \( (f_{c,tm,fl}) \) is 7.6 N/mm² and the partial safety factor \( (\gamma_c) \) is 1.5 \([-]\), resulting in a flexural tensile design strength \( f_{c,t,d} = 5.07 \text{ N/mm}^2 \). In combination with the strength requirement \( (f_{c,t,d} > \sigma_{c,t,max}) \) the required concrete thickness can be determined.
To meet the strength requirement, the concrete thickness $t_c$ should be bigger than 1.56 m to have no cracking at all. Unfortunately, the with an increased concrete thickness the self weight and thus the loads will increase as well, leading to bigger stresses and a bigger required concrete thickness. Probably a thickness of about 2 m may result in sufficient strength, but this will give a very heavy design which is not in line with the goal to reduce the robustness and weight of the structure.

This problem may be solved with the use of reinforcement bars and prestressing, as for the sill slabs of the original design in Section 5.6. Assuming that prefabricated prestressed slabs will be used with a length of 12 m, a width of 2 m and 0.6 m thickness, so 25 slabs times 3 layers per 50-section. The drape of the curved fictitious tendon is maximum: 0.4 m. Resulting in a concrete cover of 0.10 m and an eccentricity of 0.20 m for both the support and halfway the span. For these slabs it is required that the maximum tensile stress is not larger than 5 N/mm$^2$, with equations 5.15 resulting in the following conditions:

$$\frac{P_\infty}{A_c} - \frac{P_\infty \cdot \varepsilon_p}{W_t} + \frac{M_{Ed}}{W_t} \leq 5.00$$

In which:
- $P_\infty$: Required prestressing force at $t = \infty$ tbd [kN]
- $A_c$: Cross-sectional area of the concrete slab 1.2 [m$^2$]
- $\varepsilon_p$: Eccentricity from fictitious tendon to neutral axis 0.20 [m]
- $W_t$: Bending moment resistance at topside of the slab 0.03 [m$^3$]
- $M_{Ed}$: Design value of the bending moment 2060 [kNm]

With the presented values for the described parameters, a minimum amount of prestressing $A_{p,req} = 12,740$ mm$^2$ per 2 m wide slab. Using prestressing tendons consisting of 7 strands $\Phi_{strand} = 13$ mm ($A_{strand} = 140$ mm$^2$), an amount of 13 tendons is required per slab. Considering the relative small cross-section, it is probably not profitable to use such an amount of prestressing steel and thus the used slab dimensions or material are not sufficient. Considering the maximum compression stress in the slab, it is not possible at all to use such an amount of prestressing. Without external loading, the compression stress as a result of the prestressing would be bigger than the compression strength.

Apparently a design with a 0.50 m thick concrete slab is not sufficient for this situation. Probably a design with a water retaining slab constructed out of steel, some kind of plastic or microfibers might be better. This should be investigated in more detail. For now the load combination associated to the 0.50 m thick concrete slab and a 1/1000 years storm event will be used as load for the supporting steel elements.

**Steel H-Girders**

To distribute the design loads from the concrete retaining wall to the bearing cross section, the steel H-Girders need to have a great strength. A design for which the girder height $h_{girder} = 3000$ mm, flange width $b_{girder} = 2000$ mm and all steel thickness is everywhere $t_{girder} = 300$ mm is sufficient to resist these loads combined with the own self weight of the girders, if steel class S355 is used. The occurring stresses and load distribution for this design are presented in Figure 6.8. These are very heavy girders, which are not standardized. To conclude, this designed solution provides sufficient strength, but the economical value can be questioned. It might be a better solution to have a much lighter structure for the retaining wall, reducing the loads on the H-girders. This recommendation is in line with the recommendation concluded from the design of the concrete retaining wall.
Figure 6.8: The occurring stresses in the H-girders for the 1/1,000 years load combination. As presented is the maximum stress about 300 $N/mm^2$, which is smaller than the design strength of steel, so the strength of these H-girders is sufficient.

Bearing Cross-section consisting of Steel Tubes

Each 50 m a triangular cross section is present consisting of multiple smaller triangles, as described in the beginning of this section and presented in Figure 6.4. The members of these triangular cross sections are hollow steel tubes which distribute the external loads to the foundation. For the majority of the steel tubes a steel class S355 is sufficient. For the bottom tube (members S4, S12 and S17 in Figure 6.9) however, a higher steel class is required. This tube will need a steel class S460, to have sufficient strength for the same tube dimensions as the other members. The diameter of the tubes in this bearing cross section $\Phi_{tubes,y} = 1.4 \, m$ and the thickness of the steel $t_{tube,y} = 100 \, mm$.

Figure 6.9 shows the loads acting on the steel truss, resulting from the H-Girders. Figure 6.9-(a) shows the resulting stress in each member, the related actual values are presented in Table 6.1. As presented, the maximum stress $\sigma_{s,max} = 425 \, N/mm^2$ can be found in member S4. The regular steel class S355 is not sufficient to prevent this member from yielding, therefore the complete bottom beam is constructed out of S460 steel. A yield strength $f_y = 460 \, N/mm^2$ is characteristic for this class, which is sufficient for load transmission. The resulting reaction forces in the sill are presented in Figure 6.9-(b), where all horizontal loads are transferred to the support at the bay side. It can be concluded that the presented design of the truss will be stiff and strong enough to resist the design load, associated with a 1/1,000 years storm event.

The bearing cross-sections are interconnected in transverse (x) direction by hollow steel tubes S355, $\Phi_{tubes,x} = 1.22 \, m$ and the thickness of the steel $t_{tube,x} = 25 \, mm$. At the two bottom corners, O1 and O2 in Figure 6.9-(b), a rectangular steel beam with sides of 2.5 m is present on which the driving mechanism acts to move the gate. Additionally, these rectangular beams give additional strength in x-direction and divide the support reactions over a 4 m wide foundation area.

In order to check some kind of 'worst-case scenario', the model is defined in a way that the support at the bay side bears all horizontal loads. In reality the other side may bear some of the horizontal loads as a result of friction with the foundation. Simulation with all horizontal loads on one side results in the maximum stresses in the steel members, resulting in higher strength requirements.
Figure 6.9: Maximum loads acting on the Bearing Sections of the Steel Truss Barrier, for a 1/1000 years storm: (a) Resulting stresses in the steel members; (b) Resulting support reactions, acting on the foundation. Note that members S4, S12 and S17 are parts of a tube which needs the higher steel class S460, where all other members are constructed out of S355 Steel. The actual values of the loads are presented in Table 6.1.

<table>
<thead>
<tr>
<th>Member</th>
<th>Normal Force $N_y$ [kN]</th>
<th>Stress $N/mm^2$</th>
<th>Member</th>
<th>Normal Force $N_y$ [kN]</th>
<th>Stress $N/mm^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
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<td>-76</td>
<td>S10</td>
<td>52,700</td>
<td>129</td>
</tr>
<tr>
<td>S2</td>
<td>-25,900</td>
<td>-63</td>
<td>S11</td>
<td>48,700</td>
<td>119</td>
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<tr>
<td>S3</td>
<td>1,640</td>
<td>4.0</td>
<td>S12</td>
<td>116,000</td>
<td>284</td>
</tr>
<tr>
<td>S4</td>
<td>186,000</td>
<td>450</td>
<td>S13</td>
<td>-36,200</td>
<td>-89</td>
</tr>
<tr>
<td>S5</td>
<td>-41,700</td>
<td>-102</td>
<td>S14</td>
<td>-34,200</td>
<td>-84</td>
</tr>
<tr>
<td>S6</td>
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<td>-78,700</td>
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<tr>
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<td>S9</td>
<td>-17,200</td>
<td>-42</td>
<td>S18</td>
<td>-113,000</td>
<td>-277</td>
</tr>
</tbody>
</table>

6.4.3 Dynamics

As mentioned before, the gate structure is very flexible depending on whether or not supports are missing. For the situation with all supports the typical displacement that can be seen for most elements is about $\delta_0 \approx 0.04 \text{ m}$ in all directions for a maximum head difference in and out of the bay of 8.0 $\text{ m}$ plus the wave loading. As presented in Figures 6.3 and 6.9(b), the total horizontal load acting on the steel gate is about $216 \text{ MN}$ per section of 50 $\text{ m}$. With the simplified formulation of the stiffness as presented in Equation 6.6, in which horizontal load $F_y = 216 \cdot 10^3 \text{ kN}$ and horizontal deformation $\delta_y = 0.04 \text{ m}$, this gives a stiffness of $k_{y,0} = 5.40 \cdot 10^6 \text{ kN/m}$. When one support is missing however, the displacements become 50 times larger resulting in $\delta_{y,s} \approx 2.0 \text{ m}$, which can be associated with a stiffness of $k_{y,s} = 1.08 \cdot 10^8 \text{ kN/m}$, but since the barrier will than be supported again this situation will not occur in reality. See Figure 6.10 for a presentation of the maximum nodal displacements, resulting from MatrixFrame modelling software.
\[ k_{y,0} = \frac{F_y}{\delta_y} \]  \hspace{1cm} (6.6)

The dead weight of 50 m barrier is about 37.5 \( \cdot \) 10^3 kN, which in combination with the above described base stiffness \( k_{y,0} \) in a eigenfrequency \( p = \sqrt{K/M} \approx 3.8 \text{ rad/s} \) or 0.60 Hz. With the in Chapter 5 determined load frequency of \( \omega_{\text{waves}} \approx 0.70 \text{ rad/s} \) (0.11 Hz) it can be concluded that the flexibility of the gate does not result in any issues related to resonance, according to the resonance criterion expressed by Equation 5.9.

The gate slides at a smooth surface, to enable a proper movement of the gate. This surface however, will therefore not be able to provide any damping. Damping may be provided by both abutments, at the ends of the gates. This should be investigated in more detail.

The floatability check presented in Appendix F shows that the selfweight of the total barrier is about 300 \( \cdot \) 10^3 kN. The combination of hollow tubes, H-girders and water retaining wall has a total submerged volume of 18,190 m^3. The same amount of water has a self weight of 183 \( \cdot \) 10^3 kN < 300 \( \cdot \) 10^3 kN, so the structure is not floating by itself. The decision has been made not to use any aid measures to make the gate floating, since a floating unit is very sensitive to the water motion which may be rough due to the storm winds. Additionally, complete filling of the dry dock with water is not required for gate movement, which reduces the closing time.

The chosen system to move the barrier is by the so-called rack and pinion: Pushing the gate away from the dry dock to the other side, using rotating cogwheels which are installed at the dry dock (Figure 6.11(c)). The prongs of these cogwheels fit exactly in the ribbed outside of the gate elements that slide over the foundation to make the complete gate be sliding over the sill. The problem of siltation will be eliminated by the use of a specially designed head of the gate, inspired by a classic steam train head, as presented in Figure 6.11(a). The failure mechanism associated to uneveness’s of the rail has been addressed in the beginning of Section 6.4 which should not be a problem thanks to the flexibility of the truss. The gate can be constructed as one structure, reducing the complexity of the movement mechanism.

Figure 6.10: Presentation of the maximum deformations of the truss due to the head difference of 8.0 m in combination with waves loading. Values are given in meter.

### 6.5 Design of the Movement Mechanism

The floatability check presented in Appendix F shows that the selfweight of the total barrier is about 300 \( \cdot \) 10^3 kN. The combination of hollow tubes, H-girders and water retaining wall has a total submerged volume of 18,190 m^3. The same amount of water has a self weight of 183 \( \cdot \) 10^3 kN < 300 \( \cdot \) 10^3 kN, so the structure is not floating by itself. The decision has been made not to use any aid measures to make the gate floating, since a floating unit is very sensitive to the water motion which may be rough due to the storm winds. Additionally, complete filling of the dry dock with water is not required for gate movement, which reduces the closing time.

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Figure 6.11: Elements considering the movement of the gate: (a) A classic cattle guard at a steam train; (b) Impression of sediment guard at the gate front-end; (c) Rack and pinion movement mechanism.

6.6 Design of the Foundation

The type of foundation will be the same for the Composite Truss Barrier as for the Concrete Roller Door: A pile foundation consisting of driven steel close-ended tubular piles, bearing concrete slabs which have the function of sill and rail. However, the vertical loads are significant smaller for the Composite Truss Barrier, resulting in more slender piles.

6.6.1 Sill

At the bottom of the channel a slender concrete slab is installed on top of a pile foundation, as presented in Figure 6.13. This slab provides a smooth surface for the gate to move on and has the function of distributing the loads from the gate to the pile foundation.

External loads

The load bearing elements are the two thicker parts at each side of the sill, where the pile foundation is installed. These two sill ends are called the footings of the sill. The horizontal loads will be acting at the bay side on the foundation. The piles at this side will bear the major part of the horizontal loads, but a significant part is expected to be transmitted by the center part of the slab to the other side, where transmitted to the soil by the remaining piles. This causes a compression force in the center of the sill. The maximum horizontal design load is 216,100 $kN$, acting on the sill each 50 m.

As presented in Figures 6.3 and 6.9, the maximum vertical loads due to the $1/1,000$ years storm event are 53,600 $kN$ and 79,800 $kN$ for the bayside and channel side respectively. To determine the total vertical design loads, the self weight of the barrier, the selfweight of the sill and the weight of the water column on top of the sill have to be added. This results in total vertical loads of 107,400 $kN$ and 133,200 $kN$ per 50 m footing, for the bayside and channel side respectively.

Internal load transmission

The expected internal load transmission through the sill is presented in Figure 6.12. As presented, it is expected that the majority of the vertical loads $\sigma_V$ will be distributed to the piles directly underneath the acting load. It is assumed that about 50 percent of the horizontal load $\sigma_H$ is distributed to the other piles on this side, and that the other 50 percent will be distributed through the central part of the sill towards the footing on the other side, as presented in Figure 6.12(b). Combining this assumption with the top view of the sill, presented in Figure 6.12(a), the horizontal stress at the other side of the sill ($\sigma_{H,R}$) can be determined:
\[ \sigma_{H,R} = \frac{F_{H,R}}{A_{sill}} \] (6.7)

With the distributed load \( F_{H,R} = 0.5F_H = 108,050 \text{ kN} \) and minimum effective cross-sectional area of the sill \( A_{sill} = 2 \cdot 50 = 100 \text{ m}^2 \), this results in a maximum internal stress \( \sigma_{H,R} = 1,080 \text{ kN/m}^2 \), which is equal to about 1.08 \( \text{N/mm}^2 \). This is much smaller than the concrete compressive strength \( f_{c,cd} = 30 \text{ N/mm}^2 \) for Concrete Class C45/55. With a maximum vertical stress at the sill \( \sigma_{V,R} \approx 0.85 \text{ N/mm}^2 \), for which an effective cross-sectional area \( A_{foot} = 12.5^2 = 156 \text{ m}^2 \) is used, it can be concluded that the concrete compressive stresses can be easily transmitted by the sill.

Figure 6.12: Expected internal load transmission for the sill. Image (a) shows a top view of a 50 m wide part of the sill with the horizontally acting external load. The image shows that the sill is wide enough to have an equally distributed horizontal stress \( \sigma_H \) on the other side of the sill as a result of the horizontal load. Cross-section A-A’, Image (b), shows a detail of the footing at the bay side, in which both the horizontal and vertical load transmission is presented.

Additionally, Figure 6.12(a) introduces a splitting force \( N_{spl} \), resulting from the horizontal load spreading out over the sill. According to Walraven and Braam (2012) this splitting tensile force is required to have force equilibrium in transverse direction, expressed as follows:

\[ N_{spl} = \frac{1}{4} F_P \left( 1 - \frac{h_1}{h} \right) \] (6.8)

For a width of the load \( h_1 = 4 \text{ m} \), a concrete slab width \( h = 50 \text{ m} \) and a point Load \( F_P = F_{H,R} \), this results in a splitting tensile force \( N_{spl} \approx 25,000 \text{ kN} \). Splitting reinforcement, also known as stirrups, is required to bear this load. The amount of splitting reinforcement \( A_{spl} \) depends on the maximum allowed steel stress \( \sigma_s \). To avoid crack width problems, a maximum allowed steel stress \( \sigma_s = 200 \text{ N/mm}^2 \) is used, resulting in \( A_{spl} = 125,000 \text{ mm}^2 \). A design consisting of 60 transverse bars \( \phi = 52 \text{ mm} \) is sufficient. These bars can be divided over a concrete width of 6 m, resulting in 5 layers of bars each 0.5 m. An indication of the locations of these stirrups has been given in Figure 6.12(a), where the reinforcement close to the right side is applied in order to resist unforeseen loads like impact loads or forces from differential shrinkage (Walraven and Braam 2012). The design of the other types of reinforcement is presented in Appendix G Foundations, Section G.4.3.
Seepage

Besides load transmission, the slab has the function of preventing soil instability due to porous flow under the structure called seepage, which may cause piping or heave. To prevent this instability, the seepage length $L_{s,req}$ has to be bigger than the hydraulic gradient multiplied by a coefficient defined as Bligh’s coefficient depending on the soil type. Lane (1934) gives for a clay layer a value $C = 2.0$, where the hydraulic head $H = 8.0$ m. According to Lane (1934), the required safe flow length $L_{s,req} = CH = 16$ m. Lane’s Weighted Creep Theory (1934), contains the conclusion that horizontal creep is less effective in reducing uplift than vertical creep. Therefore, this theory uses a factor $\gamma_h = 1/3$ for the horizontal creep, where vertical creep is multiplied with the factor $\gamma_v = 1$, resulting in the formula for total seepage length as presented in Equation 6.9. With the used design, this results in a length of 28 m, which is sufficient.

$$L = \Sigma L_{h,i}/3 + \Sigma L_{v,i} > CH$$  \hspace{2cm} (6.9)

6.6.2 Pile foundation

As introduced, the sill is supported by a pile foundation. Similar to the foundation of the Concrete Door Alternative, this pile foundation will consist of steel close-ended tubular piles. With the above expressed loads per sill footing and 1:3 inclined piles $\Phi_t = 2.0$ m, a configuration of 6 piles per footing is sufficient to bear both the horizontal and the vertical loads. This results in the design presented in Figure 6.13 in which the piles directly under the gate support beam have an inclination of 1:3 in x-direction to bear horizontal loads associated with movement of the gate. The presented line of piles is present each 16.7 m, resulting in 3 rows of piles for each 50 m gate section and for the total 900 m sill length a total of 54 rows consisting of 12 piles each. The sill footing dimensions are somewhat reduced compared to Figure 6.12 as a result of (punching) shear checks presented in Appendix G.

![Figure 6.13: Design of the foundation for the steel barrier alternative. The initial soil stresses, required input for the pile foundation design, are presented on the right. Note that the two light grey pillars directly under the footings are inclined in x-direction to bear movement related horizontal forces.](image)
6.6.3 Protection against erosion

As a result of the river flow or sailing vessels, erosion of the channel bed and banks may occur. It has been mentioned that erosion around the structure can cause instability and failure of the structure. It is therefore important to have a scour hole far away from the structure. The bed and bank protections presented in Section 5.6.3 are expected to be sufficient for this alternative design too.

6.7 Design of the Abutments

As mentioned in the beginning of this section, this alternative is designed with the main purpose of improving the criteria on which the original concept is lacking. The structural requirements have been addressed in the previous phrases, one criterion that still can be improved is Aesthetics. The design of the abutments can greatly contribute to the improvement of this criterion, by reducing the required amount of space and integrating the abutments in the landscape. In order to end with a coherent overall design, the shapes of all support structures will be inspired by the triangular shape of the gate.

6.7.1 Abutment heads

Both abutment heads will be similar to each other, just like the original concept. But different to the original concept, there will not be any significant load transmission to the abutment heads. Where for the original design a horizontal reaction force near the top of the gate may occur, the alternative concept will have all reaction forces in the bottom of the gate at the sill structure. The function of the abutment heads for this concept will only be the reduction of the amount of water leakage, while integrating the gate into the landscape. The shape of the abutment heads follows the functional requirements, resulting in triangular heads.

6.7.2 Dry dock

For this concept, the dry dock will be integrated into the levee system. The dry dock will be a closed concrete box structure, which is covered by soil and grass resulting in a grass-covered dike. At the location of the dock, which is close to the water line, all dike designs are inspired by the design for levee section 2, as presented in Figure 3.6. To adapt the height of the gate, of which the top is at MSL + 11.0 m, and some additional required free space between the ceiling of the dock and the top of the gate of 2.0 m, the inner height of the dock will be about 35 m measured from the top of the foundation slab. The crest height of the overlying levee structure then depends on the required thickness of the top slab of the dock and the thickness of the soil layer on top of it.

Design of the Roof Structure

According to Wagemans et al. (2004) the 60 m wide dock can best be covered by either a truss frame or multiple truss girders, consisting of steel S355. In the final design, the top slab will consist of multiple steel N-girders, bearing the concrete roof which consists of hollow core slabs, and is covered by a soil layer. To protect the roof structure, a thin impermeable geotextile layer is present between the soil and the concrete slabs. For the design load combination of Equation 6.10 and the dimensions presented in Figure 6.14 the resulting design pressure \( p_{d,i} = \Sigma G_j \gamma G + Q \gamma Q \) (6.10) and design load \( q_d = 183 \text{kN/m} \) distributed over the girders, for girders which are installed every 8 m. To provide sufficient load bearing capacity, the N-girders are constructed out of H-profiles HE500B and have a total height \( h_{girder} = 3.5 \text{m} \), as presented in Figures 6.14(a) and (b).
In which:

- $p_{d,i}$: Design load (pressure) acting on element $i$. TBD [kN/m$^2$]
- $G_j$: Dead load from the self weight of construction element $j$. TBD [kN/m$^2$]
- $\gamma_G$: Load factor for the Dead Loads. 1.2 [-]
- $Q_v$: Variable load, representative for industrial buildings. 5.0 [kN/m$^2$]
- $\gamma_Q$: Load factor for the variable load. 1.50 [-]

Figure 6.14: Roof structure of the Dry Dock: (a) Dimensions of the main girders [m]; (b) Stress distribution at the main girders [N/mm$^2$]; (c) Dimensions of the concrete hollow core slabs [mm], which are installed on top of the main girders.

The hollow core slabs will be prefabricated elements with a length of 8.0 m, a width of 1.2 m and a thickness of 0.22 m as presented in Figure 6.14(c). These prefabricated concrete slabs C45/55 are fully prestressed by linear tendons of pre-tensioned steel Y1860S7, and are freely supported by the steel N-girders. The amount of prestressing has to meet the following two requirements, considering the maximum tensile stresses allowed in the concrete: (Fennis, 2012) (Walraven and Braam, 2012)

$$
t = 0 : \quad \sigma_{ct, sup} < 0 = -\frac{P_{m0}}{A_c} + \frac{P_{m0} \cdot e_p}{W_t} + \frac{M_{Ed, sup}}{W_t} < 0 \quad (6.11)$$

$$
t = \infty : \quad \sigma_{cb, L/2} < 0 = -\frac{P_{m\infty}}{A_c} - \frac{P_{m\infty} \cdot e_p}{W_b} + \frac{M_{EdL/2}}{W_b} < 0 \quad (6.12)
$$

In which:

- $\sigma_{ct, sup}$: Occurring stress in the top of the slab at the support [N/mm$^2$]
- $\sigma_{cb, L/2}$: Occurring stress in the bottom of the slab halfway span $L$ [N/mm$^2$]
- $P_{m0}$: Prestressing force at $t = 0$ [N]
- $P_{m\infty}$: Prestressing force at $t = \infty$; assumed: $P_{m\infty} = 0.8P_{m0}$ [N]
- $A_c$: Cross-sectional area of hollow core slab $141.8 \cdot 10^3$ [mm$^2$]
- $e_p$: Eccentricity prestressing steel to neutral axis 0 [mm]
- $W_t$: Section modulus (top fibre) $7.783 \cdot 10^6$ [mm$^3$]
- $W_b$: Section modulus (bottom fibre) $7.923 \cdot 10^6$ [mm$^3$]
- $M_{Ed, sup}$: Design bending moment at the supports 0.0 [kNm]
- $M_{EdL/2}$: Design bending moment halfway span $L$ 182 [kNm]
Where the first requirement gives the maximum eccentricity to have no tensile stresses in the concrete at the supports, resulting in $e_p < 55 \text{ mm}$. This maximum eccentricity $e_p = 55 \text{ mm}$ will be used in order to have a required amount of prestressing steel which is as small as possible, while meeting both the requirements. The second requirement results in a minimum prestressing force to have no tensile stress in the bottom of the girder halfway the span of 8.0 m:

$$P_{m0} > 2052 \text{kN} \tag{6.13}$$

With prestressing steel Y1860S7, where $\sigma_{mn0} = 1395 \text{ N/mm}^2$, this results in a minimum required amount of steel $A_p = 1471 \text{ mm}^2$. Where tendons are used consisting of 7 strands $\Phi_{\text{strand}} = 12.7 \text{ mm}$ and $A_{\text{strand}} = 98.71 \text{ mm}^2$ each, 3 tendons are required. Resulting in a prestress force $P_{m0} = 2892 \text{ kN}$. This may be a little bit over designed, a more economical design would have only 2 tendons $A_p = 1382 \text{ mm}^2$, allowing some tension halfway the girder at $t = \infty$. This amount of tension will be $\sigma_{cb,L/2} = 1.39 \text{ N/mm}^2$, which is smaller than the concrete tensile strength $f_{ctd} = 3.8 \text{ N/mm}^2$. A design with 2 tendons is sufficient, depending on the maximum tensile stress requirements.

For the described design of prestressing, the maximum compression stress can be found in the bottom of the concrete elements at $t = 0$, when there is no external load. For concrete class C45/55, the design compression strength $f_{cd} = 45/1.5 = 30 \text{ N/mm}^2$, which is a negative stress. As presented in Equation 6.14, the maximum compression stress that occurs in the slabs due to prestressing is smaller than the design compression strength. The used design does not give any issues considering the compression strength.

$$\sigma_{cb,t=0} = -\frac{P_{m0}}{A_c} - \frac{P_{m0} \cdot e_p}{W_b} = -\frac{1.928 \cdot 10^3}{141.8 \cdot 10^3} - \frac{1.928 \cdot 10^3 \cdot 55}{7.923 \cdot 10^6} = -27 \text{ N/mm}^2 \tag{6.14}$$

The designed bearing structure of the top of the dry dock has a total height of 4 m, resulting in a top level at $MSL + 17 \text{ m}$. This top will be the crest of the dike, covering the dry dock. The resulting dike height is 11 m, relative to the local ground surface level at $MSL + 6.0 \text{ m}$.

### 6.7.3 Additional structures

Figures 6.15 and 6.16 show the presence of a building on top of each abutment. Each structure has different functions, resulting in the two different designs. The overall design is inspired by the Thames River Barrier (Figure 4.8) in the United Kingdom and the Rubber Barrier at Ramspol in the Netherlands, presented by Figure 4.9, which both have characteristic abutment heads resulting in barriers which are easily recognizable. This improves the barrier design in terms of aesthetics and will attract tourists.

Both mentioned barriers, by which this concept is inspired on, have both round shapes, following the shape of the present gate. The shape of the abutment structures at the Houston Ship Channel will follow the shape of the steel gate as well, resulting in two structures with sharp edges comparable to the Sidney Opera House, Australia. These structures will both be symmetrical and are constructed on top of the abutments, in line with the alignment of the gate.

The final design of the specific structures follows its associated functions, a strategy defined by the American architect Louis Sullivan (1856-1924) as: “Form Follows Function” (sull 1896). Shared functions of both sides are accessibility to the abutments and observation point for both engineers (inspection) and tourists (recreation). To enable these shared functions, each structure will have its orientation directed towards the channel and be placed very close to the abutment head. Subsequently the roof of each structure will be inclined to drain rainfall water and to improve the landscape integration. The structure at the side of the dry dock will have some additional functions, which makes this structure the bigger one of the two. These additional functions are accommodation of the following types of rooms and spaces:

- Technical rooms:
  - Offices, the control center, a power room, equipment storage, toilets,
Visitor center:
A museum, offering information about the construction and operation of the gate to tourists; toilets; and a cafeteria, offering visitors food and drinks.

6.7.4 Overview
An overview of the barrier, the abutments and some of the additional structures can be seen in Figure 6.15. This figure presents a front view of the gate in closed position (A-A’) and a cross section at the gate and the dry dock (B-B’) as well. Dimensions and height levels are given as well. Figure 6.16 gives a Google Sketchup impression of the front view of the gate in closed position, which is representative for drawing (A-A’) in Figure 6.15.

Figure 6.15: Drawing of the overview of the steel gate with the two abutments. Sideview A-A’ shows the closed gate, the levees and the building. Cross-section B-B’ shows the closed dry dock, the gate, the foundation and a cross-section of the buildings.

Figure 6.16: Impression of the Steel Truss Barrier, for front view A-A’ of Figure 6.15.

140 CHAPTER 6. ALTERNATIVE DESIGN: HORIZONTALLY SLIDING STEEL TRUSS
6.8 Conclusions

In this chapter an alternative for Houston Ship Channel Barrier has been designed, combined with matching designs for the movement mechanism, foundation and abutments. The alternative design contains a horizontally sliding truss gate, consisting of steel tubes and a concrete water retaining wall on the bay side to retain storm induced surge levels up to $\text{MSL} + 8.0$ m. Thanks to the triangular shape of the gate, the external loads can nicely be distributed through the steel elements to the foundation and the stability of the gate is guaranteed.

During movement, the triangular gate will be guided through the channel by a guidance rail, which is integrated into the shape of the sill. Thanks to the flexibility of the gate, deformations of a few centimeters are allowed to adapt to any present variations in the height profile of the sill. Additionally, its flexibility contributes to the desired load transmission. This enables one to construct the gate as one piece, segmentation is not necessary.

The gate will have a total height of 33 m, measured from the top of the sill. The gate has a width of about 50 m and a total length of 400 m, by which the 335 m wide channel can be blocked while the additional length of more than 30 m on each side minimizes the amount of leakage as much as possible. Each 50 m a triangular cross section is present consisting of multiple smaller triangles. The members of these triangular cross sections are hollow steel tubes $\Phi_{\text{tube},y} = 1.4$ m, thickness of the steel $t_{\text{tube},y} = 100$ mm and steel class S355, except for the bottom tube which needs the higher steel class S460. These bearing cross-sections are interconnected in transverse direction by hollow steel tubes S355, $\Phi_{\text{tube},x} = 1.22$ m and steel thickness $t_{\text{tube},x} = 25$ mm. At the two bottom corners, a rectangular steel beam with sides of 2.5 m is present on which the driving mechanism acts to move the gate. Additionally, these rectangular beams give additional strength in x-direction and divide the support reactions over a 4 m wide foundation area. It can be concluded that the resulting design of the truss, as presented in Figures 6.4 and 6.9, is stiff and strong enough to resist the design loads associated with a 1/1,000 years storm event. The water retaining slab needs to be designed in more detail, from the computations expressed in this chapter it can be concluded that rectangular concrete slabs may not be an appropriate solution.

For further conclusions, the purpose of the development of an alternative design is repeated first: Improvement of the concrete concept. As presented in the Introduction of this Chapter, the following aspects have to be improved compared to the Technical Design presented in Chapter 5.

- **Hydraulics:** Dynamics (vibrations due to wave impact)
- **Structure:** Foundation (might be lighter)
- **Aesthetics:** Spatial Quality, Recreational Quality and Landscape Integration.

The above described aspects have been improved by the Alternative Design with the use of the characteristics described in the following phrases.

**Steel Tubes**

The use of steel tubes greatly reduces the dead weight of the barrier and therefore the total vertical load at the foundation structure. This reduces the foundation requirements resulting in a more economically feasible foundation design.

**Triangular Shape**

The triangular shape of the Steel Truss Gate greatly improves the load transmission to the foundation and it contributes to the stability of the structure. Additionally, it results in less wave impact and wave reflection compared to a vertical wall. According to this, a reduction of vibrations of the barrier is expected, which is an improvement related to Dynamics.
Additional Structures

The design of the two abutments contributes to both landscape integration and spatial quality. The architecture can make the gate a recognizable structure that can attract tourists, students and engineers. A museum is added to the design, in which hurricane history at the Houston-Galveston Region and the development of the Houston Ship Channel Barrier can be displayed to improve either flood risk education or recreational quality. Additionally the choice has been made to integrate the dry dock into the levee system, resulting in optimal landscape integration. The additional structures are presented in Figure 6.17.

![Image of barrier in closed position with additional structures](image)

Figure 6.17: Impression of the barrier in closed position with some additional structures: The levees, museum and operation building, access roads and a parking lot.

An overview of technical aspects of the two barrier alternatives is given by Table 6.2 to enable a comparison. It has been made clear that the occurring loads are much smaller for the Steel Truss Alternative. Surprisingly, the foundation slab of the steel alternative has a much larger volume per stretching meter in x-direction (channel cross section), which is because of the much larger width required to support the gate bottom. The concrete foundation slabs however, require a much larger amount of prestressing to have a large enough moment capacity. Because of this, in combination with a lighter pile foundation for the total area, it is expected that the foundation of the steel alternative is more economical. Finally it is expected that the driving mechanism for gate movement requires less power for the steel alternative due to the smaller resistance.

Overall it can be concluded that this alternative technical design is a great option for solving the flood risk problem at the Houston Ship Channel. It enables a solid protection against 1/1000 years storm events, while the natural character of the landscape has been maintained. In addition to protection for the society, industry and environment, this alternative design offers added value in terms of recreational and spatial quality. Environmental interference has been minimized and new habitat for animals is created. Although the global design of the complete protection system clearly distinguishes recreation (people), navigation (business) and environment, an integrated protection for all actors has been achieved.

142 CHAPTER 6. ALTERNATIVE DESIGN: HORIZONTALLY SLIDING STEEL TRUSS
### Table 6.2: Characteristics of the two proposed alternatives for the Horizontally Sliding Gate Barrier.

<table>
<thead>
<tr>
<th>Alternative:</th>
<th>Concrete Door</th>
<th>Steel Truss</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Gate characteristics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.1 Height of the gate (top level)</td>
<td>35 m (MSL + 13 m)</td>
<td>33 m (MSL + 11 m)</td>
</tr>
<tr>
<td>1.2 Width of the gate</td>
<td>18.0 m</td>
<td>49.5 m</td>
</tr>
<tr>
<td>1.3 Selfweight of the gate (dry)</td>
<td>8,250 - 11,550 kN/m</td>
<td>1,250 kN/m</td>
</tr>
<tr>
<td>1.4 Reinforcement ratio</td>
<td>0.2%</td>
<td>N.A.</td>
</tr>
<tr>
<td>1.5 Structural elements [m]</td>
<td>Slabs C70/85 $t_{c,h} = 1.3$</td>
<td>y-tubes S355 $\phi_y = 1.4$, $t_{s,y} = 0.10$</td>
</tr>
<tr>
<td></td>
<td>x-walls C70/85 $t_{c,x} = 1.7$</td>
<td>x-tubes S355 $\phi_x = 1.22$, $t_{s,x} = 0.025$</td>
</tr>
<tr>
<td></td>
<td>y-walls C70/85 $t_{c,y} = 1.7$</td>
<td>H-gir. S355 $h_H = 3.0$, $b_H = 2.0$, $t_H = 0.2$</td>
</tr>
<tr>
<td>Retaining wall C90/105 $t_{c,wrs} = 0.5$</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>2. Foundation characteristics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.1 Sill volume C70/85</td>
<td>130 m$^3$/m</td>
<td>200 m$^3$/m</td>
</tr>
<tr>
<td>2.2 Reinforcement ratio</td>
<td>0.2%</td>
<td>2.0%</td>
</tr>
<tr>
<td>2.3 Prestressing steel $A_p$</td>
<td>$133 \cdot 10^3$ [mm$^2$]</td>
<td>N.A.</td>
</tr>
<tr>
<td>2.4 Pile configuration ($n \cdot \Phi_{t} - \Delta x$)</td>
<td>$12 \cdot \Phi_{t} 2.5 - 20$ [m]</td>
<td>$12 \cdot \Phi_{t} 2.0 - 16.7$ [m]</td>
</tr>
<tr>
<td><strong>3. Characteristics related to gate movement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.1 Movement mechanism</td>
<td>Hydrofenders with pulleys</td>
<td>Rack and pinion</td>
</tr>
<tr>
<td>3.2 Gate head area (resistance)</td>
<td>630 m$^2$</td>
<td>400 m$^2$</td>
</tr>
</tbody>
</table>
Chapter 7

Conclusions and recommendations

7.1 Introduction

In this report a global design of a protection system is presented, consisting of levees, an environmental dune section and a navigational barrier. For the navigational barrier two alternatives are proposed, for which the technical designs are made. In this final chapter the main problem will be reviewed, reflecting on the objectives and research questions presented in Chapter 1. These research questions have all been answered during the different chapters of this report, which result in the conclusions summarized in Section 7.2. At the end of that section, conclusions are presented resulting from an evaluation of the proposed solutions for the main problem. Finally recommendations for further research and design studies are presented in Section 7.3.

7.2 Conclusions

To answer the main question, resulting in reaching the main objective of the development of a design of the Houston Ship Channel Barrier, the subquestions will be answered subsequently, giving a number of conclusions. Finally an evaluation of the two proposed solutions for the Houston Ship Channel Barrier will made, related to the specific objectives as formulated in Chapter 1.

Research questions

1. *Which previous research and design works have been executed already?*

   A great number of possible structures has been proposed the past few years, resulting from different research and design studies, giving protection strategies both inside or outside Galveston Bay. However, structures on the outside of Galveston Bay do not result in enough surge reduction at the entrance of the Houston Ship Channel, a significant residual surge can be found due to local wind set up. From this the conclusion can be made that it is definitely required to have a structure inside of Galveston Bay, protecting the most vulnerable areas.

2. *What are the local design and boundary conditions?*

   An excessive overview of the design and boundary conditions has been presented at the end of Chapter 2. This can be best summarized according the 3 following characteristics: (1) *Heavy weather conditions:* The frequency of hurricanes in along any 80 km segment of the Texas Coast is about 1 in 6 years and on average 4 major hurricanes (category 3 or higher) are expected to occur each century in the Houston-Galveston Region, provoking severe weather conditions like excessive rainfall events, devastating storm surges and great wind velocities; (2) *Critical infrastructure:* The Houston-Galveston region accommodates a very dense population, critical
industries of national importance and valuable ecosystems, making it one of the most valuable and flood-prone areas in the United States; and (3) Geology: The Greater Houston was founded on very weak soil layers and groundwater subtraction related land subsidence of up to 10 feet (3 m) can be found for the past century, while the global sea level keeps rising.

3. What are the requirements and needs of local parties and stakeholders?
The three driving actors are Society, Economy and Environment, which all need protection against storm induced water level elevation. Due to the rapidly expanding population, local residents are moving to more flood prone areas, requiring both structural and non-structural measurements like early warning systems, education and evacuation plans. The Houston Ship Channel facilities need a flood retaining structure near the HSC entrance to protect the area from water level elevations due to local wind set-up, while during normal weather conditions the channel needs to be opened to enable navigation. To enhance the natural character of the area, interference with the environment needs to be minimized and natural (tidal) currents should be allowed. Additionally, inundation of storage tanks needs to be prevented to avoid oil spills. A closure of the Houston Ship Channel will both reduce water level elevations and restrict potential oil spills from spreading out over the ecological valuable Galveston Bay.

4. What is the best location for the Houston Ship Channel Barrier, giving the most protection for the least structure?
Due to residual surge related to storm induced local wind set-up, a structure near the Houston Ship Channel entrance is required. The best location of the Navigational Barrier is at Morgan’s Point, stretching to Atkinson Island. This gives protection to the Houston Ship Channel facilities, the Port of Houston at Barbour’s Cut and Cedar Crossing Industrial Park, as well the residential areas of La Porte and Baytown. The surrounding levees, contributing to the global protection system, can make use of existing infrastructure reducing the amount of required construction works while the interference with private properties is minimized. The good accessibility of this barrier location is a great advantage as well.

5. What additional protection measures will be required?
Both structural and non-structural measures are required to complete the total protection plan. The navigational barrier needs to be enclosed in the levee alignment, which is integrated in the existing Barbour’s Cut Boulevard (an elevated road) on its west side and needs to cross the Wildlife Management Area of Atkinson Island on the east side of the barrier. East of Atkinson Island an environmental barrier section is required that crosses the 1.5 km wide Cedar Bayou. On the east side of this environmental barrier the levee alignment extends to Highway 99, where the local ground surface is high enough to be in the safe zone for a water level elevation related to a 1/1,000 years storm event.
Non-structural measures that contribute to the protection level are early warning systems, education and evacuation. Early warning systems and education contribute to the operational quality of the barrier and additionally these measures enable evacuation, by offering residents time to leave the endangered zones. Proper landuse planning can reduce the consequences by regulating development at flood prone areas, this will reduce the size of evacuation zones too.

6. What alternative solutions are possible for the gate(s) and which gate type will be most ideal following the local design- and boundary conditions?
A number of navigational barriers is possible, distinguishing horizontally and vertically movable gate types. According to a Multi Criteria Analysis the horizontally sliding door is the most suitable gate type, based on local design- and boundary conditions like the required depth and width of the navigation channel, required foundation structures, hydraulic and aesthetical requirements. As well the evaluation of worldwide existing barriers has contributed to this result. Two alternative designs have been made of barriers containing this gate type, at the chosen location near the Houston Ship Channel entrance.
Proposed solution

The main objective of this thesis was to develop a design of a movable barrier to protect the Houston Ship Channel from flooding. By the use of the answered research questions an optimal start has been made for the final technical design of a navigational barrier at the Houston Ship Channel entrance. Two alternative designs are proposed in Chapters 5 and 6, which will be briefly evaluated here.

The Technical Design proposed in Chapter 5 is based on existing work done by Penland and Cibor (2013) and contains a horizontally sliding rectangular door constructed out of concrete. This is a robust solution that is suitable for the local design and boundary conditions. However, it is far from the desired optimum, as formulated by the thesis objectives in Chapter 1. This gate requires multiple systems for movement: First a system for filling of the Dry Dock with water, secondly a system for friction reduction (hydrofenders) due to the great dead weight, thirdly a driving mechanism for movement, a system that fills the hollow spaces with water to increase the stability when in place and finally a number of systems to pull the gate back again into the dock after a storm event. Multiple moving elements depending on systems result in more possibilities to fail and thus a larger probability of failure. This failure probability however should be investigated in more detail, as stated in the list of Recommendations.

In addition to the above described improvable aspect, the foundation requirements, load transmission and aesthetics cover the aspects on which the alternative barrier design focusses on to optimize the overall barrier design. By the use of a triangular shaped truss barrier constructed out of hollow steel tubes, aspects related to both foundation and load transmission have been improved: The load gets distributed to the foundation more favorable and the total load on the foundation is much smaller thanks to the reduced dead weight. The alternative gate design in combination with a number of additional structures results in better landscape integration and adds value in terms of recreational and spatial quality. In combination with the other described elements which are part of the total protection system, this barrier design enables a solid and reliable protection of the Houston Ship Channel and its surroundings for the next 100 years.

![Figure 7.1: Impression of the Alternative Steel Truss Barrier indicated at existing infrastructure.](image)

It can be concluded that the main objective is met by the proposed Technical Design presented in Chapter 5, and that the solution has been optimized by the Alternative Technical Design presented in Chapter 6. The majority of the specific objectives given in Chapter 1 has been met, except for the description of a Construction Method and Planning and a Global Cost Estimate which are moved to the list of recommendations. However, it can be concluded that it is beneficial to construct the Houston Ship Channel Barrier as designed in this Thesis.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS 147
7.3 Recommendations

Although the overall objective has been met and all formulated research questions have been answered, a lot of work needs to be done before one can really say that this is the optimal design of the Houston Ship Channel Barrier. As a result of new insights, new questions have been developed. Recommended studies that need to be executed in the future are briefly listed below.

- **Barrier optimization**
  Following the original design of Penland and Cibor (2013), the shape of the concrete gate has not been changed. Optimization has only been executed with respect to the outer gate dimensions and the inner configuration of the hollow spaces. For future design studies, it is recommended to investigate other shapes of a horizontally sliding concrete gate.

- **Foundation optimization**
  The back (river) side of the steel gate is modelled as a free support, bearing only vertical loads. As a result of this, the internal horizontal loads have to ‘travel’ a long distance, which may be not optimal, for future studies it is recommended to investigate alternatives. With respect to the maximum loads acting on the foundations for both gate alternatives, it is also recommended to investigate the alternative of an asymmetric pile foundation.

- **Optimization of design lifetime**
  It is recommended to link the design lifetime to the design lifetime of other critical structures with high consequences in the area.

- **Optimization of design storm**
  The execution of an economical optimization of the protection level is recommended, as provided by Danzig (1956).

- **Additional non-structural protection methods**
  As concluded, non-structural measures can greatly contribute to the protection level by either increasing the response time for gate operation or the time available for evacuation. Early Warning Systems and Education are measures that can enable this increased protection level. Additionally, an evacuation plan needs to be developed, indicating the endangered zones left after the construction of a barrier. It is also recommended to have more regulation at land use planning.

- **Response Time**
  It is recommended to determine the exact required response time. This response time is very important, since a great number of actions need to be taken within the moment of hurricane alert and the actual moment landfall. Besides the above described advantages of a proper working early warning system, the durations of the different actions need to be determined. Some examples of actions that need to be finished before actual gate closure can be initiated are the duration of vessels to return to the ports and the duration of filling the dry dock with water until the required water level has been reached.

- **Gate Operation**
  A gate operation protocol needs to be written. The exact timing of closure and opening of the gate is of major importance, wrong operation timing can reduce the benefit of a barrier.

- **Gate movement system**
  The movement systems for both elaborated barrier alternatives are designed based on assumptions. The power required to move the gates needs to be determined and the operation of hydrofenders needs to be studied in more detail. The view of a mechanical engineer is recommended.
• Water retaining wall of Alternative Design
  The only structural element for which not yet a suitable design has been presented is the water retaining wall of the steel truss barrier. As described in Chapter [6] a concrete solution may not be the best option. It is recommended to investigate the application of other materials.

• Detailed Cost Estimate
  Although it is expected that the benefits in terms of risk reduction will be much larger than the investment costs, it is recommended to make a detailed cost estimate. This can contribute to the optimization of the barrier alignment, size and choice of materials to be used. Additionally, this is required for funding the project.

• Detailed Construction Planning
  A construction planning and definition of the construction method are an important part of the barrier design. It is recommended to investigate and define this in detail, it contributes to the cost estimate and can have influence on the gate optimization. Tools that may be useful in the time planning are a Gantt Chart, Task Tables and the Critical Path determination.

• Fault tree
  To determine the reliability of the barrier, it is recommended to develop a fault tree. This framework helps determining the most critical systems or structures in order of failure probability. It can contribute to the optimization of the protection level.
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List of Figures

1. The location of interest: (a) Houston-Galveston region with proposed protection measures [SSPEED 2013]; (b) Alignment of the global protection system with three different levee sections, one environmental section and the navigational section; and (c) The final location of the HSC Barrier, indication of the navigational fairways and other relevant locations. .......................................................... 7

2. Elements of the concrete gate alternative: (a) Overview of the final gate cross-section and present loads; (b) Cross-section of the foundation slabs; and (c) Pile configuration (plan view). .......................................................... 10

3. Overview of the Alternative Design of The Horizontally Sliding Steel Truss Gate. A cross-section of the gate and the foundation with impressions of the abutments and gate in closed position. .......................................................... 11

1.1 Different concepts to protect the Houston-Galveston region, obtained from [SSPEED 2013]. The existing structures are presented in black (numbers 1 and 2), in blue are the two potential locations for the Houston Ship Channel Barrier, on which the main focus will lay on. .......................................................... 22

1.2 Hurricanes at the Texas Coast. The different tracks recorded on the left, the occurrence of storms and hurricanes per month on the right. [Roth 2009] .......................................................... 23

1.3 Hurricane Allison, path and satellite view. The colors in the left figure show the amount of rainfall in inches (10 inch = 25.4 cm) recorded during the passage of the storm. [Roth 2009] .......................................................... 24

1.4 Hurricane Rita, path and satellite view. The colors and numbers in the left figure show the associated amount of rainfall in inches. See Figure 1.3 for legend. [Roth 2009] .......................................................... 25

1.5 Hurricane Ike, path and satellite view. The colors in the left figure show the amount of rainfall in inches, recorded during the passage of the storm. See Figure 1.3 for legend [Roth 2009] .......................................................... 25

1.6 Barrier objectives, by Len Waterworth. [Waterworth 2014] .......................................................... 26

1.7 Framework of the report structure. .......................................................... 29

2.1 Locations of Texas and Houston on the left. At the right the surrounding counties and contributing drainage area to the surge gate. Obtained from Christian et al. [2014]. .......................................................... 32

2.2 Left: Houston climate graph, from USCD [2014]; Right: All year winds in Galveston Bay. Period of record: 31 December 1972 - 07 June 2014. From [ISU] .......................................................... 33

2.3 The Houston Ship Channel Area, located north of Galveston Bay. West of the framed industrial area is Downtown Houston located, which is crossed by the Buffalo Bayou and the Brays Bayou, which are both flowing out into the Ship Channel. [Sebastian et al. 2014] .......................................................... 34

2.4 Upstream hydraulic input of Houston Ship Channel: (a) Buffalo Bayou Watershed; (b) Brays Bayou Watershed; and (c) San Jacinto Watershed. [HCFCD 2010] .......................................................... 35

2.5 Harris County Watersheds, from HCFCD [2007] .......................................................... 36
2.6 Model calibration for discharge \left[ m^3/s \right] at: (a) Greens Bayou; (b) San Jacinto River; (c) Brays Bayou. [Christian et al., 2014] .................................................. 36

2.7 Location of the Houston Ship Channel and other important infrastructure. .......................................................... 38

2.8 Encroachment due to increase of industrial areas and reduction of floodplains around the Houston Ship Channel near Texas Terminal, northwest from Pasadena: (a) 1953; (b) 1995; and (c) 2014. [Christian et al., 2014] ................. 38

2.9 Expected landuse changes. [Brody, 2014] ........................................................... 39

2.10 Dimensions of the deeper and wider Ship Channel, after 2006. [HOGANSAC, 2006] .................................................. 40

2.11 Dimensions of four common vessel types, obtained from Network (2014). .................................................. 41

2.12 Height contours of the area around the Galveston Bay. .......................................................... 42

2.13 Generalized soil profile [Penland and Cibor, 2013]. .......................................................... 43

2.14 Measured data of subsidence: (a) Harris county (1906-2000); (b) measurements for Pasadena (1973-2014); and (c) measurements for Baytown (1973-2014). [HGSD, 2014] .................................................. 44

2.15 Three different locations around Galveston Bay (left), with the corresponding tidal propagations presented in the graph on the right. For the design of the Houston Ship Channel Barrier, the tidal movement at Morgan’s Point is most important. [NOAA, 2014] ........................................................ 45

2.16 Original landfall location and different shifted locations in the west, with location B as the worst case landfall location. From Sebastian et al. (2014) .................................................. 46

2.17 Computed stage hydrograph immediately upstream of proposed surge gate for (a) Hurricane Ike surge; and (b) synthetic surge variations of Hurricane Ike for 15 and 30 percent increases in wind speed [Christian et al., 2014]. The rising water level for a closed gate can be related to river runoff. .................................................. 47

2.18 Modified Ike path (a) resulting in a 25 ft. surge level flood (b). Notice the presence of the huge amount of petrochemical storage tanks (black dots in the right figure), which are located at low elevations varying from 0.6 to 6.1 m above sea level. Obtained from [Christian et al., 2014]. .................................................. 49

2.19 Sea level pressure and surface wind vectors for 4 stages of hurricane Ike (2008). From Li et al. (2012) .................................................. 51

2.20 Wave characteristics as function of storm surge per hurricane category at San Luis Pass Bridge, from [Jin et al., 2010]: a) Maximum wave height \( H_{\text{max}} \); b) Maximum wave period \( T_{\text{max}} \). .................................................. 52

2.21 Two different aspects of probabilistic design. Left: Load- vs Strength distribution, giving the reliability of the structure [Danzig, 1956]. Right: Costs as a function of the increasing safety level, giving the optimal safety level for the lowest total cost [Jonkman et al., 2014]. .................................................. 54

2.22 Return periods for tropical storms and hurricanes at the Gulf of Mexico, obtained from Keim et al. (2006) .................................................. 55

2.23 Floodplain area reduction and Water Surface Elevation Level (WSEL) reduction for three different scenarios. [Christian et al., 2014] .................................................. 55

2.24 Effect of floodgate on oil tank inundation for 8.0 m surge: (a) without gate; (b) with gate. [Bedient, 2014] .................................................. 56

2.25 Relationship between return period and damage (a) and probability of exceedance and damage (b). The area below curve (b), left from a probability of exceedance value, represents the expected annual damage for that specific protection level. .................................................. 58

3.1 The different sections the Houston Ship Channel Protection System, consisting of 3 Levee sections, an Environmental section and a Navigational section. .................................................. 63

3.2 Sketch of strategic alternatives and interventions for flood risk reduction at Galveston Bay: Protection by a Coastal Spine (left) and by the Ship Channel Barrier with additional protection works (right). [Jonkman et al., 2014] .................................................. 64
3.3 Two proposed locations for the Houston Ship Channel Barrier and the alignment of the accompanying dikes, obtained from Bedient [2014].

3.4 Alignment of the protection system with the three different levee sections and associated height profile: (a) Overview; (b) Levee section 1, Morgan’s Point; (c) Levee section 2, Atkinson Island; and (d) Levee section 3, Beach City. Obtained from [Penland] [2014] [goo] [2014].

3.5 Situation at levee sections 1 and 3, with the design of the dike to be constructed.

3.6 Situation at levee sections 2, with the design of the dike to be constructed.

3.7 The environmental barrier section with surrounding fairways.

3.8 Navigational cross sections: (a) at the barrier; (b) at the channel.

4.1 Gates types: (a) Sliding/rolling door; (b) Mitre gate; (c) Sector gate; (d) Barge gate; (e) Flap gate; (f) Lifting gate; (g) Radial gate; and (h) Inflatable rubber dam. (Dircke et al. [2012]).

4.2 Horizontally sliding gate at Oranjesluizen, navigational locks in Amsterdam, the Netherlands. (Rijkswaterstaat [2014]).

4.3 Visualization of the Valve Gate concept: 3 phases in the closing process. Obtained from Colbert [2015].

4.4 Maeslantkering, obtained from Rijkswaterstaat [2014].

4.5 Barge gate barrier, obtained from Smulders [2014].

4.6 MOSE barrier with the Lido Inlet, obtained from Squires [2008].

4.7 Hartel Barrier, obtained from www.binnenvaartinbeeld.com [2008].

4.8 Thames River Barrier, obtained from Astrid [2012].

4.9 Rubber barrier at Ramspol, the Netherlands. (Rijkswaterstaat, 2014).

4.10 Development costs and construction time of storm surge barriers. (Dircke et al. [2012]).

4.11 Total scores for the different gate types.

4.12 Scores of the horizontal moving gate types, for each subcriterion.

4.13 Scores of the vertically moving gate types, for each subcriterion.

5.1 Sketch of a roller door barrier at the final alignment, in opened (left image) and closed (right image) position.

5.2 Concept of a roller door type of barrier. Side view of the barrier at the dock, with a possible foundation. Obtained from Penland and Cibor [2013].

5.3 Hydrofeet: (a) Concept of the mechanism; (b) Cross section of one hydrofoot at the Oranjesluizen; (c) Sideview of the hydrofoot, cross section A-A’ from (b); (d) Picture of the gate bottom with one hydrofoot. (Rijkswaterstaat [2014]).

5.4 Three possible failure mechanisms during closure of the gate: (a) due to side current; (b) due to sitation of a rail; and (c) due to variations (bumps) in the height profile of the rail or sill.

5.5 Loads on the structure. Note that the support reactions ($R_{v1}$ and $R_{v2}$) are only present when the gate rests on the sill. During gate movement, these two loads are not present. Horizontal force equilibrium will be established by horizontal support reaction $R_H$ by the contact between the gate and the side of the sill at the Bay side and the impaction at the abutment head. When in closed position, the six hollow (light grey) rectangular spaces can be filled with water separately, depending on the water level surrounding the gate, improving the gate stability by increasing the vertically directed dead weight.

5.6 Determination of load transmission: Strip Method. (Fennis [2012]).

5.7 Rotation of the gate, resulting in counteracting reaction forces: (a) Top View, maximum rotation relative to x-axis $\theta_0 = 1.8^\circ$; (b) Cross-section at abutment head, maximum rotation $\theta_1 = 0.77^\circ$; and (c) Cross-section at the channel, maximum rotation $\theta_2 = 6.4^\circ$. 

LIST OF FIGURES
5.9 Eight different configurations of the hollow spaces in the door cross-section. Note that
for all configurations transversal load bearing measures need to be present, like buttress
walls or steel bracings. Although these measures are not drawn, it is assumed to be
present each 50 m of gate length. .......................... 100
5.10 Qualitative representation of the load distribution to the transverse walls and trans-
mision to the bottom, for a 100 m cut of the concrete rectangular door. .......................... 102
5.11 Graph for the resulting load envelope, for the complete 48 hours duration of a hurricane.
Loads are given as the total load difference between the two sides of the barrier, for the
full height of the gate and a gate segment width of 50 m. Note that the period of the
significant wave height \( H_{s,1/1000} = 8.0 \text{ m} \) is not in the correct time scale, it is just to
give an impression of the varying water level difference, in reality the associated peak
period \( T_p \approx 9 \text{ s} \). The tidal wave \( H_T/2 \approx 0.1 \text{ m} \) is neglected. ......................... 104
5.12 Modelling of a frame structure with 6 floors. ........................................... 104
5.13 Installation of the hydrofenders in the door. The left image presents a sideview of the
door at one of the locations that contains two hydrofenders, which will be installed
every 50 m. The right image presents the cross section at the same location. The
hydrofenders are surrounded by two 1.0 m thick buttress walls, which distribute the
horizontal loads to the bottom. .................................................. 106
5.14 Prestressing design of the sill elements. .................................................. 109
5.15 Final design of the cross section of the sill. The tendon groups are presented for the
location at the ends of the slab, where in line with the neutral axis (n.a.) the eccentricity
and thus the resulting prestressing moments are zero. The curvature for the fictitious
tendon remains the same, with drape \( f_p \approx 2.0 \text{ m} \). .......................... 109
5.16 Internal load transmission of the sill: (a) Overview of loads acting on the sill, stress
distributions and point of rotation; (b) Strut and tie model for horizontal load; (c)
Design of the solving reinforcement and stirrup configuration. .......................... 110
5.17 Configuration of the pile foundation. A pile group is located every 20 m, supporting
the 20 m long sill elements. Note that half of the pile groups contain 4 piles that have
a bearing capacity for the x-directed horizontal loads. .......................... 111
5.18 Cross-section of the sill and the bed protection layers around the barrier, obtained from
Verhagen and Schiereck [2012]. .......................... 113
5.19 Top view of the abutment head of the dry dock with the different structural elements
and installations. .................................................. 115
5.20 Loads acting on the pillars: (a) Hydraulic loads by pressure differences and wave impact;
(b) Load transmission of barrier loads; and (c) Collision force by a vessel. ........................ 116
5.21 Soil stresses acting on the dry dock. .................................................. 117
5.22 Overview of the Dry Dock: Top View and Cross-Section. .......................... 118
5.23 Steps for the construction of the diaphragm walls. Obtained from Vrijling et al. [2015]. 119
6.1 Alternative design of the Maeslant Barrier: (a) Lateral view of the sliding gate in
operation position (Rigo, 2005); and (b) Scaled dimensions for the Closed-Open-super
dike ring “Rijnmond” (Arecco et al., 2013). .................................................. 124
6.2 Concept of the steel truss structure. (a) One segment of two times 50 m; (b) Segment
with concrete slab on the bay side and arrows for the movement direction; (c) Front
side, on which the concrete slab is constructed; (d) Open back side, which has a smaller
slope angle than the front side; and (e) Side view of the composite gate structure. .......................... 125
6.3 The load acts as distributed force \( E_d \) on the concrete slab that is constructed on the
bay side of the gates. The concrete slab distributes this force to the H-girders which
subsequently distribute the load to the four outer beams. This results in the load
distribution as presented in this figure, with 4 point loads in both horizontal and vertical
direction. .................................................. 126
6.4 Cross sections of the steel truss structure: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay. ................................. 126

6.5 Occurring stresses in the steel tubes for a 1/1000 years storm surge level, wind speeds and wave impacts when one support has disappeared as a result of variations in the height profile of the foundation slab. .................................................. 127

6.6 Graph of the rotational moment around $K_1$ as a function of the negative head $H_{bay} < H_{river}$, for eleven different water levels at the bay side. .................. 128

6.7 Transmission of the external design loads [kN] to the concrete slab and the steel H-girders in x-direction: (a) Dimensions of the concrete slab; (b) Loads acting on the concrete slab and resulting support reactions acting on the H-girders; (c) Resulting bending moments in the concrete slab; and (d) Stresses in the concrete slab, the maximum negative concrete stress $\sigma_{c,max} = 38 \text{ N/mm}^2$ due to this load combination can be found at the clamped connections with the H-girders. .................... 129

6.8 The occurring stresses in the H-girders for the 1/1,000 years load combination. As presented is the maximum stress about 300 $\text{N/mm}^2$, which is smaller than the design strength of steel, so the strength of these H-girders is sufficient. ................................. 131

6.9 Maximum loads acting on the Bearing Sections of the Steel Truss Barrier, for a 1/1000 years storm: (a) Resulting stresses in the steel members; (b) Resulting support reactions, acting on the foundation. Note that members S4, S12 and S17 are parts of a tube which needs the higher steel class S460, where all other members are constructed out of S355 Steel. The actual values of the loads are presented in Table 6.1. .................. 132

6.10 Presentation of the maximum deformations of the truss due to the head difference of 8.0 m in combination with waves loading. Values are given in meter. .................. 133

6.11 Elements considering the movement of the gate: (a) A classic cattle guard at a steam train; (b) Impression of sediment guard at the gate front-end; (c) Rack and pinion movement mechanism. ................................................................. 134

6.12 Expected internal load transmission for the sill. Image (a) shows a top view of a 50 m wide part of the sill with the horizontally acting external load. The image shows that the sill is wide enough to have an equally distributed horizontal stress $\sigma_H$ on the other side of the sill as a result of the horizontal load. Cross-section A-A', Image (b), shows a detail of the footing at the bay side, in which both the horizontal and vertical load transmission is presented. .................. 135

6.13 Design of the foundation for the steel barrier alternative. The initial soil stresses, required input for the pile foundation design, are presented on the right. Note that the two light grey pillars directly under the footings are inclined in x-direction to bear movement related horizontal forces. ................................................................. 136

6.14 Roof structure of the Dry Dock: (a) Dimensions of the main girders [m]; (b) Stress distribution at the main girders [N/mm$^2$]; (c) Dimensions of the concrete hollow core slabs [mm], which are installed on top of the main girders. .................. 138

6.15 Drawing of the overview of the steel gate with the two abutments. Sideview A-A' shows a the closed gate, the levees and the building. Cross-section B-B' shows the closed dry dock, the gate, the foundation and a cross-section of the buildings. .................. 140

6.16 Impression of the Steel Truss Barrier, for front view A-A' of Figure 6.15. .................. 140

6.17 Impression of the barrier in closed position with some additional structures: The levees, museum and operation building, access roads and a parking lot. .................. 142

7.1 Impression of the Alternative Steel Truss Barrier indicated at existing infrastructure. 147

A.1 Average return periods for tropical storms, hurricanes and major hurricanes of category $\geq 3$. Obtained from [Keim et al. (2006)] .................. 172
A.2 Tropical Hazard Index. Obtained from Keim et al. (2006) .......................... 173
A.3 Table showing the US hurricanes causing 25 or more deaths. Obtained from Blake and Gibney (2011) ................................................. 174
A.4 Table showing the 30 costliest US hurricanes for the period 1900-2010. Obtained from Blake and Gibney (2011) ................................................. 175
A.5 Tropical Cyclone Records in Texas. Obtained from Roth (2009) ................. 176
A.6 Hurricane Carla, track and rainfall. Obtained from Roth (2009) ................. 177
A.7 Tropical Storm Claudette, impression of a flooded area (left) and the track of the storm (right). See Figure A.6 for the related legenda of the amounts of rainfall. Obtained from Roth (2009) ................................................. 178
B.1 Facility Storm Surge Damage Assessment (High Estimate), from Rifai and Burleson (2013) ................................................. 181
B.2 Location of Oil Tanks at the Houston Ship Channel Region, obtained from Padgett and Kameshwar (2013) ................................................. 182
B.3 Inundation of storage tanks for a 25 feet (8 m) surge for situations without (a) and with (b) a barrier, obtained from Bedient (2014) ................. 183
B.4 Relationship between the probability of exceedance $P_f$ and damage. Graph (a) presents the principle of the determination of the area below the curve (Jonkman and Schweckendiek, 2015), the total risk or expected damage is approximated by summing all parts A-D and a-d. Graph (b) represents the situation for the Houston Ship Channel. ................................................. 183
C.1 Two proposed locations for the Houston Ship Channel Barrier and the indication of some relevant locations. Obtained from Bedient (2014) .......................... 185
C.2 Height profile of the ground surface for the first proposed location, obtained from goo (2014) ................................................. 186
C.3 Height profile of the ground surface for the second proposed location, obtained from goo (2014) ................................................. 187
C.4 Comparison of Options A and B for the ’MightyIke-surge’. (Dawson et al., 2013) ................................................. 188
C.5 Two options for alignment B, with the movable storm surge barrier at Morgans Point. For the original proposal was the location of the environmental section was in the North of Atkinson Island, as presented in both images. The new chosen location is more in the south, just east of the Island, which is presented in both images as well. In the left image the environmental section is completed by a barrier, in the design of the right image the choice has been made for a line of dunes to complete the environmental section. (Penland, 2014) ................................................. 189
C.6 The three different dike sections with associated local height profile: (a) Overview; (b) Section 1, Morgans Point; (c) Section 2, Atkinson Island; and (d) Beach City. (goo 2014; Penland, 2014) ................................................. 190
C.7 Sketch of the situation for dike sections 1 and 3. ................................................. 191
C.8 Situation and introduction of parameters for a dike with a berm. ................. 191
C.9 Sketch of the design for dike sections 2. Note that all height markings are moved 1 m down: The ground surface level is in fact $M S L +1 m$ and the dike crest height at the moment of construction $M S L +13.5 m$, the relative height levels and required amounts of material however are correct. This is due to the expected settlement of 1 m due to the weak subsoils, as introduced in Section 2.4. ................................................. 194
C.10 Sketch of the environmental section. ................................................. 195
D.1 Some different types of Crude Oil Tankers (left) and a picture of the Suezmax Tanker (right) ................................................. 197
D.2 Required channel width for a 1 or 2 lane traffic channel, from F&O (2014) ................................................. 198
D.3 Required channel depth, from F&O (2014) ................................................. 199
E.1 Concept of the barrier with a concrete horizontally moveable door: (a) While stocked in a dry dock; (b) In closed position during storm surge; and (c) Free body diagram with important forces regarding the stability of the structure. [Penland and Cibor, 2013] 201

E.2 Graphs for the water level at the bay side fluctuating in time and the resulting load envelope, for the complete duration of a hurricane. Loads are given as the total load for the full height of the gate and a gate segment width of 50 m. 204

E.3 Eight different configurations of the hollow spaces in the door cross-section. 205

E.4 Modelling of a frame structure with 6 floors. 210

E.5 Modelling of internal load transmission of the concrete gate: (a) Deformations, maximum $\delta_y = 0.017$ m at the top; (b) Shear forces, maximum $V_E = 1,100$ kN/m at the bottom. 211

E.6 Modelling of internal load transmission of the concrete gate: (a) Load and supports at the bottom. (b) Internal stress at the concrete, maximum $\sigma_c = 6.0\, N/mm^2$ near the connections. 213

E.7 Main types of soil retaining walls, obtained from Vrijling et al. [2015]. 215

E.8 Different installations and structural elements to be designed for the dry dock. 216

E.9 Loads at a pillar: (a) Horizontal hydraulic load; (b) Horizontal load for a 1/1000 years storm that is distributed from the gate to the pillar; (c) Load due to ship collision. 217

E.10 Design of the dry dock. The left image gives an overview of the dry dock, the soil layers and the soil stresses acting on the retaining walls. The right image gives the resultant horizontal loads and moments on the retaining wall. The sum of the horizontal loads will be transmitted to the sill, resulting in a compression stress $\sigma_{c,sill} = \Sigma H/t_{sill} = 20,160/2 = 10,080\, kN/m^2$. 219

F.1 Cross sections of the steel truss structure: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay. 225

F.2 Cross sections of the steel truss structure with only 2 supports: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay. 226

F.3 Occuring stresses in the steel tubes for a 1/1000 years storm surge level, wind speeds and wave impacts for designs 1 and 2: (a) Steel main tubes on the outsides of the sections with a diameter of 1.50 m and a steel thickness of 0.15 m and 4 supports; and (b) Steel main tubes with a diameter of 1.40 m and a steel thickness of 0.10 m for all sections, and secondary tubes of 1.22 m and 0.025 m thickness, supported on two outer beams at the bottom side. See Tables F.1 and F.2 for the actual values of the occurring stresses and the resulting strain for each beam for the two designs. 226

F.4 Use of floaters: (a) foam blocks; (b) inflatable bags. 229

F.5 Cross-section of one prefabricated hollow core slab which is installed on top of the dry dock. 230

F.6 Main girders for the dry dock roof structure: (a) Dimensions of the N-girders \(m\); (b) Design stress distribution \(N/mm^2\); (c) Maximum deflections \(m\). 231

F.7 Sketch of the crosssection of the Dry Dock for the Steel Gate Alternative. 232

F.8 Reference structures, used as an example for the Alternative barrier design: (a) Marina Barrage Singapore [PUB, 2015]; (b) Sydney Opera House [House, 2015]. 232
G.1  Dimensions of the sill, representative values are shown by Table G.2  . . . . . . . . . . 238
G.2  Preliminary design of the sill with the design loads acting on it: (a) Load situation for
      a closed barrier during a 1/1000 years storm event; (b) Load combination 0, during
      normal conditions and an opened channel; and (c) Load combination 1, additional load
      when the gate is closed during a 1/1000 years storm event.  . . . . . . . . . . . . . . . 243
G.3  Internal load distribution of the foundation slabs: (a) Stresses in the concrete, a max-
      imum value of $\sigma_c \approx 7.5 \text{ N/mm}^2$ can be seen near the connections between the slab
      body and footings; (b) Support reactions  . . . . . . . . . . . . . . . . . . . . . . . . . 246
List of Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Characteristics of the two proposed alternatives for the Horizontally Sliding Gate Barrier.</td>
</tr>
<tr>
<td>2.2</td>
<td>Soil layer classification and strength properties. Obtained from Penland and Cibor [2013].</td>
</tr>
<tr>
<td>2.3</td>
<td>Different model predictions for storm surge levels at the Houston Ship Channel entrance, near Morgan’s Point. [Stoeten, 2013; Whalin et al., 2014; FEMA, 2014].</td>
</tr>
<tr>
<td>2.4</td>
<td>The Saffir/Simpson Hurricane Winds Scale [Irish et al., 2008; Blake and Gibney, 2011].</td>
</tr>
<tr>
<td>2.5</td>
<td>Comparison of surge level at Morgan’s Point, for different studies and return periods.</td>
</tr>
<tr>
<td>2.6</td>
<td>Determination of the Net Present Value for different protection levels.</td>
</tr>
<tr>
<td>2.7</td>
<td>Summary of design conditions.</td>
</tr>
<tr>
<td>3.1</td>
<td>Input variables used for the dike height determination. See Appendix C for a detailed description of all required parameters and computations. [Verhagen and Schiereck, 2012].</td>
</tr>
<tr>
<td>3.2</td>
<td>Run-up and Run-down for a 1/100 years storm surge for the three levee sections. See Appendix C for a detailed description of all required parameters and computations. [Pullen et al., 2007].</td>
</tr>
<tr>
<td>3.3</td>
<td>Overtopping amounts for a 1/1,000 years storm for all three levee sections. See Appendix C for a detailed description of all required parameters and computations. (Pullen et al., 2007).</td>
</tr>
<tr>
<td>3.4</td>
<td>Required channel dimensions, for a 2-lane Suezmax traffic. The resulting design can be seen in Figure 3.8 [PIANC, 1997; F&amp;O, 2014].</td>
</tr>
<tr>
<td>4.1</td>
<td>Horizontally moving gates, not floating versus floating.</td>
</tr>
<tr>
<td>4.2</td>
<td>Favorable and unfavorable aspects of horizontally sliding gates. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.3</td>
<td>Favorable and unfavorable aspects of horizontally floating gates and sector gates. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.4</td>
<td>Favorable and unfavorable aspects of flap gates. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.5</td>
<td>Favorable and unfavorable aspects of lifting gates. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.6</td>
<td>Favorable and unfavorable aspects of radial gates. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.7</td>
<td>Favorable and unfavorable aspects of an inflatable rubber dam. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.8</td>
<td>Summary of the characteristics of the described gate types. [Dircke et al., 2012].</td>
</tr>
<tr>
<td>4.9</td>
<td>Weight factor for each sub criterion. Rates are from 1 (small importance) to 5 (priority, major importance).</td>
</tr>
<tr>
<td>5.1</td>
<td>Stability check for the concrete door (configuration 6). The moment around $S_1$ (Figure 5.6) is required to be positive (anti-clockwise) for stability. The points of action are the heights on which the in Figure 5.6 introduced resultant forces act, measured from point $S_1$ at MSL -22 m.</td>
</tr>
<tr>
<td>5.2</td>
<td>Resulting dimensions for the different configurations. Note that configuration 6 gives the economical optimum in terms of required volume of concrete.</td>
</tr>
</tbody>
</table>
5.3 Bearing capacities associated with the final design of the pile foundation \( \phi_t = 2.5 \text{ m} \); 
\( t_t = 0.025 \text{ m}; \alpha_t = 71^\circ \), which will vary between these values for varying bearing 
depths. As presented, the bearing capacity will always be larger than the characteristic 
design loads. ................................................................. 112

6.1 Internal loads for all members presented in Figure 6.9 related to a 1/1000 years load 
event. ................................................................. 132

6.2 Characteristics of the two proposed alternatives for the Horizontally Sliding Gate Barrier. 143

B.1 Value of a barrier, determined for different protection levels. ................................. 184

C.1 Characteristics of the local ground surface profiles at the three levee sections. ........ 190
C.2 Run-up and Run-down for a 1/100 years storm surge for the three levee sections. .... 193
C.3 Overtopping amounts for a 1/1,000 years storm for all three levee sections. ............ 193
C.4 Overtopping \( q \) and cross section area \( A \) for levee section 2 as function of the dike height, 
the slope angle and the berm width. ................................................................. 194
C.5 Cost estimate for the total system of levees. [Brittin, 2012] ...................................... 195

D.1 Required channel dimensions, for a 2-lane traffic intensity of each type of vessel. .... 197
D.2 Required channel width calculations [PIANC, 1997; F&O, 2014]. ............................ 198
D.3 Required channel depth calculations [PIANC, 1997; F&O, 2014]. ........................... 199

E.1 Design values for the load components \( F_{\Delta H} \) and \( F_{\text{waves}} \), representing the surge and 
the wind waves respectively. Characteristics for hurricane Ike are used as input values 
for the size and propagation speed of the hurricane, where wave heights are slightly 
upscaled to a 1/1000 years storm event. ................................................................. 203
E.2 Resulting dimensions for the different configurations. Note that configuration 6 gives 
the economical optimum in terms of required volume of concrete. ............................... 205
E.3 Stability check for the concrete door with configuration 6 by computing the moment 
around \( S_1 \), which is required to be positive (anti-clockwise) in order to have stability. 
The points of action, heights on which the resultant forces act, are measured from \( MSL 
-22 \text{ m} \). .............................................................................................................. 206
E.4 Strength checks. Determination of the required wall thicknesses in order to provide 
enough strength, for cross-section configurations 1 to 4. ............................................. 208
E.5 Strength checks. Determination of the required wall thicknesses in order to provide 
enough strength, for cross-section configurations 5 to 8. ............................................. 209
E.6 Dimensions of the diaphragm walls, in combination with the occurring loads and result-
ing deformation ................................................................. 221

F.1 Occuring stresses in the steel tubes and resulting strains. The table shows that all 
stresses are smaller than the steel yield strength \( f_y = 355 \text{ N/mm}^2 \), so yielding will not 
occur for loads associated with a 1/1000 years storm. ..................................................... 227
F.2 Occuring stresses in the steel tubes and resulting strains for design 2. The table shows 
that all stresses are smaller than the steel yield strength \( f_y = 355 \text{ N/mm}^2 \), so yielding 
will not occur for loads associated with a 1/1000 years storm. ........................................ 227

G.1 Design values for the relevant parameters for computing the pile tip bearing capacity. 236
G.2 Sill dimensions, the resulting cross sectional area and the location of the Neutral Axis. 
See Figure [G.1] for an explanation of the different parts. .............................................. 239
G.3 Design loads on the pile foundation for the concrete barrier alternative ...................... 239
G.4 Characteristics of the installed piles. ................................................................. 240
G.5 Bearing capacity computations, for pile diameter \( \phi_t = 2.5 \text{ m} \). ................. 240
G.6 Characteristic capacities per pile ......................................................... 241
G.7 Strength check of the concrete pile foundation ........................................ 241
G.8 Design values for the relevant parameters for computing the pile tip bearing capacity ......................................................... 242
G.9 Dimensions and spacing of the used piles, for the channel sections and the dry docks at bearing depths of 54 m and 64 m ......................................................... 244
G.10 Computation of the associated bearing capacities per pile ......................... 245
G.11 Tip bearing computations ...................................................................... 245
G.12 Skin friction computations ..................................................................... 245
G.13 Computations for the pile dead weight. Note that the density of the material depends on weather or not the structure is submerged ......................................................... 245
Appendices
Appendix A

Historic storm events

The past century multiple hurricane events are registered around the coast of the United States. Keim et al. (2006) have analyzed 105 years (1901-2005) of tropical cyclone strikes for the US Sout-East Coast, from the Mexican border to the border with Canada, with the primary objective of identifying and examining of hurricane patterns. For different hurricane categories Keim et al. (2006) developed maps with the related return periods for 45 coastal locations. Additionally a Tropical Hazard Index (THI) has been developed giving for each location a so-called hazard number. Subsequently Blake and Gibney (2011) have ranked all tropical cyclones from 1851-2010 in terms of the most deaths, the costs and the intensity, for all the United States. Needham and Keim (2011) have developed a storm surge database for the US Gulf Coast, in which the location and height of peak storm surge are identified for 195 surge events since 1880. A valuable document for coastal stakeholders, including planners, emergency managers coastal scientists and research centers, enabling an improvement in flood risk determination and planning of coastal protection.

Where Keim et al. (2006) Blake and Gibney (2011) and Needham and Keim (2011) focussed on a very wide stretch of coastline, Roth (2009) focussed specifically on hurricanes hitting the coast of Texas. In addition to overviews of tropical cyclone records in Texas, Roth (2009) offers a detailed description of each hurricane, giving the hurricane track, dates, numbers of associated deaths and amount of damage.

In this appendix an overview is given of the most deadliest and costliest hurricanes and tropical storms in the United States in a 150 years period between 1851 and 2010, in both numbers and maps of the hurricane tracks. Subsequently the most important flood events recorded at the Texas Coast will be given for both surge related flood events and rainfall related flood events. All presented data is based on the above described documents with existing works Keim et al. 2006; Roth 2009; Blake and Gibney 2011 Needham and Keim 2011).

A.1 Overview of tropical storms recorded at the coast of the United States

A.1.1 Average return periods of tropical storms per location

As introduced, Keim et al. (2006) analyzed hurricane return periods for a number of locations along the coast of the United States. Based on these return periods a Tropical Hazard rate is awarded to each location as a sum of all storms from 1901 to 2005, where tropical storms are awarded two points, hurricanes of category 1 and 2 are awarded four points and hurricanes of higher categories are awarded eight points. The average return periods for all locations are presented in Figure A.1 where Figure A.2 presents the resulting Tropical Hazard Index (THI). Note that the Houston-Galveston region (location 4) is rated 12th in this index.
Figure A.1: Average return periods for tropical storms, hurricanes and major hurricanes of category $\geq 3$. Obtained from Keim et al. (2006).
A.1.2 Ranking of tropical storms

In addition to a ranking of hurricane prone coasta areas, the different hurricanes are ranked by Blake and Gibney (2011) for the time period 1851-2010. Hurricanes are ranked based on the associated number of deaths and the related amount of damages, as presented in the tables of Figures A.3 and A.4 respectively.
A.2 Major flood events at Houston, Texas

Roth (2009) describes all hurricanes recorded at the Coast of Texas from the Sixteenth Century onwards, giving the dates, the associated number of deaths, related damages and the hurricane paths. In this section some important flood events since 1900 for the Houston-Galveston Region will be described. A distinction will be made between surge induced flood events and flood events due to extensive rainfall. The tables presented in Figure A.3 present some maximum values for a number of hurricane characteristics, recorded in Texas.
A.2.1 Surge induced flood events

For the Houston-Galveston Region the most important surge induced flood events are the Galveston floods of 1900 and 1915, the 1919 Hurricane, Hurricane Carla (1961) and Hurricane Ike (2008). The Great Galveston Flood of 1900 and Hurricane Ike (2008) are both described in Section 1.2. In addition to the in Section 1.2 described events the events of 1915, 1919 and 1961 will be briefly described in this section.

The Galveston flood of 1915

With the Great Galveston Hurricane of 1900 fresh in the minds of the residents, the Houston-Galveston region got hit by a monstrous hurricane in August 16-19th, 1915. In Galveston many people were evacuated, resulting in a relatively small death toll of 12 deaths in Galveston, where the total number of deaths for this storm is 275. A storm surge up to 5 m was noted. In Houston winds of 100 km/h were reported. Damages were estimated $50 Million.
Corpus Christi’s Devastating Hurricane of 1919

During September 14-16, 1919, a severe hurricane crossed Texas. It made landfall near Corpus Christi, about 250 km southwest from Houston, where winds of 180 km/h and a storm surge of 5 m were experienced. The maximum 24 hours amount of rainfall of 30 cm was seen at George West on September the 15th. This storm was special due to its extraordinary size, anywhere around the Gulf Coast winds were reported of over 100 km/h. In the Houston-Galveston region the 24 hours rainfall was about 16 cm and the surge elevation was almost 3 m. Over 310 lives were lost and the total damages were estimated at $20 Million. This storm led to the construction of multiple flood protection structures around the Texas Coast near Corpus Christi (Roth, 2009).
Hurricane Carla (1961)

According to Roth (2009), Carla was among the largest hurricane of historical record, recorded September 11-13, 1961. It made landfall at Port Lavaca, about 150 km southwest from Houston. This storm produced many tornadoes, rainfall, a maximum surge of 7 m and wind gusts up to 280 km/h. Total damages were estimated at $408 Million and a death toll of 38, for Texas.

Figure A.6: Hurrican Carla, track and rainfall. Obtained from Roth (2009).
A.2.2 Rainfall induced flood events

Roth (2009) mentions that any system, no matter what strength, can create major flood problems for Texas. This is due to the fact that the weaker the storm system is, the more efficient it is at producing heavy rains and catastrophic flooding. This can easily be explained by the fact that a slower moving storm has more time to pour rain. Claudette (1979), Allison (both 2001 and 1989), and Charley (1998) stand out as recent examples of this fact. For the Houston-Galveston Region the events of 1979 and 2001 have caused the biggest flood problems. Tropical Storm Allison (2001) has been described in Section 1.2. Tropical Storm Claudette (1979) will be briefly described in the following section.

Tropical Storm Claudette (1979)

This relatively weak tropical storm (category 1) made landfall near Galveston on July 25 after which it made a loop above southeast Texas, where it kept pouring for two days. According to Roth (2009), Tropical Storm Claudette produced $750 Million damages, putting it on the National Hurricane Center’s list of the most damaging tropical storm in US history. One person drowned during this storm event.

Figure A.7: Tropical Storm Claudette, impression of a flooded area (left) and the track of the storm (right). See Figure A.6 for the related legenda of the amounts of rainfall. Obtained from Roth (2009).
Appendix B

Consequences of Flooding

B.1 Introduction

As presented in the main text (Section 2.6) the risk of flooding is the result of the multiplication of probability and consequences. The probability of floods due to hurricanes has been described in the main text and is supported by Appendix A. The associated consequences are described in this Appendix, where a distinction is made between consequences related to society, environment and industry. This Appendix concludes with the combination of flooding probabilities and economic damages, by which the value of different measures is determined.

B.2 Social Damages

Read (2012) states the demographic growth of the Houston-Galveston region as a very important point of concern. Besides the growth of population he gives the growth of evacuation zones, which grew from 0.5 million residents in 1992 to 0.9 million in 2012. With this growth the estimated clearance time, which is the time required to evacuate the population in these zones, increased from 24 hours to 48 hours. The population is expected to increase to 8.8 million in 2035, and the number of residents living in the evacuation zones to 1.6 million. As presented in Figure 2.5 in Section 2.2.4, the total amount of residents living in the different watersheds is about 3.4 million (HCFCD, 2010, 2007), determined in 2007. In the scope of the Houston Ship Channel Barrier, the expected area to be flooded for a 20 feet surge level will be mainly in the San Jacintho Watershed. Therefore the number of residents in this watershed will be used for the determination of possible impact on the society. This amount is 150 thousand residents, as presented in Section 2.2.4. To conclude, Read (2012) sees the people challenge greater than the meteorological challenge, like the global sea level rise.

Also according to Brody (2014) the land use change is one of the major triggers of flood loss, see Section 2.3.1 and Figure 2.9. And still residential areas will probably increase in the future to accommodate the growing population in this area, which asks for more awareness for flood risks by developers and homeowners. Brody (2014):

"The population growth is a bigger challenge in reducing the flood risk problem than the relative sea level change."

As a result of several literature studies, the expected population growth and the inflation for the next 100 years, a human life will be valued worth $10 million in the estimation of the consequences of a flood. For a 1/1,000 years storm event a number of 100 deaths is expected, resulting in a total of approximately 1.0 Billion USD in social damages.
B.3 Environmental Damages

Because of its ecological value, Galveston Bay has been pointed as a protected area were development and management should follow the Galveston Bay Estuary Program, which is a program approved by the governor of Texas and the Administrator of the U.S. Environmental Protection Agency. The program contains guiding documents for research and other activities.

In order to describe the environmental impact of flooding of the Houston Ship Channel, the vulnerability of the area can be used. Vulnerability is described as the degree to which a system is susceptible to cope with adverse effects. Two types of vulnerability can be distinguished, as presented below, which will be described separately in this section. The estimated amount of environmental damage associated with a 1/1,000 years storm event is approximately 4.0 Billion USD.

- Spatial: inundation with flood waters.
- Environmental: spills and releases.

B.3.1 Environmental Damage Assessment

It is hard to express environmental damage in money. However, Rifai and Burleson (2013) have made a storm surge damage assessment for some parts of the Houston Ship Channel Barrier. In a qualitative way, Figure B.1 shows that for relatively small storm surge levels the damage is evident particularly in environmental damage and also for bigger flood levels the environmental damage accounts for the major part of the damage.

B.3.2 Oil Spills

A brief summary of tank inundation, the capacity of storage tanks and some historic oil spills is presented in the next phrase. Subsequently, the different characteristics of oil spills are described in more detail.

Summary

Failure of an oil storage tank can be found for typical surge levels of about 5 m, where all present tanks around the Houston Ship Channel will fail for surge levels of 6 to 8 m. The average capacity of one oil storage tank is 250 thousand barrels, which is about 40 thousand $m^3$. About 1100 tanks can be found at flood prone areas around the HSC, but not all of these tanks are fully filled at the same time. As a result of Hurricane Katrina for example, only 1 tank got inundated for which about 10 percent oil got spilled resulting in an oil spill of 4.5 thousand $m^3$. To take these things into account, only 1 percent loss will be used for the estimate of volume of oil spill for an eight meter surge event. The potential oil spill is 11 tanks times 25 thousand barrels, resulting in 0.3 million barrels or 50 thousand $m^3$ (SSPEED, 2013). To compare, this is of the same order of magnitude as the Exxon Valdez oil spill of 1989 (Gilvray, 2010).

Oil tanks at Houston Ship Channel

Extreme water events the past decades have showed the vulnerability of oil storage tanks to hurricane events and in particular to surge. According to Padgett and Kameshwar (2013), out of 4,200 tanks in the region 1,485 tanks lie within the areas that will be flooded for a storm with a 500 years return period. This potential damage in combination with the occurency of hurricanes makes the Houston Ship Channel region one of the most flood prone areas in the world. The locations of oil tanks at the Houston Ship Channel Region are shown by Figure B.2.
Tank inundation

The inundation of storage tanks for a surge event with 8 m surge is presented in Figure B.3-(a), the reduced inundated area as a result of the construction of a storm surge barrier can be seen in Figure B.3-(b). Tank inundation may result in tank failure, which can be categorized in the following failure modes: failure due to uplift and displacement, failure due to buckling of the side walls of the walls and failure by rupture. The first mode can be related to stability and the other two to strength respectively. Instability resulting in uplift and overturning can be solved by the installation of anchors. In order to reduce the probability of shell buckling, stiffening rings can be installed around the tanks. Impact of tank failure is listed below (Padgett and Kameshwar 2013).

- Spillage of hazardous material
- Environmental impact
- Clean up costs and losses
- Repair and replacement costs

Comparison with recent oil spills

As a result of hurricane Katrina only 1 tank, a 250,000 barrel above ground storage tank, was dislodged, lifted and damaged. At that moment the tank contained (only) 65,000 barrels of mixed crude oil and
released approximately 25,110 barrels over a square mile area. Although the tank met all industry standards and regulations approximately 1,800 homes were affected and a compensation of 330 million USD was agreed for. Compared to the potential spill for the Houston Ship Channel Complex, the spill after Katrina was just tiny, as presented below. In case of a direct hit of a category 4 hurricane, the economic and environmental damages will be irretrievable (Merrell, 2012).

- Katrina tank spill (1 tank).
  - 250,000 barrel capacity
  - 65,000 barrels in place at time of accident.
  - 25,000 barrels spilled

- Potential Channel spill
  - 1100 tanks at risk
  - 1 percent damaged = 11 tanks of 25,000 barrels = 53 thousand m$^3$.

To compare, this potential oil spill is of the same order of magnitude as the Exxon Valdez oil spill, where 10-30 million gallons were lost, in 1989. According to Gilvray (2010), the effects of the spill were not gone after more then 20 years even though cleaning started quite fast after the disaster happened. In 2010 an estimated 23,000 US Gallons (87 m$^3$) of the crude oil was still present in Alaska’s sand and soil. The estimated breakdown rate of such oil is less than 4 percent per year (Gilvray, 2010).

### B.4 Industrial Damages

The major part of the total amount of damages is due to economical damages in Industry and Business. This expresses the national interest of the Houston Ship Channel. As a result of the summation of annually generated economic impact, tax revenues, gross product and the capital available, the amount of economic damages as a result of a 1/1,000 year storm event is estimated at 100 Billion USD.
B.5 Combining Flooding Probabilities and Damages

In the main text the basics have been described to determine the benefits of a measure to reduce the flood risk, for an infinite time horizon ($t \rightarrow \infty$) in terms of Net Present Value. It has been introduced that for an increasing amount of damages (USD) for increasing return periods ($T$), the benefits can be derived from the determination of the area under the Probability-Damage-curve, presented in Figure B.4 (a).

$$D_{\text{total}} = A + a + B + b + C + c$$ \hspace{1cm} (B.1)
In which:

\[ A = (P_{f,1000} - P_{f,10000})D_{8,1000}; \quad a = (P_{f,1000} - P_{f,10000})(D_{8,1000} - D_{8,10000})/2; \quad \$/y\,(B.2) \]
\[ B = (P_{f,100} - P_{f,1000})D_{8,100}; \quad b = (P_{f,100} - P_{f,1000})(D_{8,1000} - D_{8,100}); \quad \$/y\,(B.3) \]
\[ C = (P_{f,26} - P_{f,100})D_{8,26}; \quad c = (P_{f,26} - P_{f,100})(D_{8,100} - D_{8,26}); \quad \$/(y\,(B.4) \]

Table B.1: Value of a barrier, determined for different protection levels.

<table>
<thead>
<tr>
<th>Protection level (Return period T)</th>
<th>Surge [m]</th>
<th>( P_f ) [y(^{-1})]</th>
<th>Damage ([10^9,\text{year}^{-1}])</th>
<th>Cost/year ([10^9,\text{year}^{-1}])</th>
<th>Reduction/year ([10^9,\text{year}^{-1}])</th>
<th>NPV ([10^9,\text{year}^{-1}])</th>
</tr>
</thead>
<tbody>
<tr>
<td>26 year:</td>
<td>4.0</td>
<td>0.038</td>
<td>$30</td>
<td>$2.68</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>100 year:</td>
<td>6.0</td>
<td>0.010</td>
<td>$90</td>
<td>$0.99</td>
<td>$1.70</td>
<td>$33.9</td>
</tr>
<tr>
<td>1,000 year:</td>
<td>8.0</td>
<td>0.001</td>
<td>$105</td>
<td>$0.11</td>
<td>$2.57</td>
<td>$51.5</td>
</tr>
<tr>
<td>10,000 year:</td>
<td>9.0</td>
<td>0.0001</td>
<td>$110</td>
<td>$0.01</td>
<td>$2.67</td>
<td>$53.4</td>
</tr>
</tbody>
</table>

APPENDIX B. CONSEQUENCES OF FLOODING
Appendix C

Protection system alignments

C.1 Barrier locations

Two locations for the Houston Ship Channel Barrier have been proposed by the SSPEED Center, indicated with locations A and B in Figure. The characteristics of these two locations are given in this Section. A comparison of the provided protection levels between alternative A and B can be seen in Figure C.4. After the selection of the final protection system alignment, the different levee sections are designed. These steps required to determine the economical optimum dike design are presented in Section C.2 in combination with a rough cost estimate.

Figure C.1: Two proposed locations for the Houston Ship Channel Barrier and the indication of some relevant locations. Obtained from Bedient (2014).
C.1.1 Barrier location A

1. Levee about five miles long. Almost entirely through undeveloped areas.
2. Limited interference with private property.
3. Very few property owners will be impacted by construction or ROW purchase.
4. Limited environmental impact.
5. If necessary the levee system can be raised to thirty feet in future.
6. Erosion and subsidence are not significant problems.
7. Very few roadway crossings required. SH 146 protected where it crosses Goose Creek.
8. No negative aesthetic impact.
9. Gate could serve as tourist attraction and an opportunity to improve the area for public use.

Figure C.2: Height profile of the ground surface for the first proposed location, obtained from [goo](2014).
C.1.2 Barrier location B

1. Levee about 7.5 miles long.

2. Very limited interference with private property.

3. Few property owners impacted.

4. Limited environmental impact.

5. Erosion and subsidence are not a problem.

6. Limited land construction required due to topography. Most construction is over shallow water.

7. Additional area to be protected includes about 2,500 acres of land, including the Barbours Cut terminal, and 2,500 acres of open water that will act as a sump for rainwater that may build up while the gate is closed.

8. More room for fresh water exchange and to reduce water velocity at gate.

9. Construction would have only minor impact on roadway network.

10. Provides access to Ship Channel islands.

11. Gate could serve as tourist attraction and an opportunity to improve Galveston Bay for public use.

Figure C.3: Height profile of the ground surface for the second proposed location, obtained from google (2014).
Figure C.4: Comparison of Options A and B for the 'MightyIke-surge'. (Dawson et al., 2013)
Variations for location B

When the movable storm surge barrier is located next to Morgans Point, described by option B, the protection system has to cross Atkinson Island. Atkinson Island is a wildlife preserve area consisting of 0.16 \( km^2 \) of woodlot, 0.36 \( km^2 \) segment is brackish marsh and then 0.08 \( km^2 \) is a spoils site left from dredging the Houston Ship Channel. At the moment visitors can enter the island by boats only, the island is opened year round but during spring and summer visitors are requested not to disturb the shorebirds. On the eastern side of Atkinson Island is the 1.6 to 2.0 \( km \) wide Cedar Bayou Channel, where the protection system requires an environmental section which allows tidal movement and the passage of fish, see Figure C.5.

![Figure C.5: Two options for alignment B, with the movable storm surge barrier at Morgans Point.](image)

For the original proposal was the location of the environmental section was in the North of Atkinson Island, as presented in both images. The new chosen location is more in the south, just east of the Island, which is presented in both images as well. In the left image the environmental section is completed by a barrier, in the design of the right image the choice has been made for a line of dunes to complete the environmental section. [Penland, 2014]

C.2 Levees

C.2.1 Levee alignment

The system of levees can be divided into 3 different sections: Morgans Point; Atkinson Island; and Beach City, from west to east respectively. The dike sections at Morgans Point and City Beach are pretty similar to each other: a narrow beach backed by a floodplain with a surface elevation of 5 to 6 \( m \) and a width of 750 and 1,000 \( m \) to the location of the levee at Morgans Point and Beach City respectively. The Levee at Atkinson Island, however, will be constructed at a ground elevation of about zero \( m \) where the other two are constructed at a ground surface level of 6 \( m \), as presented in Figure C.6. Table C.1 gives a summary of some of the characteristics of the three height profiles.
Table C.1: Characteristics of the local ground surface profiles at the three levee sections.

<table>
<thead>
<tr>
<th>Sections</th>
<th>Dike section 1</th>
<th>Dike section 2</th>
<th>Dike section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local height at dike location $z_{fs}$ [m]</td>
<td>6.0</td>
<td>0.5</td>
<td>6.0</td>
</tr>
<tr>
<td>Height at shore $z_s$ [m]</td>
<td>5.0</td>
<td>0.2</td>
<td>5.0</td>
</tr>
<tr>
<td>Floothplain width $x_{fs}$ [m]</td>
<td>750</td>
<td>2,300</td>
<td>1,000</td>
</tr>
<tr>
<td>Shore width $x_s$ [m]</td>
<td>50</td>
<td>100</td>
<td>600</td>
</tr>
<tr>
<td>Slope of floodplain $i_{fs}$ [-]</td>
<td>0.0013</td>
<td>0.0001</td>
<td>0.0010</td>
</tr>
<tr>
<td>Slope of shore $i_s$ [-]</td>
<td>0.1000</td>
<td>0.0020</td>
<td>0.0083</td>
</tr>
</tbody>
</table>

C.2.2 Slope protection

In order to maintain the environmental character of the area the choice will be made for grass covered dikes which minimizes the interference with the landscape. This will have some influence in the crest height determination in the next section, where in the wave run-up and overtopping computation a characteristic roughness correction factor of $\gamma_r$ can be used for grass slopes. (Verhagen and Schiereck, 2012)

C.2.3 Crest height determination

In order to determine the correct crest height for the levees, first an estimation will be made based on the ultimate design surge level which is 8 m, associated to the 1/1,000 years storm. The estimated crest height is than 8 m plus a freeboard of 0.5 m which results in a crest height of 8.5 m. This preliminary levee will have 1:3 slopes without a berm.
Wave run-up and overtopping amounts will be determined for a 1/100 year storm, associated with a storm surge level of 6.0 m and a significant wave height of 5.0 m, using formulas provided by Verhagen and Schiereck (2012). For sections 1 and 3 the floodplain surface height more or less equals the storm surge level and therefore can be seen as a berm at the levee slope, see Figure C.7. The dike crest for these two section is elevated only 2.5 m above the local ground surface, but due to the relatively very wide berm (floodplain $x_{fp}$) of 750 and 1,000 m respectively, it is very unlikely that the waves will cause severe run-up and overtopping since they will break at the shoreline and loose most of their wave energy. Additionally, the distance between the point of breaking and the toe of the levees is expected to be large enough for wave dissipation. As presented in Table C.2, the hand calculation confirms that the wave run-up associated to a 1/100 years storm at the levee slopes is negligible. To conclude, the levees at sections 1 and 3 will be constructed with a crest height of 8.5 m in order to provide a full flood protection for 1/100 year storms and a sufficient flood protection against a 1/1,000 years storm where overtopping may be expected to be significant but where the hinterland is protected for overflow.

Levee section 2, however, has to deal with different conditions. The floodplain of Atkinson Island is just 0.5 m above MSL and therefore cannot be seen as a dike berm. However it can be seen as a shallow foreshore, since waves will break at this floodplain as well according to the criterion of wave breaking, expressed in equation (C.1) where $H_s$ is the significant wave height and $h$ is the local water depth, wave breaking will take place if this criterion is met.

$$\frac{H_s}{h} > 0.8 \quad (C.1)$$

Since the local water depth during a storm surge is equal to the surge level minus the local ground surface level, which is about 0.5 m at Atkinson Island, the significant wave height of 5 m combined with the resulting water depth (5.5 m) will meet this criterion and thus wave breaking will take place. Only waves smaller than $0.8 \cdot 5.5 = 4.4$ m will not break at the shore of the island. However, as presented in Figure C.6(c) a surface elevation of 1 m can be found at a distance of about 50 m from
the shoreline which will work as a bank, resulting in wave breaking for waves with a wave height of 4.0 m and larger.

Most of the energy of the broken waves will be dissipated by the 2.3 km wide floodplain. Although waves smaller than 4 m might break relatively energetic at the slopes of the levee causing wave run-up. The for this situation characteristic Iribarren number is 1.73, as determined in equation (C.2) which can be associated with plunging waves with relatively big wave impact.

\[ \xi = \frac{\tan(\alpha)}{\sqrt{H/L_0}} = \frac{\tan(1/3)}{\sqrt{4/100}} = 1.73 \]  

Where:
\( \alpha \)  Slope angle  
\( H \)  Design wave height  
\( L_0 \)  Deep water wave length

For breaking waves (\( \xi < 2.5 - 3 \)) on smooth slopes the wave run-up (\( R_u \)) can be computed by the Hunt’s formula:

\[ \frac{R_u}{H} = \xi \]  

(C.3)

In addition to the levee slope angle and the incoming waves, the wave run-up is also dependent on the slope protection, the angle of attack of the incoming waves and the presence of a berm or a shallow foreshore, which can reduce the wave run-up. These aspects can be involved by the use of correction factors, by which the wave run-up has to be multiplied. For simplicity reasons only the correction factors associated with the roughness and the presence of a berm are taken into account, which are \( \gamma_r \) and \( \gamma_B \) respectively. As mentioned in Section (C.2.2) the used slope protection is grass, which has a characteristic roughness correction factor of \( \gamma_r = 0.95 \). The berm correction factor can be computed as follows:

\[ \gamma_B = 1 - \frac{B_B}{L_B}[0.5 + 0.5\cos(\frac{h_B}{x})] \]  

(C.4)

Where all parameters are already explained in Figure (C.8) besides \( x \), which is equal to two times the significant wave height \( 2H_s \). According to [Verhagen and Schiereck (2012)], the correction factor is limited by: \( \gamma_B > 0.6 \).

As presented in Table (C.2) the resulting run-up for levee sections 1 and 3 is much smaller than the freeboard of 2.5 m (\( h_c \)), so no overtopping will take place for a 1/100 year storm surge. For Levee section 2 however, a big amount of overtopping can be expected but this is not a big issue since the area behind this levee is not developed. Still the amount of overtopping should not be more than a certain allowable overtopping limit, which is determined by the quality of the slopes of the dike. The overtopping will be computed for a 1/1,000 years storm surge, since this will be the decisive value.
Table C.2: Run-up and Run-down for a 1/100 years storm surge for the three levee sections.

<table>
<thead>
<tr>
<th>Sections</th>
<th>Run-up (equation [C.3])</th>
<th>Berm width (floodplain $x_{fp}$)</th>
<th>Average distance SSL-Berm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\xi$ [-]</td>
<td>$B_b$ [m]</td>
<td>$h_b$ [m]</td>
</tr>
<tr>
<td></td>
<td>0.019</td>
<td>750</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>1.731</td>
<td>2300</td>
<td>5.8</td>
</tr>
<tr>
<td></td>
<td>0.014</td>
<td>1000</td>
<td>0.5</td>
</tr>
</tbody>
</table>

|                   | Run-up: $R_u$ [m]       | Run-down: $R_d$ [m]              |
|                   | 0.09                    | 0.04                             |
|                   | 6.93                    | 1.85                             |
|                   | 0.07                    | 0.04                             |

Where according to Battjes [1974] the run-down can be computed by:

$$R_d = R_u (1 - 0.4\xi) = H (1 - 0.4\xi) \xi$$  \hspace{1cm} (C.5)

Overtopping criterion

The amount of overtopping will be computed for a 1/1,000 years storm. It is the task to design the levees in a way that the amount of overtopping is less than the limit given by the quality of the slope, in order not to cause severe damage to the inner slope. High quality slopes are assumed, which gives the allowable overtopping limit of $q < 10 \text{ l/s}$ (Verhagen and Schiereck 2012). The average overtopping $q$ in $m^3/s$ per meter dike can be computed using formula [C.6] given in the Eurotop manual (Pullen et al. 2007).

$$Q = \frac{q \sqrt{gH_s^3}}{\tan(\alpha)}$$  \hspace{1cm} (C.6)

In which dimensionless overtopping $Q = a \cdot \exp(-b \frac{H}{\gamma})$ as a function of the dimensionless freeboard $R = \frac{h_c}{H_{c-x}}$ and three constants $a = 0.067$, $b = 4.3$ and $\sigma_b = 0.5$. For a 1/1,000 years storm surge the results are given in Table [C.3].

Table C.3: Overtopping amounts for a 1/1,000 years storm for all three levee sections.

<table>
<thead>
<tr>
<th>Sections</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1,000 years storm characteristics</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Storm surge level (+MSL) $SSL$ [m]</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Freeboard $h_c$ [m]</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Significant wave height $H_s$ [m]</td>
<td>6.0</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Design wave height (unbroken) $H_d$ [m]</td>
<td>2.0</td>
<td>5.6</td>
<td>2.0</td>
</tr>
<tr>
<td>Average water depth $h_b$ [m]</td>
<td>2.5</td>
<td>7.0</td>
<td>2.5</td>
</tr>
<tr>
<td>Correction factor $\gamma$ [-]</td>
<td>0.85</td>
<td>0.94</td>
<td>0.85</td>
</tr>
<tr>
<td>Dimensionless overtopping $Q$ [-]</td>
<td>5.76E-31</td>
<td>5.29E-02</td>
<td>1.17E-40</td>
</tr>
<tr>
<td>Dimensionless freeboard $R$ [-]</td>
<td>13.27</td>
<td>0.05</td>
<td>17.69</td>
</tr>
<tr>
<td>Average overtopping per meter dike $q$ [$m^3/s$]</td>
<td>1.90E-29</td>
<td>4.88E+00</td>
<td>3.86E-39</td>
</tr>
</tbody>
</table>

As well for a 1/1,000 years storm the amount of overtopping is negligible for levee sections 1 and 3, as presented in Table [C.3] from which can be concluded that the preliminary dike design is...
sufficient for these two sections. As expected however, the amount of overtopping at levee section 2 is not negligible at all: \(4.88 \, \text{m}^3/\text{s}\) or \(4,875 \, \text{l/s}\). Although the area behind levee 2 is not developed, this amount of overtopping is way too much with respect to the inner slope stability. Obviously, it can be concluded that the preliminary dike design is not sufficient for levee section 2. There are a few design characteristics which can be adjusted in order to decrease the amount of overtopping: the crest height, the berm width or the slopes of the dike. Table C.4 shows the effects of the adjustments and the most economical design in terms of the required amount of construction material.

Table C.4: Overtopping \(q\) and cross section area \(A\) for levee section 2 as function of the dike height, the slope angle and the berm width.

<table>
<thead>
<tr>
<th>Slope (i) ([-\cdot])</th>
<th>Berm width (B_B) ([m])</th>
<th>Dike height ([m])</th>
<th>(8.5)</th>
<th>(10)</th>
<th>(11.5)</th>
<th>(12.5)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>(q , [l/s])</td>
<td>(A , [m^2])</td>
<td>(q , [l/s])</td>
<td>(A , [m^2])</td>
</tr>
<tr>
<td>1:3</td>
<td>0</td>
<td>1655.29</td>
<td>267.75</td>
<td>722.80</td>
<td>360.00</td>
<td>315.61</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1525.48</td>
<td>342.75</td>
<td>521.36</td>
<td>435.00</td>
<td>178.19</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>1476.55</td>
<td>372.75</td>
<td>457.62</td>
<td>465.00</td>
<td>141.83</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>1429.24</td>
<td>402.75</td>
<td>401.74</td>
<td>495.00</td>
<td>112.92</td>
</tr>
<tr>
<td>1:5</td>
<td>0</td>
<td>1014.53</td>
<td>412.25</td>
<td>252.92</td>
<td>560.00</td>
<td>61.42</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>957.86</td>
<td>487.25</td>
<td>180.93</td>
<td>635.00</td>
<td>34.17</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>941.96</td>
<td>502.25</td>
<td>169.21</td>
<td>650.00</td>
<td>30.40</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>895.88</td>
<td>547.25</td>
<td>138.45</td>
<td>695.00</td>
<td>21.40</td>
</tr>
<tr>
<td>1:8</td>
<td>0</td>
<td>614.02</td>
<td>629.00</td>
<td>62.60</td>
<td>860.00</td>
<td>6.38</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>564.28</td>
<td>704.00</td>
<td>44.65</td>
<td>935.00</td>
<td>3.53</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>554.83</td>
<td>719.00</td>
<td>41.73</td>
<td>950.00</td>
<td>3.14</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>527.44</td>
<td>764.00</td>
<td>34.08</td>
<td>995.00</td>
<td>2.20</td>
</tr>
</tbody>
</table>

With respect to the overtopping criterion \((q < 10 \, l/s)\) only a small amount of options is suitable. As presented in Table C.4, the design with a dike height of 12.5 \(m\), a slope of 1:5 and a berm width of 12 \(m\) requires the smallest amount of material and is therefore the most economical design.

Figure C.9: Sketch of the design for dike sections 2. Note that all height markings are moved 1 \(m\) down: The ground surface level is in fact \(MSL +1 \, m\) and the dike crest height at the moment of construction \(MSL +13.5 \, m\), the relative height levels and required amounts of material however are correct. This is due to the expected settlement of 1 \(m\) due to the weak subsoils, as introduced in Section 2.4.

C.2.4 Environmental section

At the eastern side of Atkinson Island the environmental section is located. It crosses the 1.7 \(km\) wide Cedar Bayou. The environmental section consists of a partly hidden levee on which a road is constructed. At the Bay side a row of dunes is present which is approximately 500-1000 \(m\) wide. A beach forms the transition between the dunes and the water. In the central part of the bayou, a 200 \(m\)
long caisson bridge is constructed which enables water in and outflow. A sketch of the cross-section of the environmental section can be seen in Figure C.10 with indication of some of the different elements.

![Figure C.10: Sketch of the environmental section.](image)

C.2.5 Cost estimate

| Table C.5: Cost estimate for the total system of levees. (Brittin, 2012) |
|-----------------------------|-----------------------------|-----------------------------|
|                            | Section 1       | Section 2       | Section 3       |
| Levee length                | L [m]           | 7000            | 4500            | 9200            |
| Average cross sectional area| A [m²]          | 176.75          | 946.25          | 106.75          |
| Required volume of material  | V [m³]          | 1237250         | 4258125         | 982100,00       |
| Characteristic costs of material | Csand [e/m³] | 5.00            | 5.00            | 5.00            |
| Material costs              | C_{material} [e] | 6.2M            | 21.3M           | 4.9M            |
| Surface area of the slopes  | A_{slope} [m²]  | 110680          | 625943          | 145465          |
| Characteristic costs of slope protection | C_{grass} [e/m²] | 3.00            | 3.00            | 3.00            |
| Slope protection costs      | C_{slope} [e]   | 0.33M           | 1.9M            | 0.44M           |
| Row types                   |                | Industrial      | Undeveloped    | Undeveloped    |
| Average land value cost per m² | C_{land/m²} [e/m²] | 1.00            | 0.30            | 0.30            |
| Required space per meter dike | A_{space/m} [m²] | 176.75          | 143.00          | 106.75          |
| Total required space        | A_{space} [m²]  | 1,237,250       | 643,500         | 982,100         |
| Land value costs            | C_{land} [e]   | 1.2M            | 0.2M            | 0.3M            |
| Additional Earthner levee unit costs (Brittin, 2012) | | | | |
| Levee row clearing and grubbing | C_{A1/m²} [e/m²] | 0.60            | 0.60            | 0.60            |
| Stripping, stockpiling topsoil | C_{A2/m²} [e/m²] | 0.60            | 0.60            | 0.60            |
| Borrow site clearing        | C_{A3/m²} [e/m²] | 0.60            | 0.60            | 0.60            |
| Interior ditch clearing and grubbing | C_{A4/m²} [e/m²] | 0.60            | 0.60            | 0.60            |
| Turf establishment          | C_{A5/m²} [e/m²] | 0.50            | 0.50            | 0.50            |
| Resulting costs             | C_{A,i} [e]    | 3.6M            | 1.9M            | 2.9M            |
| Embankment spreading and compacting | C_{V1/m³} [e/m³] | 2.10            | 2.10            | 2.10            |
| Pit excavation, loading and hauling | C_{V2/m³} [e/m³] | 15.70           | 15.70           | 15.70           |
| Ditch excavation, loading and hauling | C_{V3/m³} [e/m³] | 5.30            | 5.30            | 5.30            |
| Resulting costs             | C_{V,i} [e]    | 28.6M           | 98.4M           | 22.7M           |
| Exterior backslope swales   | C_{swales/m} [e/m] | 5.30            | 5.30            | 5.30            |
| Resulting costs             | C_{swales} [e] | 37100.00        | 23850.00        | 48760.00        |
| **Total costs:**            | e | 195,000,000.00 | 40.1M           | 123.6M          | 31.3M           |
Appendix D

Channel requirements

Required dimensions
In this Appendix the required channel dimensions for the Houston Ship Channel are presented for a number of different vessel types, for a 2-lane traffic. Design rules provided by [PIANC (1997)] and [F&O (2014)] are used, resulting in the required channel width $W_c$ and depth $D_c$ presented in Table D. The other Tables D and D represent the characteristics required to compute the width and depth respectively.

Table D.1: Required channel dimensions, for a 2-lane traffic intensity of each type of vessel.

<table>
<thead>
<tr>
<th>Capacity [10^3 DWT]</th>
<th>Type:</th>
<th>ULCC</th>
<th>VLCC</th>
<th>Suezmax</th>
<th>New-Panamax</th>
<th>Aframax</th>
<th>Panamax</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vessel length L [m]</td>
<td>415</td>
<td>330</td>
<td>285</td>
<td>250</td>
<td>245</td>
<td>230</td>
<td></td>
</tr>
<tr>
<td>Vessel width B [m]</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>49</td>
<td>32</td>
<td>32</td>
<td></td>
</tr>
<tr>
<td>Vessel draught D [m]</td>
<td>28</td>
<td>25</td>
<td>20</td>
<td>15</td>
<td>14</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Channel width: $W_c$ [m]</td>
<td>525</td>
<td>450</td>
<td>375</td>
<td>348</td>
<td>227</td>
<td>227</td>
<td></td>
</tr>
<tr>
<td>Channel depth: $D_c$ [m]</td>
<td>30.5</td>
<td>27.3</td>
<td>22.2</td>
<td>17.2</td>
<td>16.2</td>
<td>14.3</td>
<td></td>
</tr>
</tbody>
</table>

Figure D.1: Some different types of Crude Oil Tankers (left) and a picture of the Suezmax Tanker (right).
### Table D.2: Required channel width calculations (PIANC, 1997; F&O, 2014).

<table>
<thead>
<tr>
<th>Type: Capacity</th>
<th>ULCC 320-550</th>
<th>VLCC 200-320</th>
<th>Suezm. 120-200</th>
<th>New-Pan. 100-150</th>
<th>Afram. 80-120</th>
<th>Panam. 60-80</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vessel length L [m]</td>
<td>415</td>
<td>330</td>
<td>285</td>
<td>250</td>
<td>245</td>
<td>230</td>
</tr>
<tr>
<td>Vessel width B [m]</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>49</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Vessel draught D [m]</td>
<td>28</td>
<td>25</td>
<td>20</td>
<td>15</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td>Manoeuvring lane width 1.5B [m]</td>
<td>105</td>
<td>90</td>
<td>75</td>
<td>73.5</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td>Vessel Clearance [m]</td>
<td>70</td>
<td>60</td>
<td>50</td>
<td>49</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Additional due to:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- crosswind (low) [m]</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>- cross current (low) [m]</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Bank clearance 0.75B [m]</td>
<td>52.5</td>
<td>45</td>
<td>37.5</td>
<td>36.75</td>
<td>24</td>
<td>24</td>
</tr>
<tr>
<td>Navigational aids Good [m]</td>
<td>7</td>
<td>6</td>
<td>5</td>
<td>4.9</td>
<td>3.2</td>
<td>3.2</td>
</tr>
<tr>
<td>Cargo hazard Medium [m]</td>
<td>35</td>
<td>30</td>
<td>25</td>
<td>24.5</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Depth/draught ratio [-]</td>
<td>1.088</td>
<td>1.093</td>
<td>1.112</td>
<td>1.149</td>
<td>1.160</td>
<td>1.189</td>
</tr>
<tr>
<td>Clearance due to ratio $D_c/D$ [m]</td>
<td>28</td>
<td>24</td>
<td>20</td>
<td>9.8</td>
<td>6.4</td>
<td>6.4</td>
</tr>
<tr>
<td>Required Channel Width: $W_c$ [m]</td>
<td>525</td>
<td>450</td>
<td>375</td>
<td>348</td>
<td>227</td>
<td>227</td>
</tr>
</tbody>
</table>

Figure D.2: Required channel width for a 1 or 2 lane traffic channel, from F&O (2014).
### Depth computations

Table D.3: Required channel depth calculations ([PIANC](#), [F&O](#) 2014).

<table>
<thead>
<tr>
<th>Vessel length</th>
<th>Vessel width</th>
<th>Vessel draught</th>
<th>Capacity [10^3 DWT]</th>
<th>Type: ULCC 320-550</th>
<th>VLCC 200-320</th>
<th>Suezm. 120-200</th>
<th>New-Pan. 100-150</th>
<th>Afram. 80-120</th>
<th>Panam. 60-80</th>
</tr>
</thead>
<tbody>
<tr>
<td>L [m]</td>
<td>B [m]</td>
<td>D [m]</td>
<td></td>
<td>415 330 285 250 245 230</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vessel length</td>
<td>Vessel width</td>
<td>Vessel draught</td>
<td></td>
<td>70 60 50 49 32 32</td>
<td>28 25 20 15 14 12</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth estimate: d [m]</td>
<td>Trim [m]</td>
<td>Tidal allowance T [m]</td>
<td>Squad Z(d/D^2) [m]</td>
<td>Exposure (medium) [m]</td>
<td>Fresh water adjustment 0.02D [m]</td>
<td>Bottom material (hard) [m]</td>
<td>Required Channel Depth: D_c [m]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29.5 26.5 21.5 16.5 15.5 13.5</td>
<td>0.3 0.3 0.3 0.3 0.3 0.3</td>
<td>0.1 0.1 0.1 0.1 0.1 0.1</td>
<td>0.655 0.563 0.589 0.686 0.711 0.783</td>
<td>0.15 0.15 0.15 0.15 0.15 0.15</td>
<td>0.56 0.5 0.4 0.3 0.28 0.24</td>
<td>0.9 0.9 0.9 0.9 0.9 0.9</td>
<td>30.5 27.3 22.2 17.2 16.2 14.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure D.3: Required channel depth, from [F&O](#) 2014.
Appendix E

Concrete barrier design

E.1 Gate

E.1.1 Concept

The material used for this gate type is reinforced concrete, C70/85. The gate is a partly hollow wall in order to decrease the required amount of material. Penland and Cibor (2013) developed some designs for this type of barrier, as presented in Figure E.1. The required door height will follow from the surge level and the wave height. The required door width will be the result of a stability analysis, which will be performed in Section E.1.3.

![Figure E.1: Concept of the barrier with a concrete horizontally moveable door: (a) While stocked in a dry dock; (b) In closed position during storm surge; and (c) Free body diagram with important forces regarding the stability of the structure. (Penland and Cibor 2013)](image)

E.1.2 Loads

The horizontal design load will be a combination of the hydraulic head, the horizontal wind and the impact of waves, associated with a 1/1000 years storm event and integrated over the total barrier height. According to Eurocode 0 the total design load can be computed using equation E.1:

\[ q_d = \gamma Q_{1,1} \cdot Q_{\Delta H} + \psi Q (\gamma Q_{\text{wind}} \cdot Q_{\text{wind}} + \gamma Q_{\text{wave}} \cdot Q_{\text{wave}} + \gamma Q_{\text{current}} \cdot Q_{\text{current}}) \]  

(E.1)
In which:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_d$</td>
<td>Design value of distributed horizontal load $[kN/m]$</td>
</tr>
<tr>
<td>$Q_{\Delta H}$</td>
<td>Maximum hydraulic head (8.0 m) $2,091 [kN/m]$</td>
</tr>
<tr>
<td>$\gamma_{Q,\Delta H}$</td>
<td>Partial factor for the main variable load $1.5 [-]$</td>
</tr>
<tr>
<td>$Q_{\text{wind}}$</td>
<td>Characteristic value of wind load $11 [kN/m]$</td>
</tr>
<tr>
<td>$\gamma_{Q,\text{wind}}$</td>
<td>Partial factor for wind load $1.5 [-]$</td>
</tr>
<tr>
<td>$Q_{\text{wave}}$</td>
<td>Characteristic value of wave load $500 [kN/m]$</td>
</tr>
<tr>
<td>$\gamma_{Q,\text{wave}}$</td>
<td>Partial factor for wave load $1.5 [-]$</td>
</tr>
<tr>
<td>$Q_{\text{current}}$</td>
<td>Characteristic value of current load ($u_{\text{current}} = 0 \text{ m/s}$) $0 [kN/m]$</td>
</tr>
<tr>
<td>$\gamma_{Q,\text{wave}}$</td>
<td>Partial factor for current load $1.5 [-]$</td>
</tr>
<tr>
<td>$\psi_Q$</td>
<td>Combination reduction factor for variable loads $0.5 [-]$</td>
</tr>
</tbody>
</table>

With the determined values for the expressed parameters, equation E.1 results in a horizontal design load $q_d = 3,520 \text{ kN/m}$. The images in Figure E.1 have introduced the presence of a sill structure by which the barrier is kept in place. As presented in image (c), a horizontal force will be present between the door bottom and the sill due to friction. It will be assumed that for the final design of the foundation the sill will act like a clamped support. With the use of the ‘Strip Method’ (Fennis, 2012) one is able to estimate the size of the part ($\alpha [-]$) of the horizontal load that is distributed to the sill and the part that is distributed to the abutments. The situation can be modelled as presented in Figure and the.

Sill: $M_{z,\text{max}} = \frac{\alpha q L_z^2}{2} [kNm]$; $\delta_{y,z} = \frac{\alpha q L_z^4}{8E_cI} (1-\alpha)g L_z^4$ $[m]$  
Sides: $M_{x,L/2} = \frac{(1-\alpha)q L_x^2}{24} [kNm]$; $\delta_{y,x} = \frac{(1-\alpha)q L_x^4}{384E_cI}$ $[m]$

In which:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q$</td>
<td>Associated design load $[kN/m]$</td>
</tr>
<tr>
<td>$M_i$</td>
<td>Bending moment at location $i$ $[kNm]$</td>
</tr>
<tr>
<td>$\delta_y$</td>
<td>Deformation in direction $y$, where $\delta_{y,z} = \delta_{y,x}$ $[m]$</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Young’s modulus of reinforced concrete C70/85 $[kN/m^2]$</td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia of the gate structure $[m^4]$</td>
</tr>
<tr>
<td>$L_z$</td>
<td>Total height of the barrier, which will be: $35.0 \text{ m}$</td>
</tr>
<tr>
<td>$L_x$</td>
<td>Total length of the barrier, which will be: $400 \text{ m}$</td>
</tr>
</tbody>
</table>

\[
\frac{\alpha q L_z^4}{8E_cI} = \frac{(1-\alpha)q L_x^4}{384E_cI} \implies \alpha L_z^4 = \frac{1}{48}(1-\alpha) L_x^4 \tag{E.2}
\]

With the requirement that the horizontal displacement in direction $y$ at location $i$ has to be equal for the two bending moments from both directions $z$ and $x$, the above described formula can be rewritten to equation E.2. With the relevant values for $L_z$ and $L_x$, it can be found that $\alpha = 0.9972$. So 99.72 percent of the horizontal load gets distributed to the sill and 0.28 percent is distributed to the sides. The total resulting horizontal load at each abutment ($F_{y,\text{abutment}}$) is than about 2,000 kN. These support reaction force will be assumed to act at the top of the barrier as a result of the maximum displacement in $y$-direction, which is at the top of the barrier.

Load distribution

At the channel, the load is somewhat differently distributed over the door height: as a result of the difference in water levels (hydraulic head) a pressure difference can be seen resulting in a horizontal
load directed towards the river, under the water surface. Additionally there is the load caused by impact of waves at the water surface, and finally there is the wind load which causes a load at the ‘dry’ door area above the water surface. In reality, each of these loads is acting at a different height $z$, as presented in Figure 5.6. For simplicity of the strength checks in the next section, it will be assumed that the sum of these loads (resulting load $q_z$) will be equally distributed over the 35 m high door, with a value $q_z = q_d/h_{door} = 3520/35 = 100 \text{kN/m}^2$.

Harmonic load

The design load $q_d = 3520 \text{kN/m}$ is actually a maximum load associated to a maximum hydraulic head. In reality, the water surface on both sides of the barrier variate in time resulting in a time dependent load. For now, the wind load will be neglected, since it is much smaller than the other two loads due to the hydraulic head and the wind waves. These two loads can both be described as a harmonic wave, with a wave period, a wave length and a propagation speed. Which results in the following sinusoidal approximation for the displacement of the water surface: (Vrijling et al., 2015)

$$\eta(r, t) = a \cdot \sin(\omega t - kr + \alpha) \quad (E.3)$$

In which

- $a = H/2$ Wave amplitude $[m]$
- $\omega = \frac{2\pi}{T}$ Angular frequency $[\text{rad/s}]$
- $k = \frac{2\pi}{L}$ Wave number $[\text{rad/m}]$
- $t$ Point in time $[s]$
- $r$ Point in space $[m]$
- $\alpha$ Phase $[\text{rad}]$

Acting on a surface with width $B$, this results in the sinusoidal load as presented in Equation E.4 describing the load components $F_i$ for either the surge wave or the significant wind waves. The values for the different components are given in Table E.1. The range of the resulting total load due to the water surface difference between the bay and the channel is given in Figure E.2.

$$F_i = B \cdot 0.5 \rho_w g a^2 = B \cdot 0.5 \rho_w g (a \cdot \sin(\omega t - kr + \alpha))^2 \quad [\text{kN}] \quad (E.4)$$

Table E.1: Design values for the load components $F_{\Delta H}$ and $F_{\text{waves}}$, representing the surge and the wind waves respectively. Characteristics for hurricane Ike are used as input values for the size and propagation speed of the hurricane, where wave heights are slightly upscaled to a 1/1000 years storm event.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Surge $F_{\Delta H}$ 80 hours</th>
<th>Waves $F_{\text{waves}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave height</td>
<td>$H_s$</td>
<td>$[m]$</td>
<td>8.0</td>
<td>6.0</td>
</tr>
<tr>
<td>Period</td>
<td>$T$</td>
<td>$[s]$</td>
<td>$2.88 \cdot 10^5$</td>
<td>9.00</td>
</tr>
<tr>
<td>Wave length</td>
<td>$L$</td>
<td>$[m]$</td>
<td>$2.55 \cdot 10^6$</td>
<td>69.0</td>
</tr>
<tr>
<td>Wave celerity</td>
<td>$c$</td>
<td>$[m/s]$</td>
<td>8.86</td>
<td>7.67</td>
</tr>
<tr>
<td>Amplitude</td>
<td>$a$</td>
<td>$[m]$</td>
<td>4.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Angular frequency</td>
<td>$\omega$</td>
<td>$[\text{rad/s}]$</td>
<td>$2.18 \cdot 10^{-5}$</td>
<td>0.698</td>
</tr>
<tr>
<td>Wave number</td>
<td>$k$</td>
<td>$[\text{rad/m}]$</td>
<td>$2.46 \cdot 10^{-6}$</td>
<td>0.091</td>
</tr>
</tbody>
</table>
Figure E.2: Graphs for the water level at the bay side fluctuating in time and the resulting load envelope, for the complete duration of a hurricane. Loads are given as the total load for the full height of the gate and a gate segment width of 50 m.

E.1.3 Cross section

The local navigational depth is 22 m, so the top of the sill will be at a level of $MSL - 22$ m and as well will be the bottom of the door. The associated storm surge level with a 1/1000 years storm event is $MSL + 8$ m, so the water depth during such a storm is about 30 m. The associated significant wave height is 6 m, resulting in water surface variations between $MSL + 5$ m and $MSL + 11$ m. Due to the vertical wall, a relatively big amount of wave impact and overtopping may be expected. A freeboard of 5 m above the mean 1/1000 storm surge level will be used for the gate, which results in a top level of $MSL + 13$ m and a total door height of 35 m. The first assumption for the door width was 15 m. The stability check however, showed that this door width was not enough for stability of any of the cross sections, or it would require a very robust massive door. The door width for the final concrete barrier therefore has been upscaled to 18 m, which will be evaluated in this section by the use of a stability check.

As introduced in Section E.1.1 the door will be constructed out of reinforced concrete. In order to reduce the required amount of material, its cross section will not be a massive rectangle but it will have multiple empty holes, resulting in less environmental impact and a more economical solution. The final design of the cross-section should have an optimal balance between the door its strength and stability and the amount of material required for the construction. Figure E.3 gives an overview of eight different configurations of the cross section. In the next section, for each of the configurations the resulting wall thicknesses required to provide enough strength and the required amount of material per meter barrier will be determined.
Figure E.3: Eight different configurations of the hollow spaces in the door cross-section.

Table E.2: Resulting dimensions for the different configurations. Note that configuration 6 gives the economical optimum in terms of required volume of concrete.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>1*2</th>
<th>1*3</th>
<th>1*4</th>
<th>1*5</th>
<th>1*6</th>
<th>1*7</th>
<th>1*8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total gate width</td>
<td>W_{gate} [m]</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Total gate height</td>
<td>h_{gate} [m]</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>Concrete</td>
<td>V_{concrete} [m^3/m]</td>
<td>280</td>
<td>418</td>
<td>383</td>
<td>323</td>
<td>299</td>
<td>269</td>
</tr>
<tr>
<td>Height openings</td>
<td>h_{opening} [m]</td>
<td>8.05</td>
<td>15.50</td>
<td>10.27</td>
<td>7.15</td>
<td>5.40</td>
<td>4.53</td>
</tr>
<tr>
<td>Submerged dead weight</td>
<td>q_{sw, gate} [kN/m]</td>
<td>4046</td>
<td>6048</td>
<td>5537</td>
<td>4671</td>
<td>4324</td>
<td>3891</td>
</tr>
<tr>
<td>Thickness slabs</td>
<td>t_{c,hor} [m]</td>
<td>0.7</td>
<td>2.0</td>
<td>1.4</td>
<td>1.6</td>
<td>1.6</td>
<td>1.3</td>
</tr>
<tr>
<td>Thickness walls</td>
<td>t_{c,ver} [m]</td>
<td>2.0</td>
<td>5.0</td>
<td>4.4</td>
<td>3.0</td>
<td>2.2</td>
<td>1.7</td>
</tr>
</tbody>
</table>

Stability check

For the stability check, use Figure 5.6 the different loads are expressed in Table E.3. There is stability as long as the moment around point $S_1$ in Figure 5.6 resulting from the horizontal loads and self weight of the barrier, will be anti-clockwise and thus result in an upward directed vertical reaction force in the sill. The self weight has already been determined in Table E.5, the design value follows from the following equation: (Vrijling et al., 2015)

$$G = q_{sw} \cdot \gamma_{G} = 3,891 \cdot 1.0 = 3,981 kN/m$$

As presented in Table E.3, the stability check turns out to be negative: the door is not stable when the holes are empty. This results in a big force at the foundation to keep the door in position. To increase the natural stability, the self weight can be increased by filling the open spaces in the cross section with water, resulting in an additional vertical load of 3,552 kN/m and a total self weight of 7,443 kN/m.

Checking the stability of the sill, the point of rotation will be moved to the center of the sill: $S_s$. The above described situation results in an eccentricity $e = 6.44 m$ (see Figure E.1), where the upward directed support reaction equals the downward directed dead weight. The results for Unity Checks E.12 and E.9 presented in Table E.5 have showed that the horizontal inner slabs of the barrier are strong enough to fill the hollow spaces with water to increase the gate its self weight.
Table E.3: Stability check for the concrete door with configuration 6 by computing the moment around \( S_1 \), which is required to be positive (anti-clockwise) in order to have stability. The points of action, heights on which the resultant forces act, are measured from \( MSL - 22 \, m \).

<table>
<thead>
<tr>
<th>Horizontal Loads</th>
<th>Point of action</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_{h,river} )</td>
<td>2,433 kN/m</td>
</tr>
<tr>
<td>( z_{H,river} )</td>
<td>7.33 m</td>
</tr>
<tr>
<td>( H_{h,bay} )</td>
<td>4,525 kN/m</td>
</tr>
<tr>
<td>( z_{H,bay} )</td>
<td>10.00 m</td>
</tr>
<tr>
<td>( H_{waves} )</td>
<td>500 kN/m</td>
</tr>
<tr>
<td>( z_{H,waves} )</td>
<td>30.00 m</td>
</tr>
<tr>
<td>( H_{wind} )</td>
<td>11 kN/m</td>
</tr>
<tr>
<td>( z_{H,wind} )</td>
<td>32.50 m</td>
</tr>
<tr>
<td>( G_0 )</td>
<td>3,891 kN/m</td>
</tr>
<tr>
<td>( M/S &gt; 0? )</td>
<td>-7,746 Unstable</td>
</tr>
<tr>
<td>( G_{filled} )</td>
<td>7,443 kN/m</td>
</tr>
<tr>
<td>( M/S &gt; 0? )</td>
<td>24,229 Stable</td>
</tr>
</tbody>
</table>

Strength check

For each configuration, presented in Figure E.3, the required wall thicknesses will be determined in order to withstand the loads, from which the amount of required material will result. The assumption has been made that cracking of the concrete is not allowed due to the contact with water, which may result in corrosion of reinforcement steel. The strength characteristics of the used concrete class (C70/85) are given in Eurocode 2 [Bamforth et al., 2008]. The design values for the compressive strength \( f_{ccd} \) and the tensile strength \( f_{ctd} \) follow from Vrijling et al. (2015):

\[
f_{ccd} = \frac{f_{ccm}}{\gamma_M} = \frac{70000}{1.5} = 46.7 \cdot 10^3 kN/m^2 \tag{E.6}
\]

\[
f_{ctd} = \frac{f_{ctm}}{\gamma_M} = \frac{3500}{1.5} = 2.3 \cdot 10^3 kN/m^2 \tag{E.7}
\]

The required strengths of the different walls in the cross section will be checked using so-called Unity Checks (UC). In a unity check the element’s strength \( f_e \) will be divided by the load in the form of the local stress \( \sigma_c \). The element has enough strength when the result of this fracture is bigger than 1, so when the strength is bigger than the load.

In order not to have an increased construction complexity, all vertical walls in the cross section will have the same thickness and the same applies to the horizontal slabs, except for the bottom slab. The following Unity checks will be performed, checking the strengths of the different elements, namely:

- Compression strength
  - Horizontal slabs - compressed by the maximum horizontal loads per horizontal slab. (Equation \[E.8\])
  - Vertical walls - compressed by the maximum vertical loads, which occur at the bottom part of the cross section as a result of the self weight. (Equation \[E.9\])
  - Top slab - At the abutment, a point load of \( F_{y,abutment} = 2,000 \, kN \) acts on the top slab as a result of the part of the horizontal load that is transmitted to the abutment instead of the sill. (Equation \[E.10\])
  - Bottom slab - The bottom slab transmits the total vertical and horizontal load to the foundation, the maximum compression stresses therefore will occur in the bottom slab. (Equation \[E.11\])

- Tensile strength
Horizontal slabs - as a result of the self weight a bending moment will occur in each horizontal slab, which will have a maximum value at the connections with the vertical walls. (Equation E.12)

Vertical walls - in order to distribute the horizontally directed vertically distributed load \( (q_z = 100 \text{kN/m}^2) \) to the horizontal inner walls, a bending moment can be found in the vertical walls, resulting in tensile stresses. (Equation E.13)

The associated Unity Checks are defined as Equations E.8 to E.13, the results are presented in Tables E.4 and E.5.

\[
\frac{f_{ccd}}{q_z \cdot L_{c,z} / t_{hor}} > 1.00 \quad (E.8)
\]

\[
\frac{f_{ccd}}{q_{sw} / t_{ver}} > 1.00 \quad (E.9)
\]

\[
\frac{f_{ccd}}{F_{y,abutment} / (t_{hor} \cdot 1)} > 1.00 \quad (E.10)
\]

\[
\frac{f_{ccd}}{q_d / t_{bot}} > 1.00 \quad (E.11)
\]

\[
\frac{f_{ctd}}{(1/12 \cdot q_{sw,slab} \cdot L_{c,y}^2) / (1/6 \cdot t_{hor}^2)} > 1.00 \quad (E.12)
\]

\[
\frac{f_{ctd}}{(1/12 \cdot q_z \cdot L_{c,z}^2) / (1/6 \cdot t_{ver}^2)} > 1.00 \quad (E.13)
\]

Due to the fact that the total horizontal design load has the same value for each of the configurations, the thickness of the bottom slab will be the same for each design as well. A thickness of 2.0 m will be used, which will be the height of the part that is clamped by the sill structure as well. The compression strength check is presented in Equation E.14, which shows that the strength is more than sufficient.

\[
\frac{f_{ccd}}{q_d / t_{bot}} = \frac{46.7 \cdot 10^3}{3520 / 2.0} = 26.5
\]

APPENDIX E. CONCRETE BARRIER DESIGN
Table E.4: Strength checks. Determination of the required wall thicknesses in order to provide enough strength, for cross-section configurations 1 to 4.

<table>
<thead>
<tr>
<th>Configurations:</th>
<th>1</th>
<th>2*4</th>
<th>1*2</th>
<th>1*3</th>
<th>1*4</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Horizontal slabs</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span ( L_{c,y} ) [m]</td>
<td>6</td>
<td>8</td>
<td>9.2</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Thickness ( t_{c,hor} ) [m]</td>
<td>0.7</td>
<td>2</td>
<td>1.4</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>Compression force ( q_{y,beam} ) [kN/m]</td>
<td>875</td>
<td>1750</td>
<td>1167</td>
<td>875</td>
<td></td>
</tr>
<tr>
<td>Compression stress ( \sigma_{c,beam} ) [kN/m²]</td>
<td>1250</td>
<td>875</td>
<td>833</td>
<td>547</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{c,y}} ) [-]</td>
<td>37</td>
<td>53</td>
<td>56</td>
<td>85</td>
<td></td>
</tr>
<tr>
<td>Selfweight ( q_{sw,beam} ) [kN/m²]</td>
<td>17</td>
<td>49</td>
<td>34</td>
<td>39</td>
<td></td>
</tr>
<tr>
<td>Maximum bending moment ( M_{z,max,beam} ) [kNm/m]</td>
<td>52</td>
<td>262</td>
<td>242</td>
<td>471</td>
<td></td>
</tr>
<tr>
<td>Tensile stress ( \sigma_{t,beam} ) [kN/m²]</td>
<td>631</td>
<td>392</td>
<td>741</td>
<td>1104</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{t,y}} ) [-]</td>
<td>7.29</td>
<td>11.72</td>
<td>6.20</td>
<td>4.17</td>
<td></td>
</tr>
<tr>
<td>Height water column ( h_{w,c} ) [m]</td>
<td>8</td>
<td>15</td>
<td>10</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Vertical water load ( q_{w,beam} ) [kN/m²]</td>
<td>76</td>
<td>151</td>
<td>98</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td>( M_{z,max,beam,w} ) when filled</td>
<td>279</td>
<td>1066</td>
<td>935</td>
<td>1273</td>
<td></td>
</tr>
<tr>
<td>Tensile stress ( \sigma_{t,beam} ) [kN/m²]</td>
<td>3419</td>
<td>1599</td>
<td>2862</td>
<td>2984</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{t,y}} ) [-]</td>
<td>1.35</td>
<td>2.88</td>
<td>1.61</td>
<td>1.54</td>
<td></td>
</tr>
<tr>
<td><strong>Combined selfweight and horizontal load for compression</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending moment halfway ( M_{z,L/2,beam} ) [kNm/m]</td>
<td>140</td>
<td>533</td>
<td>467</td>
<td>637</td>
<td></td>
</tr>
<tr>
<td>Max compression stress ( \sigma_{c,beam,max} ) [kN/m²]</td>
<td>2960</td>
<td>1675</td>
<td>2264</td>
<td>2039</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{c,y}} ) [-]</td>
<td>16</td>
<td>28</td>
<td>21</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td><strong>Vertical walls</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span ( L_{c,z} ) [m]</td>
<td>8.75</td>
<td>17.50</td>
<td>11.67</td>
<td>8.75</td>
<td></td>
</tr>
<tr>
<td>Thickness ( t_{c,ver} ) [m]</td>
<td>2.0</td>
<td>5.0</td>
<td>4.4</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>Bottom compression by sw ( q_{c,z,sw} ) [kN/m]</td>
<td>2227</td>
<td>4930</td>
<td>4466</td>
<td>3664</td>
<td></td>
</tr>
<tr>
<td>Compression stress ( \sigma_{c,side} ) [kN/m²]</td>
<td>1113</td>
<td>986</td>
<td>1015</td>
<td>1221</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{c,y}} ) [-]</td>
<td>42</td>
<td>47</td>
<td>46</td>
<td>38</td>
<td></td>
</tr>
<tr>
<td>Horizontal water load ( q_{w,side} ) [kN/m²]</td>
<td>945</td>
<td>1950</td>
<td>1307</td>
<td>1035</td>
<td></td>
</tr>
<tr>
<td>( M_{z,max,beam,side} ) when filled</td>
<td>6,029</td>
<td>49,766</td>
<td>14,821</td>
<td>6,604</td>
<td></td>
</tr>
<tr>
<td>Tensile stress ( \sigma_{t,beam} ) [kN/m²]</td>
<td>9,044</td>
<td>11,944</td>
<td>4,593</td>
<td>4,402</td>
<td></td>
</tr>
<tr>
<td>Unity check: ( \frac{f_{cd}}{\sigma_{t,y}} ) [-]</td>
<td>0.51</td>
<td>0.39</td>
<td>1.00</td>
<td>1.04</td>
<td></td>
</tr>
<tr>
<td>Configurations:</td>
<td>5</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>-----------------</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td><strong>Horizontal slabs</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span</td>
<td>(L_{c,y}) [m]</td>
<td>13.6</td>
<td>14.6</td>
<td>15.2</td>
<td>15.5</td>
</tr>
<tr>
<td>Thickness</td>
<td>(t_{c,hor}) [m]</td>
<td>1.6</td>
<td>1.3</td>
<td>1.3</td>
<td>1.3</td>
</tr>
<tr>
<td>Compression force</td>
<td>(q_{y,beam}) [kN/m]</td>
<td>700</td>
<td>583</td>
<td>500</td>
<td>438</td>
</tr>
<tr>
<td>Compression stress</td>
<td>(\sigma_{c,beam}) [kN/m²]</td>
<td>438</td>
<td>449</td>
<td>385</td>
<td>337</td>
</tr>
<tr>
<td>Unity check:</td>
<td>(\frac{f_{cd}}{\sigma_{y}}) [-]</td>
<td>107</td>
<td>104</td>
<td>121</td>
<td>139</td>
</tr>
<tr>
<td>Selfweight</td>
<td>(q_{sw,beam}) [kN/m²]</td>
<td>39</td>
<td>32</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Maximum bending moment</td>
<td>(M_{z,max,beam}) [kN/m]</td>
<td>605</td>
<td>566</td>
<td>614</td>
<td>663</td>
</tr>
<tr>
<td>Tensile stress</td>
<td>(\sigma_{t,beam}) [kN/m²]</td>
<td>1418</td>
<td>2011</td>
<td>2179</td>
<td>2355</td>
</tr>
<tr>
<td>Unity check:</td>
<td>(\frac{f_{ctd}}{\sigma_{y}}) [-]</td>
<td>3.25</td>
<td>2.29</td>
<td>2.11</td>
<td>1.95</td>
</tr>
<tr>
<td>Height water column</td>
<td>(h_{w,c}) [m]</td>
<td>5</td>
<td>4</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Vertical water load</td>
<td>(q_{w,beam}) [kN/m²]</td>
<td>49</td>
<td>41</td>
<td>32</td>
<td>26</td>
</tr>
<tr>
<td>(M_{z,max}) when filled</td>
<td>(M_{z,max,beam,w}) [kN/m]</td>
<td>1364</td>
<td>1287</td>
<td>1233</td>
<td>1202</td>
</tr>
<tr>
<td>Tensile stress</td>
<td>(\sigma_{t,beam}) [kN/m²]</td>
<td>3197</td>
<td>4568</td>
<td>4379</td>
<td>4267</td>
</tr>
<tr>
<td>Unity check:</td>
<td>(\frac{f_{ctd}}{\sigma_{y}}) [-]</td>
<td>1.44</td>
<td>1.01</td>
<td>1.05</td>
<td>1.08</td>
</tr>
<tr>
<td><strong>Vertical walls</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span</td>
<td>(L_{c,z}) [m]</td>
<td>7.00</td>
<td>5.83</td>
<td>5.00</td>
<td>4.38</td>
</tr>
<tr>
<td>Thickness</td>
<td>(t_{c,ver}) [m]</td>
<td>2.2</td>
<td>1.7</td>
<td>1.4</td>
<td>1.1</td>
</tr>
<tr>
<td>Bottom compression by sw</td>
<td>(q_{c,z,sw}) [kN/m]</td>
<td>3330</td>
<td>2939</td>
<td>2967</td>
<td>3013</td>
</tr>
<tr>
<td>Compression stress</td>
<td>(\sigma_{c,side}) [kN/m²]</td>
<td>1514</td>
<td>1729</td>
<td>2119</td>
<td>2739</td>
</tr>
<tr>
<td>Unity check:</td>
<td>(\frac{f_{ctd}}{\sigma_{y}}) [-]</td>
<td>31</td>
<td>27</td>
<td>22</td>
<td>17</td>
</tr>
<tr>
<td>Horizontal water load</td>
<td>(q_{w,side}) [kN/m²]</td>
<td>860</td>
<td>713</td>
<td>630</td>
<td>568</td>
</tr>
<tr>
<td>(M_{z,max}) when filled</td>
<td>(M_{z,max,beam,w}) [kN/m]</td>
<td>3512</td>
<td>2023</td>
<td>1313</td>
<td>905</td>
</tr>
<tr>
<td>Tensile stress</td>
<td>(\sigma_{t,beam}) [kN/m²]</td>
<td>4353</td>
<td>4200</td>
<td>4018</td>
<td>4489</td>
</tr>
<tr>
<td>Unity check:</td>
<td>(\frac{f_{ctd}}{\sigma_{y}}) [-]</td>
<td>1.06</td>
<td>1.10</td>
<td>1.14</td>
<td>1.02</td>
</tr>
</tbody>
</table>

Tables E.4 and E.5 have presented the required wall thicknesses and the resulting required amounts of concrete for the eight different configurations of the cross section. As presented in Table E.2, the smallest required amount of concrete can be found for configuration 6, with 6 holes of 4.53 m high and 14.60 m wide. According to this computation, this will be the economical optimal configuration in terms of required amount of material. For this configuration, the height of the holes provide enough space to carry out maintenance if required.

**Vibrations**

The above described stability and strength checks are executed in a deterministic way, using maximum values for the occurring loads. In reality these loads are fluctuating in time, causing vibrations on the structure. Unstability of the structure may occur due to vibrations as well. These vibrations, or excitation, may be induced by waves and unstable flow patterns. The resulting harmonic load has been described in Table E.1 in Section E.1.2. The resulting vibrations follow from basic dynamics, as...
described by the equation of motion:

\[(m + m_w) \frac{d^2y}{dt^2} + (c + c_w) \frac{dy}{dt} + (k + k_w)y = F\]  
\[(E.15)\]

In which:
- \(m\) Mass \([kg]\)
- \(y\) Displacement \([m]\)
- \(t\) Time \([s]\)
- \(c\) Damping \([Ns/m]\)
- \(k\) Stiffness \([N/m]\)
- \(F\) Impressed force \([N]\)
- \(m_w, c_w, k_w\) Hydrodynamic components of \(m, c\) and \(k\)

The strength checks of the previous section have already presented that the strength of the different gate elements is sufficient to bear the maximum loads. However, vibrations which are close to the eigenfrequency \(p\) of the structure may be a new problem, called resonance. To address this issue, the characteristics of the structure will be determined in the following phrases. The frequency which may be a problem is the one associated to the wind waves \(\omega_{waves} = 0.698\) rad/s (Table E.1), where the structure will encounter resonance when equation (E.16) is valid.

\[\bar{\omega} = \frac{\omega}{p} = 1\]  
\[(E.16)\]

![Figure E.4: Modelling of a frame structure with 6 floors.](image)

The response of the gate, in terms of displacement \(y\), speed \(\dot{y}\) and acceleration \(\ddot{y}\), depends on the stiffness of the gate. Where the motion parameters for \(y\) describe the eigenfrequency \(p\). The gate can be modelled as a frame structure\(^1\) as presented in Figure E.4. The gate width is divided into segments of 50 m each, covering the distance between two bearing sections surrounding the hydrofenders. For reinforced concrete, (ECS, 2004) gives a damping of 10 percent for reinforced concrete. The stiffness can be determined for each wall using equation (E.17). With wall height \(L = 5.83\) m, moment of inertia \(I_y = 20\) m\(^4\) (using width \(B = 50\) m and wall thickness \(t = 1.7\) m) and Young's Modulus \(E_c = 38,500\) N/mm\(^2\) this results in a stiffness \(k_{wall} = 47.7 \cdot 10^6\) kN/m for each wall.

\[k = \frac{12E_cI_y}{L^3}\]  
\[(E.17)\]

\(^1\)The response of the gate as the presented frame structure, is confirmed by a computational model using Matrixframe structural modelling software, as presented in the next section. However, this model is only a worst case scenario, giving the situation for which the buttress walls are not contributing to the load transmission.
Figure E.5 gives the results of the computational model for the response of the frame structure. The parameters presented in Figure E.4 can all be computed, resulting in the following structural matrices:

\[
\text{Mass matrix: } [M] = \begin{bmatrix}
29250 & 0 & 0 & 0 & 0 & 0 \\
0 & 29250 & 0 & 0 & 0 & 0 \\
0 & 0 & 29250 & 0 & 0 & 0 \\
0 & 0 & 0 & 29250 & 0 & 0 \\
0 & 0 & 0 & 0 & 29250 & 0 \\
0 & 0 & 0 & 0 & 0 & 29250
\end{bmatrix} \text{[kN];}
\]

\[
\text{Stiffness matrix: } [K] = \begin{bmatrix}
9.55 & -9.55 & 0 & 0 & 0 & 0 \\
-9.55 & 19.1 & -9.55 & 0 & 0 & 0 \\
0 & -9.55 & 19.1 & -9.55 & 0 & 0 \\
0 & 0 & -9.55 & 19.1 & -9.55 & 0 \\
0 & 0 & 0 & -9.55 & 19.1 & -9.55 \\
0 & 0 & 0 & 0 & -9.55 & 19.1
\end{bmatrix} \cdot 10^7 \text{[kN/m];}
\]

\[
\text{Flexibility matrix: } [F] = [K]^{-1} = \begin{bmatrix}
0.95 & 1.16 & 1.50 & 2.10 & 3.49 & 10.5 \\
1.16 & 1.16 & 1.50 & 2.10 & 3.49 & 10.5 \\
1.50 & 1.50 & 1.50 & 2.10 & 3.49 & 10.5 \\
2.10 & 2.10 & 2.10 & 2.10 & 3.49 & 10.5 \\
3.49 & 3.49 & 3.49 & 3.49 & 3.49 & 10.5 \\
10.5 & 10.5 & 10.5 & 10.5 & 10.5 & 10.5
\end{bmatrix} \cdot 10^{-9} \text{[m/kN];}
\]

\[
\text{Dynamic matrix: } [D] = [F][M] = \begin{bmatrix}
2.79 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
3.40 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
4.38 & 4.38 & 4.38 & 6.13 & 10.2 & 30.6 \\
10.2 & 10.2 & 10.2 & 10.2 & 10.2 & 30.6 \\
30.6 & 30.6 & 30.6 & 30.6 & 30.6 & 30.6
\end{bmatrix} \cdot 10^{-5} \text{[m]}
\]
The eigenvalues and corresponding eigenvectors can now be determined using the Method of Stodola (Clough and Penzien 2003), using the following expression:

\[ [D] \{ \phi \} = \lambda \{ \phi \} = \frac{1}{\rho^2} \{ \phi \} \]  
(E.18)

Now the first mode follows from the requirement \( \{ \phi \}_1 = [D] \{ \phi \}_0 \), which can be found by executing a matrix iteration, starting with a first estimate \( \{ \phi \}^2 = \{ 1, 1, 1 \} \):

\[
\begin{bmatrix}
2.79 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
3.40 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
4.38 & 4.38 & 4.38 & 6.13 & 10.2 & 30.6 \\
10.2 & 10.2 & 10.2 & 10.2 & 10.2 & 30.6 \\
30.6 & 30.6 & 30.6 & 30.6 & 30.6 & 30.6 
\end{bmatrix}\begin{bmatrix}1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{bmatrix} = \begin{bmatrix}5.755 \\ 5.817 \\ 6.011 \\ 6.537 \\ 8.171 \\ 18.39 \end{bmatrix} \cdot 10^{-4} = \begin{bmatrix}0.313 \\ 0.316 \\ 0.327 \\ 0.356 \\ 0.444 \\ 1.000 \end{bmatrix} \]  
(E.19)

Next, a trial vector of \( \{ \phi \}^T = \{ 0.313, 0.316, 0.327, 0.356, 0.444, 1.000 \} \) is assumed continuing with the next iteration:

\[
\begin{bmatrix}
2.79 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
3.40 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
4.38 & 4.38 & 4.38 & 6.13 & 10.2 & 30.6 \\
10.2 & 10.2 & 10.2 & 10.2 & 10.2 & 30.6 \\
30.6 & 30.6 & 30.6 & 30.6 & 30.6 & 30.6 
\end{bmatrix}\begin{bmatrix}0.313 \\ 0.316 \\ 0.327 \\ 0.356 \\ 0.444 \\ 1.000 \end{bmatrix} = \begin{bmatrix}4.074 \\ 4.094 \\ 4.155 \\ 4.322 \\ 4.858 \\ 8.446 \end{bmatrix} \cdot 10^{-4} = \begin{bmatrix}0.482 \\ 0.485 \\ 0.492 \\ 0.512 \\ 0.575 \\ 1.000 \end{bmatrix} \]  
(E.20)

This iteration process will be repeated after which the following mode shape and eigenfactor are found:

\[
\begin{bmatrix}
2.79 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
3.40 & 3.40 & 4.38 & 6.13 & 10.2 & 30.6 \\
4.38 & 4.38 & 4.38 & 6.13 & 10.2 & 30.6 \\
10.2 & 10.2 & 10.2 & 10.2 & 10.2 & 30.6 \\
30.6 & 30.6 & 30.6 & 30.6 & 30.6 & 30.6 
\end{bmatrix}\begin{bmatrix}0.430 \\ 0.433 \\ 0.441 \\ 0.464 \\ 0.535 \\ 1.000 \end{bmatrix} = \begin{bmatrix}0.430 \\ 0.433 \\ 0.441 \\ 0.464 \\ 0.535 \\ 1.000 \end{bmatrix} \cdot 10^{-4} = \begin{bmatrix}0.430 \\ 0.433 \\ 0.441 \\ 0.464 \\ 0.535 \\ 1.000 \end{bmatrix} \]  
(E.21)

Now the first mode shape \( \{ \phi \}_1 \), combined with Equation [E.18], gives the first natural frequency as presented by Equation [E.22]. This first natural frequency is much larger than the frequency of the load by the waves \( \omega_{wave} = 0.698 \ rad/s \), therefore the resonance criterion presented in Equation [E.16] has not been met. Since the higher order natural frequencies are by definition larger than the first mode, the resonance criterion will never be met. It can be concluded that the loads due to vibrations will not be an issue.

\[
\lambda_1 = \frac{1}{p_1^2} = 10.14 \cdot 10^{-4} \rightarrow p_1 = 31.4 rad/s \]  
(E.22)

### E.1.4 Reinforcement

It has been mentioned that significant cracking \( w_c > 0.2 \ mm \) of the concrete is not allowed to prevent contact between the water and the reinforcement steel. A high strength of the concrete is therefore required in combination with reinforcement. As presented in the previous section, the used
The concrete class is C70/85. By the use of the software MatrixFrame, the internal load transmission is quantified as presented in Figures E.5 and E.6.

As presented in Figure E.5 (b), the maximum tensile stresses in the concrete $\sigma_c \approx 6 \, N/mm^2$ for the 1/1,000 year load event. These maximum tensile stresses occur at the connections between the internal floor slabs and the outer walls. As presented in the previous section, the concrete design tensile strength $f_{cd} = 2.3 \, N/mm^2$, which is smaller than the occurring load so cracking will occur. According to Bamforth et al. (2008) concrete is able to restore very small cracks of 1 mm by itself, bigger cracks are possible to repair when the gate is in the dry dock. Reparations are expected to be required only a few times during the lifetime of the barrier, which keeps the expected amount of required maintenance pretty small. The occurring crack widths need to be limited by the reinforcement.

Figure E.6: Modelling of internal load transmission of the concrete gate: (a) Load and supports at the bottom. (b) Internal stress at the concrete, maximum $\sigma_c = 6.0 \, N/mm^2$ near the connections.

At the locations where $\sigma_c > f_c$ cracking occurs and the loads are transmitted to the reinforcement instead of the concrete. The required amount of reinforcement can be determined using the formulations provided by Bamforth et al. (2008). Due to time dependent processes like creep, the modulus of elasticity of the concrete can reduce by $E_{cd} = E_{cm}/\gamma_c$. In this calculation a factor $\gamma_c = 1.2$ [-] is used resulting in a design value for the modulus $E_{cd} = 32 \, GPa = 32 \times 10^3 \, N/mm^2$.

Figure E.11 (b), gives the set of equations to determine the loads in the concrete $N_c$ and in the reinforcement steel $N_s$. In ultimate limit state, the concrete strain is $\epsilon_c = 3.5 \times 10^{-3}$ [-] and the steel strain is $\epsilon_s = \sigma_s/E_s$ in which the steel stress $\sigma_s = f_{yd}$. By determination of the maximum height of the concrete compression zone $x_u$, the maximum load in the concrete can be determined. As a result of the horizontal force equilibrium ($N_c = N_s$) the required amount of reinforcement steel $A_s$ follows from Equation E.23.

$$x_u = \frac{A_s f_{yd}}{\alpha b f_{cd}} \rightarrow A_s > \frac{\alpha b x_u f_{cd}}{f_{yd}}$$  \hspace{1cm} (E.23)

In which:
- $f_{yd}$ Design yield strength of steel B500B: \hspace{1cm} 435 \, [N/mm^2]
- $f_{cd}$ Design compression strength of concrete C70/85: \hspace{1cm} 46.7 \, [N/mm^2]
- $\alpha$ Area factor, for C70/85: \hspace{1cm} 0.62 \, [-]
- $b$ Width of the wall, will be considered per 1 m \hspace{1cm} 1,000 \, [mm]
- $x_u$ Height of concrete compression zone \hspace{1cm} [mm]

Fennis (2012) gives for concrete slabs a minimum amount of reinforcement as presented by Equation E.24. In which the characteristic value of the mean concrete tensile strength $f_{ctm} = 3.5 \, N/mm^2$ for concrete class C70/85 and the characteristic value of the reinforcement steel yield strength $f_{yk} = 500 \, N/mm^2$. For an effective slab thickness $d \approx 1,200 \, mm$ this gives a minimum amount of reinforcement.
of $A_{s,\text{min}} = 2,184 \, \text{mm}^2/\text{m}$, which is larger than $0.0013bd = 1,560 \, \text{mm}^2/\text{m}$ and therefore meets the requirement.

$$A_{s,\text{min}} = 0.26 \frac{f_{\text{s,mbd}}}{f_{\text{yk}}} > 0.0013bd \quad (E.24)$$

This minimum amount of reinforcement results in a height of the concrete compression zone $x_u \approx 35 \, \text{mm}$ and a resulting moment resistance $M_{\text{Rd}} \approx 800 \, \text{kNm}$. The maximum occurring bending moment at the connections is approximately 1,300 $\text{kNm}$ so the minimum reinforcement is not sufficient. Sufficient reinforcement can be found for application of a base reinforcement $\Phi18 - 100$ (2,544 $\text{mm}^2/\text{m}$) in y-direction. It is recommended to add, for instance, $\Phi12 - 100 = 1,131 \, \text{mm}^2/\text{m}$ at the connections between the walls and the slabs to have concentrated reinforcement to resist the high negative (hogging) bending moment at these connections. It is expected that this reinforcement application will also be sufficient for the reinforcement of the concrete walls.

### E.2 Abutments

#### E.2.1 Functions

As mentioned, there will be an abutment on each side of the channel. One will be big, offering enough space to store the gate during normal weather conditions and to enable maintenance. The other one will be small, clamping the gate on its associated side and bearing the available horizontal loads by transmission to the soil. Obviously, these two docks have different functions. Both abutments will carry the function of load transmission. Horizontal loads will be distributed to the soil by the abutment heads. Vertical loads will be transmitted to the soil by the foundation. In addition to load transmission, the biggest dock, also called the Dry Dock, has the function of offering pace for both storage and maintenance. To fulfill this function, the dock requires a few elements:

- A system that enables filling the dock with water and also and emptying of the dock.
- Soil from outside the dock causes a great horizontal load, which has to be retained by the use of soil retaining walls.
- The dock has to be accessible for persons, materials and equipment. This requires access stairs or lifts, and movable equipment.

### Load transmission

Load transmission will be done at the abutment heads, which will be concrete pillars pointing in the channel. Besides hydraulic loads distributed from the gates to the abutments, the pillars have to be designed to resist other loads as well. The most important of these loads is the load due to ship collision, which is according to [Vrijling et al. (2015)] theoretically the same process as mooring of a ship, described by Equation [E.25] in which the pillar is assumed to be a linear elastic structure.

$$F = k\Delta x = \sqrt{2kE_{\text{kin, max}}} \quad (E.25)$$

In which:

- $F$ Force on a linear elastic structure $[\text{kN}]$
- $k$ Stiffness of the structure $[\text{kN/m}]$
- $\Delta x$ Deformation in direction $x$ $[\text{m}]$
- $E_{\text{kin, max}}$ The total amount of kinetic energy to be absorbed $[\text{kNm}]$
Where the total amount of kinetic energy to be absorbed by the structure can be computed by the following equation:

\[
E_{kin} = \frac{1}{2} (m_s + m_w) v_s^2 C_E C_S C_C
\]

(E.26)

In which:

- \( C_E \) Eccentricity coefficient 0.75
- \( C_S \) Softness coefficient 0.9
- \( C_C \) Configuration coefficient 1.0
- \( m_s \) Mass of the ship 230 \( \cdot \) 10\(^6\) kg
- \( m_w \) Mass of the amount of displaced water 91.8 \( \cdot \) 10\(^6\) kg
- \( v_s \) Velocity of the ship perpendicular to the structure 0.15 m/s

The given values for the parameters are obtained by the use of Vrijling et al. (2015) and are characteristic for the design vessel, which is a Suezmax tanker. This type of vessel has capacities of 130-200 DWT, length \( L = 285 \) m, width \( W = 50 \) m and draught \( D = 20 \) m. Resulting in a total amount of kinetic energy to be absorbed by the structure as presented in Equation E.27. The design values for all other parameters will be discussed in Section 2.3 Design of the pillars.

\[
E_{kin} = \frac{1}{2} (230 + 91.8) \cdot 10^6 \cdot 0.15^2 \cdot 0.75 \cdot 0.9 \cdot 0.1 = 2.44 \cdot 10^3 kNm
\]

(E.27)

Soil retaining

In order to create a relatively deep building pit or dry dock, multiple actions have to be taken besides excavations. Issues that are associated to this are the soil stability and the ground water level. The soil is unstable if the required slope of the surface exceeds the angle of repose of the soil. To prevent sliding of the soil, when this maximum angle to the horizontal is exceeded, a soil retaining wall is required. These are walls intended to resist horizontal soil or water pressure. In order to create the abutments, a proper design of the walls is required to prevent water leakage and soil sliding. Vrijling et al. (2015) distinguishes three main types of soil retaining structures: (Figure E.7)

1. Gravity walls - Often masonry or concrete walls, including L-walls.
2. Sheet pile walls - Mostly steel profiles, possibility of anchoring, including combi-walls.

Figure E.7: Main types of soil retaining walls, obtained from Vrijling et al. (2015).
E.2.2 Overview

An overview of the both abutments is given in Figure 5.2 where the dry dock is presented in more detail in Figure 5.21 Chapter 5. As presented, the main elements the abutments consist of are the abutment heads (pillars), the bottom slab (sill) and the soil retaining walls on the sides. These three elements are designed in the following sections. Figure E.8 presents an overview of the abutment head at the dry dock side. Some elements that have to be designed are pointed out with A-G, where the sill at the channel bottom (G) will be designed in Appendix G as part of the Foundation design.

![Figure E.8: Different installations and structural elements to be designed for the dry dock.](image)

E.2.3 Design of the pillars

The pillars are located at the abutment heads, where the channel bank begins at the bottom of the channel. The local depth is $MSL - 22.0 \text{ m}$, and decreases towards the crest of the dike that surrounds the channel. The top of the pillars will be at a level of $MSL + 14 \text{ m}$, being 1.0 m higher than the top of the gate to provide guidance over the full gate height. This results in a pillar height of 36 m from top to the channel bottom.

The abutment head will have the same shape for each side, consisting of two robust pillars at
each abutment which have the function to bear the horizontal loads, keeping the gate at its place and minimizing the amount of leakage. The pillars will have the shape of an octagon consisting of eight sides of 4 m, resulting in a diameter of 9.66 m. The eight outer walls of each pillar will have a thickness of 2.0 m, resulting in the following cross-sectional areas: Total \( A_{\text{total}} = 77.3 \, \text{m}^2 \); Hollow space \( A_{\text{in}} = 26.5 \, \text{m}^2 \); and Effective concrete area \( A_{\text{c}} = 50.8 \, \text{m}^2 \). The resulting stiffness \( k_{\text{pillar}} \) of this structure against a distributed load over the pillar height follows from the following equation \[ E.28 \]

\[
\delta = \frac{q z^4}{8EI} = \frac{F}{k} \delta_{\text{m}} \Rightarrow k_{\text{pillar}} = \frac{8EI}{z^4}
\]

In which:
- \( k \): Stiffness of the structure \([\text{kN/m}]\)
- \( \delta \): Horizontal displacement \([\text{m}]\)
- \( q \): Horizontal load, distributed over the pile height \( z \) \([\text{kN/m}]\)
- \( F \): Total horizontal load: \( F = q z \) \([\text{kN}]\)
- \( z \): Effective pile height \( 30 \, \text{m} \)
- \( E \): Stiffness of the concrete \( 38,500 \, \text{N/mm}^2 \)
- \( I \): Moment of inertia \( 640 \, \text{m}^4 \)

Note that the effective pile height \( z = 30 \, \text{m} \), which is the wet height for a 1/1000 years storm event from the channel bottom (\( MSL - 22.0 \, \text{m} \)) to the surge level \( MSL + 8.0 \, \text{m} \). In reality the horizontal load is not equally distributed over this height, but to observe a worst case scenario it will be taken as constant resulting in a total load \( F = q z \). In addition to this, the real stiffness will be bigger when the hollow space in the pillars is filled with sand, which increases the mass of the structure. Figure E.9 shows three different load types acting on the pillar.

Figure E.9: Loads at a pillar: (a) Horizontal hydraulic load; (b) Horizontal load for a 1/1000 years storm that is distributed from the gate to the pillar; (c) Load due to ship collision.

**Hydraulic load**

The hydraulic load acting on the pillars is similar to the hydraulic loads acting on the Barrier. The maximum hydraulic load can be found for a 1/1,000 years storm event at the side of the Bay, where a surge elevation of 8.0 m and maximum waves of 6.0 m can be found. As presented before, this load \( q_d = 3,520 \, \text{kN/m} \) distributed over the horizontal or a horizontally acting pressure of about 100 \( \text{kN/m}^2 \). If this pressure is equally distributed to the concrete, it results in an overall compression stress \( \sigma_c = 0.1 \, \text{N/mm}^2 \), which is much smaller than either the compression or the flexural strength. It is not expected that this load gives any problems considering the strength.
Load from gate

As presented in Figure E.9, a horizontal point load is acting on the pillar on the river side when the
gate is in operation. The design value of this load is associated with a 1/1,000 years storm event.
According to the Strip Method (Section 5.3.2) this point load has a value of about \( F = 2,000 \, kN \).
The location the load acts on the pillar is assumed to be on the highest 1 m gate height, due to
rotation around the sill. With \( z = 35 \, m \) this results in the following displacement, which gives no
issue.

\[
\delta = \frac{Fz^3}{3EI} = \frac{2000 \cdot 35^3}{3 \cdot 38,500 \cdot 10^3 \cdot 640} = 0.0012 \, m
\]  
(E.29)

Collision force

As mentioned before, the collision force of ships is an important load by which the pillars have to
be designed. This load is described by he stiffness of the structure and the total amount of kinetic
energy of the design vessel that has to be absorbed. The amount of energy to be absorbed has been
determined by Equation E.27, which resulted in \( E_{kin} = 2.44 \cdot 10^9 \, kNm \). The stiffness of the structure
depends on its design, including the foundation. For the pillar a stiffness \( k_{pillar} = 7.30 \cdot 10^6 \, kN/m \)
has been found by the use of Equation E.28, resulting in a collision force presented in Equation E.30.
The computed collision force causes a maximum horizontal displacement \( \delta = 2.6 \, cm \), which does not
exceed the survivability limit state \( \) (Vrijling et al., 2015).

\[
F = \sqrt{2kE_{kin,max}} = \sqrt{2 \cdot 7.30 \cdot 10^6 \cdot 2.44cdot10^3} = 189 \cdot 10^3 \, kN
\]  
(E.30)

\[
F = k \cdot \delta \rightarrow \delta = \frac{F}{k} = \frac{189 \cdot 10^3}{7.30 \cdot 10^6} = 0.026 \, m
\]  
(E.31)

Note that the collision force is a point load, which makes the displacement formula to determine
the stiffness look like Equation E.29 instead of the one presented by Equation E.28, resulting in
stiffness reduction of \( \frac{3}{8} \). However, at the determination of the stiffness in Equation E.28 an effective
pillar length \( z = 30 \, m \) has been used, where under normal navigational conditions, the channel depth
\( z_0 = 22 \, m \). For these conditions a stiffness \( k_{pillar,z0} = 7.00 \cdot 10^6 \, [kN/m] \) results, which is comparable
to the one initially used.

E.2.4 Design of the bottom slab

The shape of the bottom slab of the abutments will be more or less the same as the shape of the sill at
the channel. As presented in Appendix \( \) the pile foundation will be exactly the same, using inclined
piles with an inclination of 1:3 penetrating the soil to a bearing depth varying between \( MSL - 54 \, m \)
and \( MSL - 64 \, m \). It is expected that the soil retaining diaphragm wall at the sides of the dock
will have at least the same bearing depth as the pile foundation, but without inclination. Therefore
enough space between the retaining walls and the pile foundation is required. The maximum pile
length is associated with a bearing depth of \( MSL - 64 \, m \). The bottom of the sill is at a depth of
\( MSL - 28.5 \, m \). A substraction of these values results in a vertical distance between the pile length
and pile foot of 35.5 m and a horizontal distance of 11.8 m. The minimum required additional bottom
slab width is thus about 12.0 m. The total sill width is than given by the following summation:

\[
W_{sill, dock} = W_{gate} + 2 \cdot W_{sidesupport} + 2 \cdot W_{piles} = 18.0 + 2 \cdot 5.0 + 2 \cdot 12.0 = 52.0 \, m
\]  
(E.32)

Figure E.10 gives an overview of the soil loads acting on the dry dock. As presented in the right
image, the resulting horizontal load on the sill is 20,160 kN/m. With a sill thickness \( t_{sill, dock} = 2.0 \)
This results in an average compression stress \( \sigma_{c,sill} = 10,080 \text{ kN/m}^2 \) or 10.08 \( \text{N/mm}^2 \). The design compression strength associated with reinforced concrete C70/85 is \( f_{cd} = 46.67 \text{ N/mm}^2 \), which is much larger than required, so the sill thickness of 2.0 \( m \) is sufficient to bear the horizontal loads from the soil.

Up bursting Check

Besides horizontal loads, the sill has to deal with great vertical loads. As presented in Appendix G the maximum vertical loads can be found at the channel and not in one of the docks. The foundation however will be similar, so it will be assumed that the sill is able to resist the vertical downward directed loads. The bottom of the dock, the sill, will be located much lower than the ground water level. Due to this, an upward water pressure will be acting on the sill. When the upward load is larger than the downward load, the structure may fail. This failure mode is called upbursting.

The upward load will be maximum when the dock is completely empty, so without any water or gate storage. This is only the case during construction, since the gate will be only able to move in and out of the dry dock by filling the dock with water. The upward directed water load can be determined with the following equation, in which \( z_{sillbottom} \) and \( z_{groundwater} \) are the levels of the bottom of the sill and the ground water respectively and \( \gamma_w \) is the specific weight of water:

\[
p_{up} = (z_{sillbottom} - z_{groundwater}) \cdot \gamma_w = (-26 - 0) \cdot 10 = -260 \text{ kN/m}^2
\]  

(E.33)

The downward directed sill weight for a sill thickness of 2.0 \( m \) is 50 \( \text{kN/m}^2 \), using a specific weight of reinforced concrete \( \gamma_c = 25 \text{ kN/m}^3 \), which is much smaller than the upward directed load. Due to this it can be expected that the piles of the pilefoundation will act like tension piles. As presented in Appendix G Table G.6 the minimum vertical resistance of the tension piles \( R_{t,t,V} = 10,400 \text{ kN} \), as a result of skin friction and pile dead weight. On each sill slab \( (A = 20 \times 52 \text{ m}^2) \) there are 12
piles installed, so the average additional downward directed load is 120 kN/m². The total downward directed load \( q_{c,\text{down}} \) and upward water load \( q_{p,\text{up}} \) over the 52 m wide sill become:

\[
q_{c,\text{down}} = (50 + 120) \cdot 52 = 8,840 \text{ kN/m} \quad \text{(E.34)}
\]
\[
q_{p,\text{up}} = -260 \cdot 52 = -13,520 \text{ kN/m} \quad \text{(E.35)}
\]

To have a vertical equilibrium, the connections between the sill and the soil retaining walls have to bear a load of at least \( \frac{13520 - 8840}{2} = 2,340 \text{ kN/m} \) each. When the piles are not taken into account this required strength of the connection is 5,500 kN/m. The resulting maximum bending moment at the clamped connections \( M_{\text{sill,connect,max}} = 47,320 \text{ kNm/m} \) when the pile load is not added, and minimum \( M_{\text{sill,connect,min}} = 31,550 \text{ kNm/m} \) when the piles are included as an average downward directed load. It will be expected that the worst-case maximum loads will be between these values.

### E.2.5 Wall design

Due to the relatively long required lifetime of the abutment walls, the big horizontal loads and small allowance of deformations, the choice is made to construct the walls out of reinforced concrete like diaphragm walls. These walls will be beared on the soil at more or less the same depth as the pile foundation. As presented in the right image of Figure [E.10](#), the wall is able to resist a bending moment around the wall bearing point \( D \), requiring a compression force from the slab halfway.

A second possible instability is due to rotation around the connection between the slab and the wall at a height \( MSL - 25 \text{ m} \).

#### Reinforcement

To determine the required reinforcement in the wall, the maximum required bending moment capacity needs to be checked. With the use of the displacement and occurring load associated with approach 1 in Table [E.6](#), the bending moment at the connection can be determined resulting in \( M_{Ed} = 83,400 \text{ kNm/m} \).

The bending moment capacity has to be bigger than the determined maximum occurring load. Due to the great dimensions of the wall, it is expected that a relatively big amount of reinforcement is expected to retain this load. Vrijling et al. (2015) gives for concrete class C45/55 a maximum bending moment ratio:

\[
\rho_{\text{max}} = \frac{A_s}{A_c} = 0.0305 \quad \text{(E.36)}
\]

In which \( A_s \) is the total cross-sectional area of the reinforcement and \( A_c \) is the total cross-sectional area of the wall, which will be considered per meter in horizontal direction so \( A_c = 1 \cdot t_{\text{wall}} = 3 \text{ m}^2/\text{m} \). The maximum reinforcement ratio \( \rho_{\text{max}} \) can get higher values for stronger types of concrete.

In this design, concrete C70/85 is used. The strength criterion \( M_{Rd} > M_{Ed} \), in combination with the formulas provided by Figure [E.11](#)(b), gives the following set of equations to determine the required amount of reinforcement steel.

\[
A_s f_{yd} \cdot 0.4t_{\text{wall}} + abx_u f_{cd} \cdot (0.5t_{\text{wall}} - \beta x_u) > M_{Ed} \quad \text{(E.37)}
\]

In which:
- \( f_{yd} \) Design yield strength of steel B500B: \( 435 \cdot 10^3 \text{ [kN/m}^2\text{]} \)
- \( f_{cd} \) Design compression strength of concrete C70/85: \( 46.7 \cdot 10^3 \text{ [kN/m}^2\text{]} \)
- \( t_{\text{wall}} \) Thickness of the diaphragm wall: \( 3.0 \text{ [m]} \)
- \( x_u \) Area factor, for C70/85: \( 0.62 \text{ [-]} \)
- \( b \) Width of the wall, will be considered per 1 m \( 1.0 \text{ [m]} \)
Length of concrete compression zone, yet unknown.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_u$</td>
<td></td>
<td>[m]</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$M_{Ed}$</td>
<td>Bending moment acting on the connection</td>
<td>[kNm/m]</td>
</tr>
</tbody>
</table>

Table E.6: Dimensions of the diaphragm walls, in combination with the occurring loads and resulting deformation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing depth</td>
<td>$t_{wall}$</td>
<td>[m]</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Effective length</td>
<td>$z_{eff}$</td>
<td>[m]</td>
<td>31</td>
<td>31</td>
</tr>
<tr>
<td>Youngs Modulus</td>
<td>$E_c$</td>
<td>[kN/m$^2$]</td>
<td>$38.5 \cdot 10^6$</td>
<td>$38.5 \cdot 10^6$</td>
</tr>
<tr>
<td>Moment of inertia</td>
<td>$I_y$</td>
<td>[m$^4$/m]</td>
<td>2.25</td>
<td>2.25</td>
</tr>
</tbody>
</table>

**Deformations**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design point load</td>
<td>$F_d$</td>
<td>[kN]</td>
<td>8,068</td>
<td>8,068</td>
</tr>
<tr>
<td>Point of action</td>
<td>$z_{F_d}$</td>
<td>[m]</td>
<td>10.33</td>
<td>10.33</td>
</tr>
<tr>
<td>Displacement at $z_{F_d}$</td>
<td>$w_{z,F_d}$</td>
<td>[m]</td>
<td>0.034</td>
<td>0.034</td>
</tr>
<tr>
<td>Rotation at $z_{F_d}$</td>
<td>$\theta_z$</td>
<td>[rad]</td>
<td>0.0005</td>
<td>0.0005</td>
</tr>
<tr>
<td>Displacement at MSL + 6m</td>
<td>$w_{z,top}$</td>
<td>[m]</td>
<td>0.137</td>
<td>0.137</td>
</tr>
</tbody>
</table>

**Bearing Capacity Check**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Self weight</td>
<td>$q_{sw,wall}$</td>
<td>[kN/m]</td>
<td>4,562</td>
<td>5,297</td>
</tr>
<tr>
<td>Tip Bearing</td>
<td>$F_{t,tip}$</td>
<td>[kN/m]</td>
<td>68,200</td>
<td>90,300</td>
</tr>
</tbody>
</table>

Figure E.11: Diaphragm wall: (a) modelling the loads to determine the horizontal displacements; (b) Internal load distribution. The wall strength is sufficient if $M_{Rd} > M_{Ed}$

The requirement of vertical load equilibrium results in $N_s = N_c + N_{sw}$, in which selfweight $N_{sw}$ at this point results from:
\[ N_{sw} = z_{eff} \cdot t_{wall} \cdot \gamma_c = 31 \cdot 3.0 \cdot 25 = 2,325 kN/m \] (E.38)

Where the specific weight of reinforced concrete \( \gamma_c = 25 kN/m^3 \), resulting in \( N_c = N_s - 2,325 kN/m \) and \( x_u = \frac{A_s \cdot f_{yd} - 2,325}{\alpha_{bf}} \). Rewriting equation [E.37] results in the required amount of reinforcement steel, which is according to the set of equations below lower than the maximum allowable reinforcement ratio.

\[ A_s f_{yd} \cdot 0.4 t_{wall} + \left( A_s f_{yd} - N_{sw} \right) \cdot \left( 0.5 t_{wall} - \beta \frac{A_s f_{yd} - N_{sw}}{\alpha_{bf} f_{yd}} \right) > M_{Ed} \] (E.39)

\[ 522 \cdot 10^3 A_s + (435 \cdot 10^3 A_s - 2,325) \cdot (1.53 - 5.27 A_s) > 83,400 kNm/m \] (E.40)

\[ A_s > 0.087 m^2 \] (E.41)

\[ \rho_s = 0.029 < \rho_{max} [-] \] (E.42)

When using a reinforcement ratio \( \rho_s = 0.03 \) the amount of reinforcement steel \( A_s = 0.090 m^2 \) or \( 90.0 \cdot 10^3 mm^2 \). According to Vrijling et al. (2015), the bar diameter \( \Phi \) of the primary reinforcement is usually between 6.0 and 50 mm, resulting in cross-sectional areas in the range of 30 to 1960 mm² per bar. The design of the reinforcement will be in the range of 300 bars to 50 bars per 1 m width and 3 m wall thickness.

E.2.6 Water inlet

Just behind each pillar, there will be a structure with the function of water inlet when the gate has to operate, and a water retention when the gate is stored and the dock needs to keep dry during calm weather conditions. This structure will be a classic Mitre gate (See Figure 4.1-(b) in Chapter 3), consisting of two straight steel doors which are connected to a big hinge at the retaining walls on one side and when closed connected to each other at the center of the dry dock entrance.

Each door will have a door height which is the same as the gate height, stretching from \( MSL - 24 m \) to \( +13 m \), resulting in a total height of 37 m. When the dock is closed-off, the gates connect to each other with some angle to have a better horizontal load retaining capacity. The sill width at the location of the water inlet is minimum, giving \( W_{sill,inlet} = W_{gate} + 2 = 20.0 m \). The angle of the doors with \( y \) is 1:2 resulting in 11.2 m wide mitre doors, as presented in Figure E.12-(b). When the gate is closed the two mitre doors are partly opened, closing off the dock by connecting to the gate again.

The two mitre doors will be constructed out of steel to enable two slender doors that do not require a lot of space. The load distribution is presented in Figure E.12-(a), where the maximum loads can be found for the situation with the biggest hydraulic head, which is of course when the dock is dry and the water levels in the channel are somewhat higher, without operation requirements. The effective length at this moment is about 25 m: \( MSL - 24 m \) to \( \pm MSL + 1 m \).

When a hurricane has been forecasted, and the concrete door barrier needs to operate, the dry dock needs to be filled with water. This will be established by opening inlets present at the bottom of each mitre door. These inlets are so-called valve gates, which open or close by a turning helix. The openings contain a steel grid to prevent the inlet of organisms, organic material and debrees. The size of the openenings greatly influence the inlet duration, which should be minimized while the loads considering the inlet discharge may not be damaging large.
Figure E.12: Abutment head of the dry dock: (a) Normal conditions, the mitre gates are closed; (b) Storm conditions, the concrete door is closed and the mitre gates are connected to its sides.
Appendix F

Composite barrier design

F.1 Gate

F.1.1 Concept

The gate is a composite triangular structure that moves over a concrete sill. This concept is investigated in this section by checking the load distribution for different types of modelling, varying the number of supports and the cross sections of the steel tubes. The concept can be seen in Figure F.1.

Figure F.1: Cross sections of the steel truss structure: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay.

F.1.2 Loads

With the use of the MatrixFrame computer software, the transmission of the design loads from concrete slab to the steel H-girders to the steel cross sections has been simulated. These simulations, with input the design load combination, have resulted in the final gate design by giving the required cross sections of the different elements. In this Section first the external design is discussed after which the internal design is addressed as well.
F.1.3 External design

The truss will be designed in a triangular shape, consisting of multiple small triangles as presented in Figure F.2. The horizontal forces will be transmitted to the foundation by the presented triangular supporting cross-sections, which will be present every 50 m resulting in 8 spans and 9 steel supporting sections. The height of the truss will be 33.0 m, which is divided into 3 different layers of 11.0 m each. The maximum width at the bottom layer of the truss is 49.5 m, which is divided into 3 different layers as well having a width of 16.5 m each.

![Figure F.2: Cross sections of the steel truss structure with only 2 supports: (a) Structural drawing of the truss structure with dimensions; (b) Sketch of the situation with a 1/1000 years storm surge level in the bay.](image)

F.1.4 Internal design

![Figure F.3: Occuring stresses in the steel tubes for a 1/1000 years storm surge level, wind speeds and wave impacts for designs 1 and 2: (a) Steel main tubes on the outsides of the sections with a diameter of 1.50 m and a steel thickness of 0.15 m and 4 supports; and (b) Steel main tubes with a diameter of 1.40 m and a steel thickness of 0.10 m for all sections, and secondary tubes of 1.22 m and 0.025 m thickness, supported on two outer beams at the bottom side. See Tables F.1 and F.2 for the actual values of the occurring stresses and the resulting strain for each beam for the two designs.](image)
Table F.1: Occuring stresses in the steel tubes and resulting strains. The table shows that all stresses are smaller than the steel yield strength \( f_y = 355 \text{ N/mm}^2 \), so yielding will not occur for loads associated with a 1/1000 years storm.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Length ( L )</th>
<th>( A_{\text{tube}} )</th>
<th>Normal Force ( N_y )</th>
<th>Stress ( \sigma )</th>
<th>Strain ( \epsilon )</th>
<th>( \Delta L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>12.3 m</td>
<td>0.636 m(^2)</td>
<td>-12234 kN</td>
<td>-19.2 N/mm(^2)</td>
<td>-0.01 %</td>
<td>-1.1 mm</td>
</tr>
<tr>
<td>S2</td>
<td>12.3 m</td>
<td>0.636 m(^2)</td>
<td>-34804 kN</td>
<td>-54.7 N/mm(^2)</td>
<td>-0.03 %</td>
<td>-3.2 mm</td>
</tr>
<tr>
<td>S3</td>
<td>12.3 m</td>
<td>0.636 m(^2)</td>
<td>-12284 kN</td>
<td>-19.3 N/mm(^2)</td>
<td>-0.01 %</td>
<td>-1.1 mm</td>
</tr>
<tr>
<td>S4</td>
<td>16.5 m</td>
<td>0.636 m(^2)</td>
<td>-48262 kN</td>
<td>-75.9 N/mm(^2)</td>
<td>-0.04 %</td>
<td>-6.0 mm</td>
</tr>
<tr>
<td>S5</td>
<td>16.5 m</td>
<td>0.094 m(^2)</td>
<td>-5142 kN</td>
<td>-54.8 N/mm(^2)</td>
<td>-0.03 %</td>
<td>-4.3 mm</td>
</tr>
<tr>
<td>S6</td>
<td>16.5 m</td>
<td>0.094 m(^2)</td>
<td>-26559 kN</td>
<td>-283.0 N/mm(^2)</td>
<td>-0.13 %</td>
<td>-22.2 mm</td>
</tr>
<tr>
<td>S7</td>
<td>15.6 m</td>
<td>0.636 m(^2)</td>
<td>-88983 kN</td>
<td>-139.9 N/mm(^2)</td>
<td>-0.07 %</td>
<td>-10.4 mm</td>
</tr>
<tr>
<td>S8</td>
<td>15.6 m</td>
<td>0.094 m(^2)</td>
<td>-31947 kN</td>
<td>-340.4 N/mm(^2)</td>
<td>-0.16 %</td>
<td>-25.3 mm</td>
</tr>
<tr>
<td>S9</td>
<td>15.6 m</td>
<td>0.636 m(^2)</td>
<td>-14678 kN</td>
<td>-23.1 N/mm(^2)</td>
<td>-0.01 %</td>
<td>-1.7 mm</td>
</tr>
<tr>
<td>S10</td>
<td>12.3 m</td>
<td>0.094 m(^2)</td>
<td>7759 kN</td>
<td>82.7 N/mm(^2)</td>
<td>0.04 %</td>
<td>4.8 mm</td>
</tr>
<tr>
<td>S11</td>
<td>12.3 m</td>
<td>0.094 m(^2)</td>
<td>19796 kN</td>
<td>210.9 N/mm(^2)</td>
<td>0.10 %</td>
<td>12.4 mm</td>
</tr>
<tr>
<td>S12</td>
<td>16.5 m</td>
<td>0.636 m(^2)</td>
<td>-114652 kN</td>
<td>-180.2 N/mm(^2)</td>
<td>-0.09 %</td>
<td>-14.2 mm</td>
</tr>
<tr>
<td>S13</td>
<td>16.5 m</td>
<td>0.094 m(^2)</td>
<td>-21292 kN</td>
<td>-226.9 N/mm(^2)</td>
<td>-0.11 %</td>
<td>-17.8 mm</td>
</tr>
<tr>
<td>S14</td>
<td>15.6 m</td>
<td>0.094 m(^2)</td>
<td>-16721 kN</td>
<td>-178.2 N/mm(^2)</td>
<td>-0.08 %</td>
<td>-13.2 mm</td>
</tr>
<tr>
<td>S15</td>
<td>15.6 m</td>
<td>0.094 m(^2)</td>
<td>-39718 kN</td>
<td>-62.4 N/mm(^2)</td>
<td>-0.03 %</td>
<td>-4.6 mm</td>
</tr>
<tr>
<td>S16</td>
<td>12.3 m</td>
<td>0.094 m(^2)</td>
<td>15870 kN</td>
<td>169.1 N/mm(^2)</td>
<td>0.08 %</td>
<td>9.9 mm</td>
</tr>
<tr>
<td>S17</td>
<td>16.5 m</td>
<td>0.636 m(^2)</td>
<td>-133574 kN</td>
<td>-210.0 N/mm(^2)</td>
<td>-0.10 %</td>
<td>-16.5 mm</td>
</tr>
<tr>
<td>S18</td>
<td>15.6 m</td>
<td>0.636 m(^2)</td>
<td>-59792 kN</td>
<td>-94.0 N/mm(^2)</td>
<td>-0.04 %</td>
<td>-7.0 mm</td>
</tr>
</tbody>
</table>

Table F.2: Occuring stresses in the steel tubes and resulting strains for design 2. The table shows that all stresses are smaller than the steel yield strength \( f_y = 355 \text{ N/mm}^2 \), so yielding will not occur for loads associated with a 1/1000 years storm.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Length ( L )</th>
<th>( A_{\text{tube}} )</th>
<th>Normal Force ( N_y )</th>
<th>Stress ( \sigma )</th>
<th>Strain ( \epsilon )</th>
<th>( \Delta L )</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>-53076 kN</td>
<td>130 N/mm(^2)</td>
<td>0.06 %</td>
<td>7.6 mm</td>
</tr>
<tr>
<td>S2</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>-33368 kN</td>
<td>82 N/mm(^2)</td>
<td>0.04 %</td>
<td>4.8 mm</td>
</tr>
<tr>
<td>S3</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>-12284 kN</td>
<td>30 N/mm(^2)</td>
<td>0.01 %</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>S4</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-29997 kN</td>
<td>73 N/mm(^2)</td>
<td>0.03 %</td>
<td>5.7 mm</td>
</tr>
<tr>
<td>S5</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-61865 kN</td>
<td>151 N/mm(^2)</td>
<td>0.07 %</td>
<td>11.9 mm</td>
</tr>
<tr>
<td>S6</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-24632 kN</td>
<td>60 N/mm(^2)</td>
<td>0.03 %</td>
<td>4.7 mm</td>
</tr>
<tr>
<td>S7</td>
<td>15.6 m</td>
<td>0.408 m(^2)</td>
<td>-35505 kN</td>
<td>87 N/mm(^2)</td>
<td>0.04 %</td>
<td>6.5 mm</td>
</tr>
<tr>
<td>S8</td>
<td>15.6 m</td>
<td>0.408 m(^2)</td>
<td>-33764 kN</td>
<td>83 N/mm(^2)</td>
<td>0.04 %</td>
<td>6.2 mm</td>
</tr>
<tr>
<td>S9</td>
<td>15.6 m</td>
<td>0.408 m(^2)</td>
<td>-14678 kN</td>
<td>36 N/mm(^2)</td>
<td>0.02 %</td>
<td>2.7 mm</td>
</tr>
<tr>
<td>S10</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>28069 kN</td>
<td>69 N/mm(^2)</td>
<td>0.03 %</td>
<td>4.0 mm</td>
</tr>
<tr>
<td>S11</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>18360 kN</td>
<td>45 N/mm(^2)</td>
<td>0.02 %</td>
<td>2.6 mm</td>
</tr>
<tr>
<td>S12</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-67655 kN</td>
<td>166 N/mm(^2)</td>
<td>0.08 %</td>
<td>13.0 mm</td>
</tr>
<tr>
<td>S13</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-48839 kN</td>
<td>120 N/mm(^2)</td>
<td>0.06 %</td>
<td>9.4 mm</td>
</tr>
<tr>
<td>S14</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-46045 kN</td>
<td>113 N/mm(^2)</td>
<td>0.05 %</td>
<td>8.4 mm</td>
</tr>
<tr>
<td>S15</td>
<td>15.6 m</td>
<td>0.408 m(^2)</td>
<td>-37901 kN</td>
<td>93 N/mm(^2)</td>
<td>0.04 %</td>
<td>6.9 mm</td>
</tr>
<tr>
<td>S16</td>
<td>12.3 m</td>
<td>0.408 m(^2)</td>
<td>36402 kN</td>
<td>89 N/mm(^2)</td>
<td>0.04 %</td>
<td>5.2 mm</td>
</tr>
<tr>
<td>S17</td>
<td>16.5 m</td>
<td>0.408 m(^2)</td>
<td>-116494 kN</td>
<td>285 N/mm(^2)</td>
<td>0.14 %</td>
<td>22.4 mm</td>
</tr>
<tr>
<td>S18</td>
<td>15.6 m</td>
<td>0.408 m(^2)</td>
<td>-83947 kN</td>
<td>206 N/mm(^2)</td>
<td>0.10 %</td>
<td>15.3 mm</td>
</tr>
</tbody>
</table>

APPENDIX F. COMPOSITE BARRIER DESIGN
The gate meets the requirements regarding strength and stability. However, for a gate of these dimensions it is more or less impossible to have it rested on the four bottom beans at all locations due to small fluctuations in the sill surface. Therefore a modelling with only 2 supports might be a more realistic approach, resulting in an increase of the loads in some tubes. In this case the load will be bigger than the yield strength, which should be avoided by using different steel tubes. In the final design, the sections (beams in y direction) will be constructed out of steel tubes with a diameter of 1.4 m and a thickness of 0.10 m, the beams in x direction will have the original diameter of 1.22 m and a thickness of 0.025 m.

F.2 Moving mechanism

F.2.1 Floatability

The floatability of the gate depends on its selfweight. The selfweight of the concrete slab is given by the following equation:

\[ G_{slab} = V_{slab} \cdot \rho_c \cdot g = L \cdot b \cdot t \cdot \rho_c \cdot g = 400 \cdot 36.9 \cdot 0.50 \cdot 2.500 \cdot 9.81 = 181 \cdot 10^3 \text{kN} \]  

(F.1)

The selfweight of the steel truss structure follows from the weight of all the steel tubes, given by the following equation:

\[ G_{truss} = (A_{tube1} \cdot L_{tube1} + A_{tube2} \cdot L_{tube2}) \cdot \rho_x \cdot g = (0.408 \cdot 2395 + 0.094 \cdot 6093) \cdot 8000 \cdot 9.81 = 121.6 \cdot 10^3 \text{kN} \]  

(F.2)

The selfweight of the total barrier will than be about 300 \( \cdot \) 10^3 kN. To have a floating structure, this self weight has to be smaller than the weight of water for the total volume of the barrier elements. The total volume of all these elements is:

\[ V_{gate} = V_{slab} + V_{tube1} + V_{tube2} = 7380 + \pi 0.7^2 \cdot 2395 + \pi 0.61^2 \cdot 6093 = 18190 \text{m}^3 \]  

(F.3)

A volume of 18190 m^3 water has a self weight of 183 \( \cdot \) 10^3 kN < 300 \( \cdot \) 10^3 kN, so the gate won’t float by itself. The submerged weight of the steel truss is determined in equation (F.4) together with the weight of the water column above the foundation (Equation F.5) this contributes to the vertical load on the foundation during operation.

\[ G_{truss} = (A_{tube1} \cdot L_{tube1} + A_{tube2} \cdot L_{tube2}) \cdot (\rho_s - \rho_w) \cdot g = (0.408 \cdot 2395 + 0.094 \cdot 6093) \frac{8000 - 1025}{1000} \cdot 9.81 = 106 \cdot 10^3 \text{kN} \]  

(F.4)

\[ G_{water} = (b_{river} \cdot b_{foundation} \cdot L_{x} - (A_{tube1} \cdot L_{tube1} + A_{tube2} \cdot L_{tube2})) \cdot \rho_w \cdot g = (22 \cdot 33 \cdot 400 - 1550) \frac{1025}{1000} \cdot 9.81 = 2.90 \cdot 10^3 \text{kN} \]  

(F.5)

The loads \( G_{truss} \) and \( G_{water} \) are the totals for the complete barrier. Distributed over the length \( x \) these loads are \( q_{truss} = 265 \text{kN/m} \) and \( q_{water} = 7, 250 \text{kN/m} \) respectively. The weigh of the truss will be transmitted to the foundation by two point loads, which are for each section of \( x = 50 \text{m} \) about 13, 250 kN together or \( F_{y, truss,1} = 8.0 \cdot 10^3 \text{kN} \) and \( F_{y, truss,2} = 5.25 \cdot 10^3 \text{kN} \). The water load is distributed over the total foundation area by \( q_{water,f} = 220 \text{kN/m}^2 \). See Appendix G for the design calculations of the foundation.

Hydrofeet will not be able for this type of gate. In case of floating elements, these elements need to have a volume of 12 \( \cdot \) 10^3 m^3 to have a fully floating structure. Over the total 400 m length of the structure, this means that the floating elements need to have a cross section of 30 m^2. This can be
managed by a foam block of $3 \cdot 5 \ m^2$ on each side of the barrier, or an inflatable bag with a diameter of 4.36 m, over the full length of the barrier, as presented in Figure F.4 in the correct scale.

Since it will take a lot of time to mount foam blocks each time the gate needs to close, the inflatable bags are preferred. In this case the inflatable bags are attached to two hoses at the back end which inject air when the gate needs to move. When in position then hoses stop injecting air after which the gate will slowly submerge to the channel bottom.

![Figure F.4: Use of floaters: (a) foam blocks; (b) inflatable bags.](image)

**F.2.2 Motion system**

It is decided not to have a floating gate. The used driving system is by rack and pinion, pushing the gate over the sill by rotation at the dock. The gate will subsequently slide over the surface of the sill.

**F.3 Docks**

**F.3.1 Preliminary design of roof structure: Concrete hollow core slabs.**

On top of the dry dock a prefabricated prestressed hollow core slab will be installed, using straight tendons of prestressed steel Y1860S7 and $\sigma_{p0} = 1395 \ N/mm^2$. The tendons will consist of 7 strands $A_{\text{str}and} = 140 \ mm^2$. The hollow spaces in the slab will have a circular shape $D = 2.40 \ m$, and can be used as space for cables required for installations and systems relevant for maintenance purposes. The concrete strength class aimed at is C70/85, but it is assumed that the class is only C50/60 at the time when the prestressing is applied to the concrete. Associated to this process, the time dependent Young’s modulus of concrete class C70/85 having reached a class C50/60 strength can be defined using the following equation: (Bamforth et al., 2008; Fennis, 2012)

$$E_{cm} = \left( \frac{f_{cm}(t)}{f_{cm}} \right)^{0.3} \left( \frac{50 + 8}{70 + 8} \right)^{0.3} 38,500 \approx 35 \cdot 10^3 N/mm^2$$  \hspace{1cm} (F.6)

The length of a top slab will be the total width of the dry dock $W_{\text{dock}} = 60 \ m$, plus some additional width to connect to the diaphragm walls resulting in a total of 65 m. The full 450 m long dock will be covered width these slabs, where the width of each slab will be 15 m resulting in a total number of 30 prefabricated slabs. The cross-section of a part of the top-slab is given in Figure F.5.
Figure F.5: Cross-section of one prefabricated hollow core slab which is installed on top of the dry dock.

Geometrical data:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c$</td>
<td>Effective concrete area per slab</td>
</tr>
<tr>
<td>$I_c$</td>
<td>Moment of inertia of the concrete section</td>
</tr>
<tr>
<td>$A_p$</td>
<td>Total amount of prestressing steel</td>
</tr>
<tr>
<td>$\varepsilon_p$</td>
<td>Excentricity from prestressing tendons to neutral axis</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Modulus of elasticity of the concrete</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modulus of elasticity of the prestressing steel</td>
</tr>
</tbody>
</table>

[Fennis (2012)] gives design rules and steps for prestressing with pre-tensioned steel. The prestress force is transferred to the concrete element by a small shortening of the element, resulting in an axial force and a bending moment. Before the prestressing force is transferred to the beam, the stress in the steel is $\sigma_{p,max} = P_{max}/A_p$, but due to shortening of the element this force gets reduced by $-\Delta P_{el}$. The net prestressing force that gets transferred to the concrete than results in $P_{p,c} = P_{max} - \Delta P_{el}$, which can be determined using the shortening of the element $\Delta L_p$.

$$\Delta L_p = \frac{\Delta P_{el} \Delta x}{E_p A_p} \rightarrow \Delta P_{el} = \frac{\alpha_e \rho_p f}{1 + \alpha_e \rho_p f} P_{max} \tag{F.7}\$$

Where:

$$\alpha_e = \frac{E_s}{E_c} = 5.54 \quad \rho_p = \frac{A_p}{A_c} = 0.00219 \quad f = 1 + \frac{A_p \varepsilon_p^2}{I_c} = 2.71 \tag{F.8}\$$

Resulting in the following forces:

$$\Delta P_{el} = \frac{\alpha_e \rho_p f}{1 + \alpha_e \rho_p f} P_{max} = 0.032P_{max} = 2,248 kN \quad \Delta \sigma_{p,el} = 0.032 \sigma_{p,max} = 45.9 N/mm^2 \tag{F.9}\$$

$$P_{max} = A_p \sigma_{p,max} = 0.049 \sigma_{p,max} = 70,600 kN \quad \sigma_{p,max} = \frac{\sigma_{p0}}{1 - 0.032} = 1441 N/mm^2 \tag{F.10}\$$

The maximum compressive stress in the pre-compressed tensile zone (bottom fibre), which is assumed to be at a distance $h/2$ from the centroidal axis, may not be larger than $0.70 f_{ck}(t) = 0.70 \cdot 50 = 35 \ N/mm^2$ [Fennis 2012]. It follows from the following equation:

$$\sigma_{cb} = (P_{max} - \Delta P_{el}) \left( \frac{1}{A_c} + \frac{\varepsilon_p \cdot h/2}{I_c} \right) \tag{F.11}\$$

Which will result in a compressive stress $\sigma_{cb} = -8.66 \ N/mm^2$, this does not exceed the compression strength and thus meets the requirements.
F.3.2 Final design of the roof structure

Main girders

According to Wagemans et al. (2004) the 60 m wide dock can best be covered by either a truss frame or multiple truss girders, consisting of steel S355. The final design of the roof structure of the dry dock will therefore consist of multiple steel N-girders on which concrete hollow core slabs are instllled. The N-girders will have a length $L_{\text{girder}} = 60$ m and a total height $h_{\text{girder}} = 3.5$ m. The girder is constructed out of HE500B steel profiles, which are installed as presented in Figure F.6-(a).

![Figure F.6: Main girder for the dry dock roof structure: (a) Dimensions of the N-girders [m]; (b) Design stress distribution [N/mm²]; (c) Maximum deflections [m].](image)

The design load combination follows from the design variable load and the dead load resulting from the installed systems and the self weight of the girders, concrete slabs and the soil cover on top of the slabs. This load gives the design stress distribution as presented in Figure F.6-(b). The maximum deflection of the roof structure is presented in Figure F.6-(c). The maximum deflection $\delta_{\text{max}} = 0.200$ m and meets the serviceability limit, presented in Equation F.12 (Wagemans et al., 2004).

$$\delta_{\text{max}} < 0.004L$$  \hspace{1cm} (F.12)

Hollow core slabs

The hollow core slabs are prefabricated and prestressed, constructed out of concrete C45/55 and prestressed by linear tendons of pre-tensioned steel Y1860S7. The slabs have a length $L_{\text{slabs}} = 8.0$ m and a width $W_{\text{slab}} = 1.2$ m. A sketch of the dry dock with the final roof construction is presented in Figure F.7.
F.4 Architecture

The architectural design of this alternative barrier design is aiming at adding value by not only being a barrier but also a "lifestyle attraction". This is, among others, achieved by additional structures: there is a park on top of it, an exhibition and a restaurant inside. All machinery can be installed well visible to the public. Reference structures providing these qualities too are the Marina Bay Barrage (Singapore) and the Sydney Opera House (Australia), both presented in Figure F.8.

Figure F.8: Reference structures, used as an example for the Alternative barrier design: (a) Marina Barrage Singapore [PUB 2015]; (b) Sydney Opera House [House 2015].
Appendix G

Foundations

G.1 Introduction

A civil structure always needs a foundation, of which the main function is to support the structure by counteracting uneven settlements that may lead to failure of the structure during its lifetime. There are several ways to support a structure, like soil improvements, steel plates or piles, which are available in a great variety.

At the location of the Houston Ship Channel Barrier, the soil is very weak. As presented in Section 2.4 a 28-38 m thick layer of clay is present at the channel bottom at MSL -22 m that stretches to a depth of MSL -50 m to -60 m where dense sands can be found. The clay layer has a poor bearing capacity, but the dense sands should be a good layer to construct the foundation on. A foundation type that fits best for a situation like this, is the closed bottom steel pile foundation. This type will be designed in this Appendix, starting with a general overview of pile foundations in Section G.2 and followed by the designs of the foundation for the Concrete and Composite Alternatives in Sections G.3 and G.4 respectively. This Appendix concludes with a seepage check in Section G.5 Geotechnical requirements for the Netherlands are given in the NEN 9997 (Geotechniek 2014), these guidelines are used for the design of the foundation of the HSC Barrier.

G.2 General pile foundations

G.2.1 Types

According to Abede and Smith (1994), it is very likely that heavy constructions like a storm surge barrier have to be built on piles. The shallow soil will not provide enough bearing capacity and therefore the horizontal and vertical loads have to be transmitted to the more solid subsoils. There are several types of pile foundations, which can be classified with respect to load transmission and functional behaviour or with respect to the type of material. Types of load transmission are:

- End bearing piles
  Most of the carrying capacity is derived from the penetration resistance of the soil at the toe of the pile or the point bearing capacity of the subsoil which make the piles behave as columns. The load is transmitted to the soil by both friction and cohesion.

- Cohesion piles
  The carrying capacity is mainly dependent on the skin friction between the pile shaft and the cohesive soil. During the process of driving this kind of piles closely to each other greatly reduces the porosity and compressibility of the soil, which makes the soil become moulded and less strong. This type of piles is also known as compaction piles.
Friction piles

The load gets transferred to the ground by skin friction, but for these piles the soil is not compacted by the pile driving process. This type of pile foundations is also known as floating pile foundations.

With respect to the type of material a classification of piles can be made as well. Possible materials to use are timber, concrete and steel which have all their characteristics and limitations. A foundation consisting of timber piles for example, is the cheapest solution but this is not favourable in cases below ground water level since the water will damaged the piles after some time. Concrete piles may be in these situations a better option, but they will be less resistant against tensile forces, which may be an issue in the case of the HSC Barrier. Therefore a foundation of consisting of steel piles will probably be the best solution: They are suitable for handling and driving in long lengths; Penetration in the soil goes easy due to the combination of a relatively small cross-sectional area and a high strengt; and construction is relatively easy. The pile surface has to be protected against corrosion by a coating. (Abede and Smith, 1994)

G.2.2 Type of piles used for the foundation Houston Ship Channel Barrier

For steel piles it is possible to install the piles by either driving or boring, which may be favourable when soil vibrations are not allowed due to nearby structures. At the Houston Ship Channel however, it is expected that vibrations due to driving will not be an issue and since bored piles are much more expensive, it is decided to use steel close-ended piles which are installed by driving.

G.2.3 Pile arrangements

The easiest way to install piles is by vertical driving, however, for retaining structures the foundation has to transmit a horizontal load to the subsoil as well. The transmission of horizontal forces can only be done by the installation of inclined piles. As presented in Appendices E and F the positions where the horizontal loads are transmitted are clearly pointed out, so obviously the inclined piles are required close to these positions. In order to support the remaining parts of the sill, some additional vertical declined piles may be installed as well where needed.

G.2.4 Typical failure mechanisms

Geotechniek (2014) gives the following limit states or failure mechanisms, which have to be considered during the design stage:

1. Loss of overall stability;
2. Failure by exceeding the carrying capacity of the pile foundation;
3. Rise or insufficient tensile resistance of the pile foundation;
4. Failure of the ground through a horizontal load on the pile foundation;
5. Structural failure of the pile in compression, tension, bending, buckling or shearing;
6. Simultaneous failure of the substrate and the pile foundation;
7. Simultaneous failure of the substrate and of the structure;
8. Excessive settlement;
9. Excessive rise;
10. Exceptional horizontal displacement;
11. Unacceptable vibration.
G.2.5 Pile bearing capacity

The bearing capacity of the used piles is the sum of a number of different components, which are dependent to either the characteristics of the piles or the local soil characteristics. It can be determined as follows:  

\[ F_{t,\text{max}} = F_{t,\text{tip}} + F_{t,\text{skin,+}} - F_{t,\text{skin,-}} - F_{t,dw} \text{kN} \]  

In which \( F_{t,\text{tip}} \) is the bearing capacity of the tip of the pile, \( F_{t,\text{skin,+}} \) and \( F_{t,\text{skin,-}} \) are the positive and negative skin friction, and \( F_{t,dw} \) is the dead weight of the pile. According to Sin (2003), one may neglect the negative skin friction for steel close-ended tubular piles when founded on a sand layer, which is the case in the design of the foundation for the HSC Barrier. The remaining components will be introduced in this section after which the required pile dimensions and pile spacing will be determined.

Pile tip bearing capacity

The tip bearing capacity of the pile can be determined using equation G.2:

\[ F_{t,\text{tip}} = A_t \cdot P_{t,\text{tip}} \]  

In which \( A_t \) is the cross sectional area of the pile tip as a function of the pile diameter:

\[ A_t = \frac{1}{4} \phi_t^2 \text{m}^2 \]  

The maximum soil pressure under the pile tip is expressed by \( P_{t,\text{tip}} \) and can be determined using the formula provided by Brinch Hansen (1970):

\[ P_{t,\text{tip}} = \sigma_{su,i} + N_c \cdot s_c \cdot i_c + \sigma_{\phi,i} \cdot N_q \cdot s_q \cdot i_q + 0.5 \cdot B_{eff} \rho_v \cdot N_{\gamma} \cdot s_{\gamma} \cdot i_{\gamma} \text{kN/m}^2 \]  

In which:

- \( \sigma_{su,i} \) [kN/m³] Undrained shear stress of clay soil layer
- \( \sigma_{\phi,i} \) [kN/m³] Initial vertical effective stress
- \( \rho_v \) [kN/m³] Effective volumetric weight of the soil below construction depth
- \( B_{eff} \) [m] Width of effective foundation area
- \( N_c \) [-] Bearing capacity factor for cohesion
- \( N_q \) [-] Bearing capacity factor for surcharge including soil coverage
- \( N_{\gamma} \) [-] Bearing capacity factor of soil below the foundation
- \( s_c \) [-] Shape factor for cohesion
- \( s_q \) [-] Shape factor for surcharge including soil coverage
- \( s_{\gamma} \) [-] Shape factor of soil below the foundation
- \( i_c \) [-] Inclination factor for cohesion
- \( i_q \) [-] Inclination factor for surcharge including soil coverage
- \( i_{\gamma} \) [-] Inclination factor of soil below the foundation

The factors for the bearing capacity \( (N_c, N_q, N_{\gamma}) \), the shape \( (s_c, s_q, s_{\gamma}) \) and the inclination \( (i_c, i_q, i_{\gamma}) \) for undrained soil conditions are given by the following set of equations:

\[ N_c = (N_q - 1) \cot \phi_i \]  

\[ N_q = \cot \phi_i + \frac{1}{\sin \phi_i} \cdot \frac{\tan \phi_i}{1 + \sin \phi_i} \]  

\[ N_{\gamma} = 2 \cdot (N_q - 1) \tan \phi_i \]  

\[ s_c = 1 + 0.2 \frac{B_{eff}}{L_{eff}} \]  

\[ s_q = 1 + \frac{B_{eff}}{L_{eff}} \cdot \sin \phi_i \]  

\[ s_{\gamma} = 1 - 0.3 \cdot \frac{B_{eff}}{L_{eff}} \]  

\[ i_c = 0.5 \left( 1 + \frac{1}{\sqrt{1 - \frac{F_{H,\text{tot}}}{A_{eff} \sigma_{su,i}}} \cdot \cot \phi_i} \right) \]  

\[ i_q = \frac{F_{H,\text{tot}}}{F_{V,\text{tot}} \cdot \sigma_{su,i} \cdot \cot \phi_i} \]  

\[ i_{\gamma} = i_q \]
In which:  
\[ \varphi_i \quad [^\circ] \] Angle of internal friction soil layer  
\[ L_{\text{eff}} \quad [m] \] Length of effective foundation area  
\[ A_{\text{eff}} \quad [m^2] \] Area of effective foundation area  
\[ F_{H,\text{tot}} \quad [kN] \] Total horizontal load  
\[ F_{V,\text{tot}} \quad [kN] \] Total vertical load

For circular steel tubular piles:

\[ B_{\text{eff}} = L_{\text{eff}} = \frac{B_{\text{eff}}}{L_{\text{eff}}} = 1 \quad (G.7) \]
\[ A_{\text{eff}} = \frac{\pi}{4} \phi_i^2 \quad (G.8) \]

According to Vrijling et al. (2015) the capacity of the soil below the foundation is negligible compared to the surcharge including soil coverage for steel tubular pile foundations. Combined with the fact that the steel tubular piles are founded on a non-cohesive sand layer, equation \( G.3 \) reduces to:

\[ P_{t,\text{tip}} = \sigma_{\varphi,i,i} \cdot N_q \cdot s_q \cdot i_q \quad (G.9) \]

Now the design values for \( \sigma_{\varphi,i,i} \), \( N_q \) and \( s_q \) can be computed using the angle of internal friction for the bearing sand layer, which is \( \varphi_s = 40^\circ \). This gives the design values presented in Table G.1 for bearing layer depths of 64 m and 54 m, assuming that the piles will penetrate 4 m into the bearing sand layer. The value for the inclination factor \( i_q \) depends on the loads per pile and the pile dimensions, which will be determined in the next sections.

<table>
<thead>
<tr>
<th>Depth of pile tip</th>
<th>( \sigma_{\varphi,i,i} ) [kN/m²]</th>
<th>( N_q ) [-]</th>
<th>( s_q ) [-]</th>
<th>( P_{t,\text{tip}} ) [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSL -54 m</td>
<td>236</td>
<td>64.2</td>
<td>1.64</td>
<td>24.85 \cdot 10^4 \cdot i_q</td>
</tr>
<tr>
<td>MSL -64 m</td>
<td>306</td>
<td>64.2</td>
<td>1.64</td>
<td>32.23 \cdot 10^4 \cdot i_q</td>
</tr>
</tbody>
</table>

**Pile skin friction**

The friction between the surface of the steel tubular pile and the soil contributes to the bearing capacity of the pile foundation. This skin friction can be either negative or positive, where negative skin friction (drag) decreases the pile bearing capacity and positive skin friction increases the pile bearing capacity. As mentioned before, the negative skin friction can be neglected for steel close-ended tubular piles (Sin, 2003). This makes that only the positive skin friction contributes to the total wall skin friction, which can be computed using the following formula (Hussin, 2006):

\[ F_{t,\text{skin,}+} = \pi \cdot \phi_t \cdot h_{s,i} \cdot P_{t,\text{skin}} \quad (G.10) \]

In which:
\[ \phi_t \quad [m] \] Diameter of the steel tubular pile
\[ h_{s,i} \quad [m] \] Thickness of soil layer \( i \)
\[ P_{t,\text{skin}} \quad [kN/m^2] \] The skin friction pressure

The skin friction pressure depends on the type of soil: cohesive or non-cohesive, as presented in Equations \( G.11 \) and \( G.12 \) respectively (API, 2000).
$P_{\text{skin,c}} = \alpha_s \cdot \sigma_{\text{su},i}$ \quad (G.11)

$P_{\text{skin,s}} = K_n \cdot \sigma'_{v,i} \cdot \tan(2/3 \cdot \varphi_i)$ \quad (G.12)

In which:

- $\alpha_s \quad [-]$ Adhesion factor
- $\sigma_{\text{su},i} \quad [kN/m^2]$ Undrained shear strength
- $K_n \quad [-]$ Coefficient of neutral soil pressure
- $\sigma'_{v,i} \quad [kN/m^2]$ Initial effective vertical stress in the middle of soil layer $i$

The undrained shear strength for stiff clay has already been introduced in Chapter 2 and presented in Figure 5.3 by $\sigma_{\text{su},i} = 36 \; kN/m^2$. The initial effective vertical stress in the middle of the dense sand layer depends on the depth and thus the location, as determined in Table G.8. API (2000) gives for the adhesion factor the following two equations:

$$\alpha_{s,i} = 0.5 \left( \frac{\sigma_{\text{su},i}}{\sigma'_{v,i}} \right)^{-0.5} \quad \text{for} \quad \frac{\sigma_{\text{su},i}}{\sigma'_{v,i}} \leq 1.0$$ \quad (G.13)

$$\alpha_{s,i} = 0.5 \left( \frac{\sigma_{\text{su},i}}{\sigma'_{v,i}} \right)^{-0.25} \quad \text{for} \quad \frac{\sigma_{\text{su},i}}{\sigma'_{v,i}} \geq 1.0$$ \quad (G.14)

In the situation of the HSC Barrier the effective vertical stress is always greater than the undrained shear stress due to the great depths, so the first equation has to be used for the determination of the adhesion factor $\alpha_s$. According to Hussin (2006), the coefficient of neutral soil pressure $K_n$ for driven piles can be determined using the equation G.15, which gives us the constant $K_n = 0.5$. The design values for the different parameters and the resulting value for the skin pressure is given in Table G.8:

$$K_n = 1.4 \cdot (1 - \sin \varphi_i) = 1.4 \cdot (1 - \sin(40)) = 0.5 \quad (G.15)$$

Pile deadweight

The deadweight of the piles has a negative effect on the bearing capacity, since it causes an additional downward directed load. The deadweight can easily be computed with the following computations, where equations G.16 and G.17 represent the deadweight computations for the piles without and with a water column above it respectively.

$$F_{t,\text{dw}} = L_t \cdot \left( \rho_s \cdot \frac{1}{4} \pi \phi_t^2 + \rho_{\text{con}} \cdot \frac{1}{4} \pi (\phi_t - 2t_t)^2 \right) \cdot g$$ \quad (G.16)

$$F_{t,\text{dw,submerged}} = L_t \cdot \left( (\rho_s - \rho_w) \cdot \frac{1}{4} \pi \phi_t^2 + (\rho_{\text{con}} - \rho_w) \cdot \frac{1}{4} \pi (\phi_t - 2t_t)^2 \right) \cdot g$$ \quad (G.17)

In which:

- $L_t \quad [m]$ Pile length
- $\rho_s \quad [kg/m^3]$ Mass density of steel
- $\rho_{\text{con}} \quad [kg/m^3]$ Mass density of concrete
- $\rho_w \quad [kg/m^3]$ Mass density of water
- $\phi_t \quad [m]$ Pile diameter
- $t_t \quad [m]$ Wall thickness of the tube outer steel
G.3 Foundation of the Concrete Door

The foundation of the concrete door consists of a sill constructed out of prestressed prefabricated concrete slabs, which are supported on a pile foundation. The concrete slabs form a sill structure, for which the shape enables guidance of the gate during movement and load transmission during operation. In this section first the design of the sill will be presented after which the pile foundation is designed, associated with the Concrete Sliding Door Alternative.

G.3.1 Design of the sill

The initial design of the sill has the shape as presented in Figure G.1, for which the accompanying dimensions are listed in Table G.2. This initial design contains two sides which restrict the gate from lateral movement, guiding it through the channel. This design however, is very sensitive to siltation due to the bowl-shaped top in which sediment gets trapped. Therefore, the final sill design will have a slightly different shape, as presented in Figure 5.15 of Section 5.6, reducing the probability of sedimentation by a spherical top. The taken steps however, are comparable for either the final design or the initial design.

![Figure G.1: Dimensions of the sill, representative values are shown by Table G.2](image)

G.3.2 Pile foundation design

As introduced in Section G.2, the type of piles used for the pile foundation are steel close-ended tubular piles which are installed by driving. The number and diameter of these piles greatly depend on the required bearing capacity of the pile foundation and thus on the loads acting on it. First the design load will be determined after which a suitable pile foundation is designed.
Table G.2: Sill dimensions, the resulting cross sectional area and the location of the Neutral Axis. See Figure G.1 for an explanation of the different parts.

<table>
<thead>
<tr>
<th>Dimensions of sill</th>
<th>Cross-sectional area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum sill width $w_{sill}$ m</td>
<td>30</td>
</tr>
<tr>
<td>Slide track width $w_{slide}$ m</td>
<td>15</td>
</tr>
<tr>
<td>Slope width $w_1$ m</td>
<td>1.5</td>
</tr>
<tr>
<td>Sediment collector width $w_2$ m</td>
<td>0.5</td>
</tr>
<tr>
<td>Side width $w_3$ m</td>
<td>4</td>
</tr>
<tr>
<td>Side extension $w_4$ m</td>
<td>1.5</td>
</tr>
<tr>
<td>Side declination $w_5$ m</td>
<td>3</td>
</tr>
<tr>
<td>Side to body $w_6$ m</td>
<td>3</td>
</tr>
<tr>
<td>Body declination $w_7$ m</td>
<td>3</td>
</tr>
<tr>
<td>Body width $w_8$ m</td>
<td>12</td>
</tr>
<tr>
<td>Bottom level $h_{sill}$ m</td>
<td>-28.5</td>
</tr>
<tr>
<td>Slide track thickness $h_{slide}$ m</td>
<td>1</td>
</tr>
<tr>
<td>Side height $h_1$ m</td>
<td>3</td>
</tr>
<tr>
<td>Extension height $h_2$ m</td>
<td>1.5</td>
</tr>
<tr>
<td>Declination height $h_3$ m</td>
<td>1</td>
</tr>
<tr>
<td>Base height $h_4$ m</td>
<td>1</td>
</tr>
</tbody>
</table>

Table G.3: Design loads on the pile foundation for the concrete barrier alternative

<table>
<thead>
<tr>
<th></th>
<th>At Dry Dock</th>
<th>At the Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sill weight $q_{sill,sw}$ kN/m</td>
<td>3,182</td>
<td>1,877</td>
</tr>
<tr>
<td>Additional weight by water above sill $q_{sill,water}$ kN/m</td>
<td>0</td>
<td>312</td>
</tr>
<tr>
<td>Total weight distributed over $y$ (ULS) $q_{sill,y}$ kN/m</td>
<td>348</td>
<td>362</td>
</tr>
<tr>
<td>Added vertical design load by gate $q_{sill,y}$ kN/m</td>
<td>220</td>
<td>263</td>
</tr>
<tr>
<td>Vertical design load per full slab (y) $q_{sill,y,piles}$ kN/m</td>
<td>11,350</td>
<td>12,500</td>
</tr>
<tr>
<td>Horizontal design load per full slab (y) $q_{sill,y,piles}$ kN/m</td>
<td>0</td>
<td>2,350</td>
</tr>
<tr>
<td>Vertical design load per full slab as point $F_z$ kN</td>
<td>340,000</td>
<td>375,000</td>
</tr>
<tr>
<td>Horizontal design load per full slab al spoint $F_y$ kN</td>
<td>0</td>
<td>70,400</td>
</tr>
</tbody>
</table>

Design loads

The total load that needs to be transmitted by the pile foundation to the bearing soil layer results from the loads acting on the gate, the dead weight of the gate and the deadweight of the sill. In addition to the loads related by bearing the gate, there is also the load of the water column on top of the sill. The values of the different load components are presented in Table G.3, as well as the resulting design load per 20 m slab.

Table G.4: Pile design

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile diameter $\phi_{t}$ m</td>
<td>2.50</td>
</tr>
</tbody>
</table>

Pile design

To decrease the construction complexity but support repetition and construction time, all piles will have the same diameter. Since the bearing capacity will depend on both the diameter and the pile length, a distinction will be made between bearing depths of 64 m and 54 m. The chosen pile diameter $\phi_{t} = 2.50$ m, giving the pile characteristics as presented in Table G.4.
Table G.4: Characteristics of the installed piles.

<table>
<thead>
<tr>
<th>Pile bearing depth:</th>
<th>MSL – 64 m</th>
<th>MSL – 54 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical span</td>
<td>$L_z$ m</td>
<td>35.5</td>
</tr>
<tr>
<td>Pile length</td>
<td>$L_t$ m</td>
<td>37.4</td>
</tr>
<tr>
<td>Cross sectional area steel tube</td>
<td>$A_t$ $m^2$</td>
<td>0.194</td>
</tr>
<tr>
<td>Total area of tip</td>
<td>$A_{t,tip}$ $m^2$</td>
<td>4.91</td>
</tr>
<tr>
<td>Outer diameter</td>
<td>$\phi_t$ m</td>
<td>2.5</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>$t_t$ m</td>
<td>0.025</td>
</tr>
<tr>
<td>Inclination angle of pile</td>
<td>$\alpha_t$ [$^\circ$]</td>
<td>71.6</td>
</tr>
<tr>
<td>Inclination of pile</td>
<td>$\tan(\alpha_t)$ [-]</td>
<td>0.33</td>
</tr>
<tr>
<td>Number of piles per slab</td>
<td>$n_t$ [-]</td>
<td>12</td>
</tr>
</tbody>
</table>

Pile bearing capacity

Within the two explained depths a distinction will be made between the dry dock and the channel, for which the elements will be submerged and loaded by a water column. The results of the pile bearing capacity computations are presented in Table G.5.

Table G.5: Bearing capacity computations, for pile diameter $\phi_t = 2.5$ m.

<table>
<thead>
<tr>
<th>Total pile bearing capacity: $F_{t,max}$ kN</th>
<th>MSL – 64 m</th>
<th>MSL – 54 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Dock</td>
<td>154,300</td>
<td>128,900</td>
</tr>
<tr>
<td>Channel</td>
<td>111,600</td>
<td>90,700</td>
</tr>
<tr>
<td>Pile tip bearing $F_{t,tip}$ kN</td>
<td>147,800</td>
<td>120,100</td>
</tr>
<tr>
<td>Positive skin friction $F_{t,skin+}$ kN</td>
<td>12,580</td>
<td>12,580</td>
</tr>
<tr>
<td>Negative skin friction $F_{t,skin-}$ kN</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Pile dead weight $F_{t,dw}$ kN</td>
<td>6,080</td>
<td>3,748</td>
</tr>
<tr>
<td>Maximum soil bearing $P_{t,tip}$ kN/m²</td>
<td>30112</td>
<td>24459</td>
</tr>
<tr>
<td>Undrained shear stress of clay $\sigma_{su,i}$ kN/m²</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td>Initial verticla effective stress $\sigma_{vi,i}$ kN/m²</td>
<td>286</td>
<td>286</td>
</tr>
<tr>
<td>Bearing capacity factor (surcharge) $N_q$ [-]</td>
<td>64.20</td>
<td>64.20</td>
</tr>
<tr>
<td>Shape factor (surcharge) $s_q$ [-]</td>
<td>1.64</td>
<td>1.64</td>
</tr>
<tr>
<td>Inclination factor (surcharge) $i_q$ [-]</td>
<td>1.00</td>
<td>0.81</td>
</tr>
<tr>
<td>Skin friction $F_{t,skin+}$ kN</td>
<td>12,580</td>
<td>12,580</td>
</tr>
<tr>
<td>Soil layer thickness (stiff clay) $w_{s,i}$ m</td>
<td>38.0</td>
<td>38.0</td>
</tr>
<tr>
<td>Skin friction pressure: (cohesive) $P_{skin,c}$ kN/m²</td>
<td>34.6</td>
<td>34.6</td>
</tr>
<tr>
<td>Initial verticla effective stress at clay $\sigma_{vi,c}$ kN/m²</td>
<td>133</td>
<td>133</td>
</tr>
<tr>
<td>Undrained shear stress of clay $\sigma_{su,c}$ kN/m²</td>
<td>36.0</td>
<td>36.0</td>
</tr>
<tr>
<td>Adhesion factor $\alpha_s$ [-]</td>
<td>0.96</td>
<td>0.96</td>
</tr>
<tr>
<td>Soil layer thickness (dense sand) $w_{s,s}$ m</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Skin friction pressure: (non cohesive) $P_{skin,s}$ kN/m²</td>
<td>71.8</td>
<td>71.8</td>
</tr>
<tr>
<td>Neutral soil pressure coefficient $K_n$ [-]</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Initial verticla effective stress $\sigma_{vi,s}$ kN/m²</td>
<td>286</td>
<td>286</td>
</tr>
<tr>
<td>Pile dead weight $F_{dw}$ kN</td>
<td>6,080</td>
<td>3,748</td>
</tr>
<tr>
<td>Length of inclined steel tubular piles $L_t$ m</td>
<td>37.4</td>
<td>37.4</td>
</tr>
<tr>
<td>Mass density of steel $\rho_s$ kN/m³</td>
<td>78.0</td>
<td>68.0</td>
</tr>
<tr>
<td>Mass density of concrete $\rho_c$ kN/m³</td>
<td>24.5</td>
<td>14.5</td>
</tr>
</tbody>
</table>
Table G.6: Characteristic capacities per pile

<table>
<thead>
<tr>
<th>Pile bearing depth:</th>
<th>Location:</th>
<th>MSL − 64 m</th>
<th>MSL − 54 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dry Dock</td>
<td>Channel</td>
<td>Dry Dock</td>
</tr>
<tr>
<td>Bearing capacity compression piles</td>
<td>$F_{t,c,\max}$ kN</td>
<td>154,300</td>
<td>128,900</td>
</tr>
<tr>
<td>Bearing capacity tension piles</td>
<td>$F_{t,t,\max}$ kN</td>
<td>18,660</td>
<td>16,330</td>
</tr>
<tr>
<td>Vertical bearing capacity factor</td>
<td>$C_V$ [-]</td>
<td>0.949</td>
<td>0.949</td>
</tr>
<tr>
<td>Horizontal bearing capacity factor</td>
<td>$C_H$ [-]</td>
<td>0.316</td>
<td>0.316</td>
</tr>
<tr>
<td>Vertical resistance compression piles</td>
<td>$R_{t,c,\max}$ kN</td>
<td>146,400</td>
<td>122,300</td>
</tr>
<tr>
<td>Vertical resistance tension piles</td>
<td>$R_{t,t,\max}$ kN</td>
<td>-17,700</td>
<td>-15,500</td>
</tr>
<tr>
<td>Horizontal resistance compression piles</td>
<td>$R_{t,c,\max}$ kN</td>
<td>48,800</td>
<td>40,800</td>
</tr>
<tr>
<td>Horizontal resistance tension piles</td>
<td>$R_{t,t,\max}$ kN</td>
<td>5,900</td>
<td>5,164</td>
</tr>
</tbody>
</table>

Strength check

To determine the strength of the total foundation, the resistances per pile have to be multiplied by the number of piles bearing loads in that specific direction. This has been done for 10 piles bearing in both vertical ($z$) and horizontal ($y$) direction, distinguishing compression and tension piles. The results are presented in Table G.7, as a total per slab. Table G.7 gives the design loads for these two directions as well. A comparison between the load and the bearing capacities for each direction, shows that all bearing capacities are greater than the loads, so the strength of the foundation should be sufficient. The smallest 'Unity Check' (dividing the strength by the load) can be found for the vertical bearing capacity at the channel with a pile bearing depth of MSL − 54 m (most right column).

The table shows that the greatest bearing capacities can be found for bigger pile bearing depths. Since the bearing depths for the piles will vary between MSL − 64 m and MSL − 54 m, it will be assumed that the bearing capacities will vary between the capacities associated with these two depths, presented in Table G.7. According to this, the complete foundation should be strong enough to bear the design loads associated to a 1/1000 years storm, from which one can conclude that the foundation has been designed properly.

Table G.7: Strength check of the concrete pile foundation

<table>
<thead>
<tr>
<th>Pile bearing depth:</th>
<th>Location:</th>
<th>MSL − 64 m</th>
<th>MSL − 54 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dry Dock</td>
<td>Channel</td>
<td>Dry Dock</td>
</tr>
<tr>
<td>Vertical design load per full slab as point</td>
<td>$F_{z,piles}$ kN</td>
<td>340,600</td>
<td>374,900</td>
</tr>
<tr>
<td>Horizontal design load per full slab al spoint</td>
<td>$F_{y,piles}$ kN</td>
<td>0</td>
<td>70,400</td>
</tr>
<tr>
<td>Vertical bearing capacity per slab</td>
<td>$R_{z,piles}$ kN</td>
<td>643,500</td>
<td>533,900</td>
</tr>
<tr>
<td>Horizontal bearing capacity per slab</td>
<td>$R_{y,piles}$ kN</td>
<td>273,500</td>
<td>229,600</td>
</tr>
</tbody>
</table>

G.4 Foundation of the Steel Barrier

G.4.1 Sill

In case of the Composite barrier, the imposed loads will are acting at the side corners of the triangular truss. As a result of this, all loads will be transmitted to the sides of the sill. This results in a very slender body of the sill and more robust sides (footings). The final sill design is presented in Chapter of the main report, in this Appendix some initial steps are outlined to end with the presented design.
G.4.2 Pile foundation

The pile foundation will be designed following design rules provided by Eurocode and the Dutch NEN [Geotechniek 2014]. The required formulas to determine the bearing capacity of the piles is introduced in Section G.2, this section describes the results of the iterative design steps. First the loads will be determined and subsequently the pile foundation is designed. Some input variables are presented in Table G.8.

Table G.8: Design values for the relevant parameters for computing the pile tip bearing capacity.

<table>
<thead>
<tr>
<th>Layer: Stiff Clay</th>
<th>Dense Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location: A-A' B-B'</td>
<td></td>
</tr>
<tr>
<td>Thickness of soil layer $h_{s,i}$ [m]</td>
<td>38 28</td>
</tr>
<tr>
<td>Effective initial vertical stress $\sigma'_{vi,i}$ [kN/m²]</td>
<td>133 98</td>
</tr>
<tr>
<td>Adhesion factor $\alpha_s$ [-]</td>
<td>0.96 0.82</td>
</tr>
<tr>
<td>Neutral soil pressure coefficient $K_n$ [-]</td>
<td>- 0.5</td>
</tr>
<tr>
<td>Skin friction pressure $P_{t,skin}$ [kN/m²]</td>
<td>34.6 29.7</td>
</tr>
</tbody>
</table>

Load situations

The load on the foundation depends on the situation. Two situations will be distinguished:

1. Normal condition - Average water levels, the channel is open for navigation.

2. 1/1000 years storm conditions - 8 m storm surge, gate is closed and rests on the foundation.

The loads for these normal conditions do not vary as much as the loads during storm conditions. Additionally there is not a significant horizontal force acting on the foundation during these normal conditions, so probably only vertical declined piles will be required for these conditions. All though only compression forces are expected due to the absence of horizontal forces, the base pile foundation will be constructed using the same type of piles as the inclined piles, with an inclination of 1/3, to enhance the contribution to horizontal loads during storm events. Additionally, the use of the same piles at all locations reduces the complexity of the foundation structure. Repetition - repeating the same construction procedures - may cause a decrease in construction time resulting in lower construction costs. [Horst 2011]

It is expected that the loads during storm events are the much bigger than during the normal conditions. Therefore the requirements of the pile foundation will be determined for the loads associated to a 1/1000 years storm, as presented in Figure G.2.

---

1 The sill design and load transmission are different in the final design. The checks presented in this Section are just as investigation of different alternatives.
To determine the permanent loads, the sill will be divided into two different sections: The footing and the body, which both will be loaded by the self weight of the sill and the load of the water column above the sill, as presented in Figure G.2-b. The characteristic values for these two loads have been determined in Appendix F, the design values will be obtained using the factors \( \gamma_{water} = 1.2 \) and \( \gamma_{sw} = 1.0 \) for the loads by water and self weight respectively. By the use of factor \( \gamma_{water} \) the increasing height of the water column is taken into account, when the gate blocks the drainage of the river runoff and local rainfall. The results in the following design values: [Vrijling et al., 2015; Verruijt, 2001; Geotechniek, 2014; Abede and Smith, 1994; Tol, 2003]

\[
q_{d,water} = q_{h,water} \gamma_{water} = 220 \cdot 1.2 = 264 \text{kN/m}^2
\]  
\[
q_{d,sw} = \frac{\rho_c - \rho_w}{10} \gamma_{sw} \cdot g \cdot h_{sill} = \frac{2500 - 1025}{1000} \cdot 1.0 \cdot 9.81 \cdot h_{sill} = 14.5h_{sill} \text{kN/m}^2
\]

Where \( h_{sill} \) is the local thickness of the sill which differs for the body and the footing, resulting in the following load combinations:

\[
q_{d,body} = q_{d,water} + q_{d,sw, body} = 242 + 14.5 \cdot 2.0 = 271 \text{kN/m}^2
\]

\[
q_{d,footing} = q_{d,water} + q_{d,sw, footing} = 242 + 14.5 \cdot 6.0 = 329 \text{kN/m}^2
\]
The variable loads on the barrier have been determined in Appendix F and Chapters 3 and 5 for a 1/1000 year storm surge, in which load factors $\gamma_i$ are implemented already. With the use of the MatrixFrame computer software, the transmission of the design loads from concrete slab to the steel H-girders to the steel cross sections has been simulated as presented in Figures 6.3 and 6.9 in Chapter 6. The steel truss transmits the loads to the foundation as presented in Figure G.2.

When the gate is in closed position, significant horizontal loads occur due to the hydraulic head difference, wave impact and wind load at the gate surface. These horizontal loads are transmitted to the two footings of the foundation. The horizontal loads have to be transmitted to the bearing soil layer at a depth of 60 m, which requires inclined piles. For the inclined pile foundation design an inclination of $z/y = 3/1$ will be used for all piles, giving an inclination angle $\theta_t = 18.4^\circ$ and $\tan\theta_t = 1/3$ with respect to the vertical. The assumption is made that the inclined piles will penetrate 4 m into the dense sand layer.

### Design of the pile foundation

Using the preliminary sill design as presented in Figure G.2 and a pile foundation of 4 inclined piles per footing a set of computations has been made to check the strength and stability of the foundation, with the use of the introduced equations for the pile bearing capacity varying the pile diameter and the pile distance in the x-axis. According to the first checks, the foundation does not meet the vertical strength requirements as long as there are no additional piles to bear the sill body. When the self weight of the sill body and the water column above the sill do not contribute to the load on the footings however, a pile foundation of 4 inclined piles with a diameter $\phi_t = 2.0$ m and a transversal distance between the piles of 10 m in x-direction, does meet the requirements. The results of the computation for this pile configuration are presented in Table G.10, a presentation of the piles at each footing is given in Figure 6.12-(b) in which the shape of the footing is optimized and more piles can be added at each footing in order to bear the sill body. The associated bearing capacity per pile is given in Table G.10.

<table>
<thead>
<tr>
<th>Barrier type: Driving depth: 54 m</th>
<th>Driving depth: 64 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dock</td>
<td>Channel</td>
</tr>
<tr>
<td>Cross section (steel) $A_t$ $m^2$</td>
<td>0.124</td>
</tr>
<tr>
<td>Total cross sectional area $A_{t,tip}$ $m^2$</td>
<td>2.011</td>
</tr>
<tr>
<td>Outer diameter $\phi_t$ m</td>
<td>1.6</td>
</tr>
<tr>
<td>Wall thickness $t_t$ m</td>
<td>0.025</td>
</tr>
<tr>
<td>Inclination of pile $\tan(\alpha_t)$</td>
<td>0.33</td>
</tr>
<tr>
<td>Number of piles (y) $n_t$</td>
<td>8</td>
</tr>
<tr>
<td>Pile distance (h.t.h.) $\Delta y_t$ m</td>
<td>3.3</td>
</tr>
<tr>
<td>Pile distance(h.t.h.) $\Delta x_t$ m</td>
<td>10</td>
</tr>
<tr>
<td>Capacity compr. piles $F_{t,c,max}$ kN</td>
<td>49092</td>
</tr>
<tr>
<td>Capacity tension piles $F_{t,t,max}$ kN</td>
<td>3292</td>
</tr>
<tr>
<td>Horizontal resistance/footing $R_{y,foot}$ kN</td>
<td>33155</td>
</tr>
<tr>
<td>Vertical resistance/footing $R_{z,foot}$ kN</td>
<td>86962</td>
</tr>
<tr>
<td>Horizontal load/footing $F_{y,foot,d}$ kN</td>
<td>0</td>
</tr>
<tr>
<td>Vertical load/footing $F_{z,foot,d}$ kN</td>
<td>53487</td>
</tr>
<tr>
<td>$F_{z,foot,d,B}$ kN</td>
<td>11168</td>
</tr>
<tr>
<td>Horizontal UC: $R_{y,foot,d}/F_{y,foot,d} &gt; 1$</td>
<td>OK</td>
</tr>
<tr>
<td>Vertical UC: $R_{z,foot,d}/F_{z,foot,d} &gt; 1$</td>
<td>OK</td>
</tr>
</tbody>
</table>

Table G.9: Dimensions and spacing of the used piles, for the channel sections and the dry docks at bearing depths of 54 m and 64 m.
### Table G.10: Computation of the associated bearing capacities per pile.

<table>
<thead>
<tr>
<th>Barrier type:</th>
<th>Driving depth: 54 m</th>
<th>Driving depth: 64 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing capacity $F_{t,max}$ kN</td>
<td>49092</td>
<td>50497</td>
</tr>
<tr>
<td>Tip bearing $F_{t,tip}$ kN</td>
<td>45800</td>
<td>45808</td>
</tr>
<tr>
<td>Positive skin friction $F_{t,skin^+}$ kN</td>
<td>5245</td>
<td>6557</td>
</tr>
<tr>
<td>Negative skin friction $F_{t,skin^-}$ kN</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Death weight of the pile $F_{t,dw}$ kN</td>
<td>1953</td>
<td>1868</td>
</tr>
</tbody>
</table>

### Table G.11: Tip bearing computations

<table>
<thead>
<tr>
<th>Barrier type:</th>
<th>Driving depth: 54 m</th>
<th>Driving depth: 64 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tip bearing $F_{t,tip}$ kN</td>
<td>45800</td>
<td>45808</td>
</tr>
<tr>
<td>Maximum soil bearing $P_{t,tip} kN/m^2$</td>
<td>22779</td>
<td>14581</td>
</tr>
<tr>
<td>Undrained shear stress $\sigma_{su,i} kN/m^2$</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td>Initial vertical eff. stress $\sigma_{vi,i} kN/m^2$</td>
<td>216</td>
<td>216</td>
</tr>
<tr>
<td>Bearing capacity factor $N_q [-]$</td>
<td>64.20</td>
<td>64.20</td>
</tr>
<tr>
<td>Shape factor $s_q [-]$</td>
<td>1.64</td>
<td>1.64</td>
</tr>
<tr>
<td>Inclination factor $i_q [-]$</td>
<td>1.00</td>
<td>0.64</td>
</tr>
</tbody>
</table>

### Table G.12: Skin friction computations

<table>
<thead>
<tr>
<th>Barrier type:</th>
<th>Driving depth: 54 m</th>
<th>Driving depth: 64 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skin Friction $F_{t,skin^+}$ kN</td>
<td>5245</td>
<td>6557</td>
</tr>
<tr>
<td>Clay friction pressure: $P_{skin,c} kN/m^2$</td>
<td>29.5</td>
<td>29.5</td>
</tr>
<tr>
<td>Clay layer thickness $w_{s,c}$ m</td>
<td>28.0</td>
<td>28.0</td>
</tr>
<tr>
<td>Initial vertical effective stress $\sigma_{vi,c} kN/m^2$</td>
<td>98.0</td>
<td>98.0</td>
</tr>
<tr>
<td>Undrained shear stress of clay $\sigma_{su,i} kN/m^2$</td>
<td>36.0</td>
<td>36.0</td>
</tr>
<tr>
<td>Adhesion factor $\alpha_s [-]$</td>
<td>0.82</td>
<td>0.82</td>
</tr>
<tr>
<td>Sand friction pressure: $P_{skin,s} kN/m^2$</td>
<td>54.2</td>
<td>54.2</td>
</tr>
<tr>
<td>Sand layer thickness: $w_{s,s}$ m</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Initial vertical effective stress $\sigma_{vi,c} kN/m^2$</td>
<td>216.0</td>
<td>216.0</td>
</tr>
</tbody>
</table>

### Table G.13: Computations for the pile dead weight. Note that the density of the material depends on weather or not the structure is submerged.

<table>
<thead>
<tr>
<th>Barrier type:</th>
<th>Driving depth: 54 m</th>
<th>Driving depth: 64 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile dead weight $F_{t,dw}$ kN</td>
<td>1953</td>
<td>1868</td>
</tr>
<tr>
<td>Pile length $L_t$ m</td>
<td>28.5</td>
<td>28.5</td>
</tr>
<tr>
<td>Mass density steel $\gamma_s kN/m^3$</td>
<td>78</td>
<td>68</td>
</tr>
<tr>
<td>Mass density concrete $\gamma_c kN/m^3$</td>
<td>25</td>
<td>14</td>
</tr>
</tbody>
</table>
The final design of the foundation is slightly different than presented in this section. As a result of another sill design, the load transmission through the foundation will be different. However, the resulting requirements presented in this Appendix are comparable to the requirements of the final design. The design steps are similar as well, which has resulted in a final design of the foundation as presented in Section 6.6 of the main report.

The internal load distribution for the final design of the foundation slabs, has been modelled by the use of computer software *MatrixFrame*, giving the images shown by Figure G.3. For the body of the foundationslab, *Matrixframe* gives a maximum vertical deformation of about 0.03 m, as a result of the selfweight of the conrete and the water column above.

![Image of load distribution](image)

Figure G.3: Internal load distribution of the foundation slabs: (a) Stresses in the concrete, a maximum value of $\sigma_c \approx 7.5 \text{ N/mm}^2$ can be seen near the connections between the slab body and footings; (b) Support reactions.

### G.4.3 Reinforcement

For the foundation slabs it is expected that reinforcement is required. According to Eurocode 2 ([Bamforth et al., 2008](Bamforth+etal.2008)) the following checks related to the reinforcement need to be performed:

1. sufficient reinforcing steel exist for the recording of the bending moments;
2. shear capacity;
3. punch resistance.

#### Bending Moment Reinforcement

The maximum bending moment is expected to occur at the connection between the sill footings and the sill body. Where the moment resulting from the distributed load related to the selfweight of the sill body $G_{body}$ and the water column $Q_{water}$ above the sill is present. This results in the following load combination.

$$q_{d, body} = \gamma G_{body} + \gamma Q_{water}$$  \hspace{1cm} (G.22)
In which:

- \( G_{\text{body}} \) [\( kN/m^2 \)] Load by submerged self weight: \( = t_{\text{body}} \cdot (\gamma_c - \gamma_w) \);
- \( Q_{\text{water}} \) [\( kN/m^2 \)] Load by water column: \( = h_{\text{water}} \cdot \gamma_w \);
- \( \gamma_G \) [-] Partial load factor for permanent loads, Section 6.3: \( \gamma_G = 1.10 \);
- \( \gamma_Q \) [-] Partial load factor for variable load, Section 6.3: \( \gamma_Q = 1.143 \);
- \( \gamma_c \) [\( kN/m^3 \)] Density of the concrete: \( \gamma_c = 25 \) kN/m\(^3\);
- \( \gamma_w \) [\( kN/m^3 \)] Density of water: \( \gamma_w = 10 \) kN/m\(^3\);
- \( t_{\text{body}} \) [\( m \)] Thickness of the concrete body;
- \( h_{\text{water}} \) [\( m \)] Height of the water column above the concrete.

For a body thickness \( t_{\text{body}} = 2.0 \) m and a maximum height of the water column at the protected channel side \( h_{\text{water}} = 4.0 \) m, this results in a design load \( q_{d,\text{body}} = 405 \) kN/m\(^2\). For a maximum body length of the sill \( L_{\text{body}} = 37 \) m, the maximum bending moments occurring in the center of the sill \( M_{Ed,\text{span}} \) and \( M_{Ed,\text{foot}} \) are:

\[
M_{Ed,\text{span}} = \frac{1}{24} q_{d} L_{\text{body}} = 23,100; \quad M_{Ed,\text{foot}} = -\frac{1}{12} q_{d} L_{\text{body}} = -46,200 \quad [kN/m] \quad (G.23)
\]

The bending moment capacity \( M_{Rd} \) is the result of the internal moment provided by the concrete compression force \( N_c \) and the reinforcement tensile force \( N_s \) as expressed by the set of equations presented below. To meet the requirement \( M_{Rd} > M_{Ed} \), an amount of reinforcement steel \( A_{s,\text{span}} = 0.03 \) m\(^2\)/m and \( A_{s,\text{foot}} = 0.06 \) m\(^2\)/m is required for the center of the sill and the connection with the footings respectively. A base reinforcement of \( \Phi 60 – 100 \) with secondary reinforcement \( \Phi 2 – 50 \) should be sufficient, where at the footings a second layer of the base reinforcement should be added. This results in a reinforcement ratio \( \rho_l = A_{s}/A_{c} = 2\% \) at the connections between body and footings.

\[
M_{Rd} = N_s \cdot 0.4 t_{\text{body}} + N_c \cdot (0.5 t_{\text{body}} - \beta x_u) \quad (G.24)
\]

\[
N_s = f_{yd}A_s; \quad N_c = \alpha c bx_u f_{ccd}; \quad \sum N_i = 0 \rightarrow N_s = N_c \quad (G.25)
\]

In which:

- \( f_{ccd} \) [\( N/mm^2 \)] Design compression strength of the concrete C45/55;
- \( f_{yd} \) [\( N/mm^2 \)] Design yield strength of the reinforcement steel B500;
- \( \alpha_c \) [-] Area factor of concrete: \( \alpha_c = 0.75 \);
- \( \beta \) [-] Depth ratio of concrete: \( \beta = 0.35 \);
- \( x_u \) [\( mm \)] Height of the concrete compression zone.

**Shear Reinforcement**

The shear capacity can be described using Equation (G.26)

\[
v_{Rd,c} = 0.12k(100\rho f_{ck})^{1/3} \geq 0.035k^{1.5}f_{ck}^{0.5} \quad (G.26)
\]

Where for concrete class C45/55 and effective concrete area of \( A_{c,eff} = b_w \cdot d = 12.5 \cdot 12.5 = 156 \) m\(^2\), the following input variables are found:

\[
k = 1 + \sqrt{200/d} = 1.13 \leq 2; \quad \rho_l = \frac{A_{s}}{A_{c}} \approx 0.02; \quad f_{ck} = 45 N/mm^2 \quad (G.27)
\]
Which in combination with Equation (G.26) result in a shear capacity \( v_{Rd,c} = 0.61 \text{ N/mm}^2 \). The design shear load can be determined using Equation (G.28) and results for the foundation of the steel truss barrier in a maximum shear stress \( v_{Ed,c} = 0.85 \text{ N/mm}^2 \). Since \( v_{Rd,c} < v_{Ed,c} \) the application of shear reinforcement is required. Equation (G.29) gives the required shear reinforcement, which for the expressed variables results in \( A_{sw} = 9.77 \text{ mm}^2 \) for reinforcement bars with spacing \( s = 1.0 \text{ m} \).

\[
v_{Ed,c} = \frac{V_{Ed}}{b_{wz}} \quad \text{(G.28)}
\]

\[
A_{sw} = \frac{v_{Ed} b_{w}}{f_{ywd} \cot \theta} \quad \text{(G.29)}
\]

In which:
- \( A_{sw} \) \([\text{m}^2]\) Shear reinforcement area;
- \( s \) \([\text{m}]\) Spacing of reinforcement bars;
- \( f_{ywd} \) \([\text{N/mm}^2]\) Yield strength of the reinforcement steel (S500) \( f_{ywd} = 435 \text{ N/mm}^2 \);
- \( \theta \) \([\text{rad}]\) Angle of internal concrete tensile tie \( \theta = 2.5 \text{ rad} \).

**Punching Shear Reinforcement**

Since the piles of the pile foundation are installed relatively close to each other, the punching shear check will be done for a pile group instead of for a single pile. As presented, the pile group will consist of 6 piles \( \Phi_t = 2.0 \text{ m} \) with a width of \( \Phi_t \) and a total length 12.5 \text{ m} . According to Fennis (2012) the basic control perimeter is set at 2 times the width of \( \Phi_t \) from the loaded area. Where shear reinforcement is required, the shear resistance is the sum of the concrete and shear reinforcement resistances. This shear resistance can be determined by the use of the following expression:

\[
v_{Rd,cs} = 0.75 v_{Rd,c} + 1.5 \frac{d}{s_r} A_{sw} f_{ywd,eff} \frac{\sin \alpha}{u_1 d} \quad \text{(G.30)}
\]

In which:
- \( A_{sw} \) \([\text{m}^2]\) area of shear reinforcement in each perimeter around the pile;
- \( s_r \) \([\text{m}]\) Radial spacing of layers of shear reinforcement;
- \( \alpha \) \([\text{rad}]\) Angle between the shear reinforcement and the plane of the slab;
- \( f_{ywd,eff} \) \([\text{N/mm}^2]\) Effective design strength of the punching shear reinforcement;
- \( d \) \([\text{m}]\) Mean effective depth of the slabs.

The shear stress should always be smaller than the shear resistance according to the following expression:

\[
v_{Ed} = \frac{\beta V_{Ed}}{u_d} < v_{Rd,c} = 0.636 \text{N/mm}^2 \quad \text{(G.31)}
\]

For each pile group we find for factor \( \beta = 1.4 \) \([-]\), shear load \( V_{Ed} = 133,200/3 = 44,400 \text{ kN} \) and slab thickness \( d \approx 6.0 \text{ m} \). For column perimeter \( u_0 \), basic perimeter \( u_1 \) and outer perimeter (where reinforcement is no longer required) \( u_{out} \) the following set of equations is used:

\[
u_0 = 2(c_x + c_y); \quad u_1 = 2(c_x + c_y) + 2\pi \cdot 2d; \quad u_{out} = \frac{\beta V_{Ed}}{d v_{Rd,c}} \quad \text{(G.32)}
\]

\[
u_0 = 2(2.0 + 12.5) = 29\text{m}; \quad u_1 = 29 + 2\pi \cdot 12.0 = 104\text{m}; \quad u_{out} = \frac{1.4 \cdot 44,400}{6.0 \cdot 636} = 16.3\text{m} \quad \text{(G.33)}
\]
The resulting values show that \( u_{\text{out}} < u_0 \), of which it can be concluded that no punching shear reinforcement is required. This can in the first place be checked by rearranging the value for the outer perimeter. The position of this outer perimeter can be determined giving the Equation \[ u_{\text{out}} = 2(c_x + c_y) + 2 \pi \cdot r_{\text{out}} \rightarrow r_{\text{out}} = \frac{u_{\text{out}} - 2(c_x + c_y)}{2 \pi} \] (G.34)

To have a second check at the statement that punching shear reinforcement is not required, the shear capacities at the column \( u_0 \) and at basic perimeter \( u_1 \) are checked. Shear capacity at the column is limited by expression \[ v_{Rd,max} = 7.38 \text{ N/mm}^2 \] where the resulting shear stress \[ v_{Ed} = 0.36 \text{ N/mm}^2 \]. For the basic perimeter \( u_1 \) we find \[ v_{Rd,c} = 0.636 \text{ N/mm}^2 \] and \[ v_{Ed} = 0.10 \text{ N/mm}^2 \]. So for both locations \( v_{Rd,c} > v_{Ed} \), which confirms that punching shear reinforcement is not required. It can be concluded that the foundation slab is somewhat overdimensioned, the thickness of the foundation footings may be reduced in order to optimize the design.

\[ v_{Rd,max} = 0.5 \cdot 0.6(1 - \frac{f_{ck}}{250} \cdot \frac{\alpha_c}{\gamma_M} \cdot f_{ck}) \] (G.35)

### G.5 Seepage

Due to the hydraulic head difference, a porous flow may occur in the soil below the structure its foundation which may cause erosion. This phenomenon is called seepage, and should be avoided. A number of methods has been delivered in the past century to determine the issue of seepage for the design of a structure. The two most useful methods for a water retaining structure are Bligh’s Creep Theory (1930) and Lane’s Weighted Creep Theory (1934), which will be presented in this section.

#### Bligh’s Creep Theory

According to Bligh’s theory, water creeps along the bottom contour of the structure, which is in this case the sill. The method of Bligh is slightly different than the newer theory provided by Lane. However, they both make use of Bligh’s coefficient \( C \), which depends on the soil type.

#### Lane’s Weighted Creep Theory

The theory of Lane contains the conclusion that horizontal creep is less effective in reducing uplift than vertical creep. Therefore, this theory uses a factor \( \gamma_h = 1/3 \) for the horizontal creep, where vertical creep is multiplied with the factor \( \gamma_v = 1 \). The formula for total seepage length is than:

\[ L = \sum L_{h,i}/3 + \sum L_{v,i} \] (G.36)

The structure meets the seepage requirements as long as: \( L > CH \). Where \( H \) is the hydraulic head and \( C \) is the coefficient that depends on the soil type, for stiff clay Lane uses \( C = 2 \). Combined with the hydraulic head of 8 m this results in \( CH = 16 \text{ m} \). For the foundation of the concrete barrier, Equation (G.36) results in a seepage length \( L_c = 23 \text{ m} \) where for the foundation of the steel barrier a seepage length \( L_s = 31 \text{ m} \) is found. For both foundations the seepage length is larger than the factor \( CH \), so the foundations do not require an additional seepage screen.